United States
Department of
Agriculture

Forest Service

Engineering Staff

Washington, DC

September 1994

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FHWA-FLP-94-006

Retaining Wall Design Guide

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Technical Report Documentation Page

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This study was part of the Coordinated Federal Lands Highway Technology Implementation Program (CTIP).

16. Abstract

This report was developed to assist the engineer and manager in planning and utilizing geotechnical engineering information, in the decisionmaking process for selecting an appropriate type of retaining wall. It serves as a technical reference, summarizing the fundamentals of design for retaining walls used by low volume road agencies. The design guide presents a compendium of standard design, including sample calculations, standards and specifications.

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Corrections to the USDA-Forest Service Retaining Wall Design Guide;
(EM-7170-14 and FHWA-FLP-94-006)
First update July 1999
Second update February 2001

Note: After the page number the number in () refers to date the modification was updated

Page 76 (7/99)

Change equation 3-12 to the following;

$$Ka = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Page 126 (2/01)

Change equation 3-38 to the following;

$$q_d = c\,N_c + \gamma D_f N_q + 1/2\,\gamma B\,N_\gamma$$

Page 196 (7/99)

Change equation 3-107 to the following;

$$\leq \frac{[qL + \gamma_r(H)L]\tan\phi}{q(H)K_{ab} + 0.5\gamma_b(H^2)K_{ab}}$$

Page 196 (7/99)

Change equation 3-110 to the following;

$$L \geq \sqrt{\frac{FS_{OT}(K_{ab})H^2[\gamma_b(H) + 3q]}{3[q + \gamma_r(H)]}}$$

<u>Page 205 (7/99)</u>

Change the FHWA corrosion rates to the following;

15 μm/yr = 0.59 mil/yr for the first two years
4 μm/yr = 0.16 mil/yr for subsequent years

reference: *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines*, FHWA-SA-96-071, page 78

<u>Page 205 (7/99)</u>

Change the carbon steel loss to the following;

12 μm/yr = 0.47 mil/yr after zinc depletion

reference: *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines*, FHWA-SA-96-071, page 78

<u>Page 265 (7/99)</u>

Change the first line under 4B.2.2 Internal Stability to the following;

<u>Consider internal stability</u>-follow procedure outlined on pages 139 to 147.

<u>Page 269 (7/99)</u>

Change the title of the drawing to the following;

Cross-Section of Proposed Concrete Cantilever Wall

<u>Page 269 (7/99)</u>

Delete the following;

Scale 1" = 20'

Page 277 (7/99)

Change the calculation of P_{ah} to the following;

$P_{ah} = P_a\cos\beta = (16.68 \text{ kips/ft})\cos 38° = 13.14 \text{ kips/ft}$

Page 278 (7/99)

Change the value of P_{ah} in calculating the sum of moments (ΣMc) to 13.14 kips/ft

Page 278 (7/99)

Change the equation when calculating y_V to the following;

$$y_V = \frac{(78.84 + 5.7 + 52.85)}{25.7} \text{ ft}$$

Page 278 (7/99)

Change the calculation for determining the factor of safety for sliding to the following;

$$= \frac{(0.47)(10.24 + 15.45)}{13.14} = 0.92 < 1.5$$

Page 281 (7/99)

Change the calculation for determining the factor of safety for sliding to the following;

$$= \frac{(0.47)(10.24 + 15.45) + 7.02}{13.14} = 1.45$$

Page 289 (2/01)

Change the first bullet in Step 7.3 to the following;

Horizontal earth pressure (σ_x) can be calculated at each layer using the equation in Step 7.4 on page 291.

Page 291 (7/99)

After the second bullet S_x is not being calculated, σ_x is being calculated.

Page 291 (7/99)

Change the first sentence of the third bullet to the following;

Calculate P_H, the horizontal load, that needs to be resisted by each reinforcement layer.

Page 293 (7/99)

Change the first sentence of the second bullet in Step 7.5 to the following;

A 3.4-mil coating is applied to the wires for corrosion protection.

Page 294 (7/99)

Change the calculations for corrosion to the following;

FHWA guidelines state;

> 0.59 mil/yr loss of zinc for the first two years
> 0.16 mil/yr loss of zinc for subsequent years
> 0.47 mil/yr loss of carbon steel

Determine the number of years required to corrode zinc corrosion protection, y

3.4 mil = 0.59 mil/yr (2yr) + 0.16 mil/yr (y)

$$y = 13.9 \text{ yrs}$$

- Calculate the radius of carbon steel at the end of the design life, T_c,

$$T_c = T_n - T_s$$

where T_n is the radius of the wire at construction, which equals half the wire diameter, d/2.

$$T_n = \frac{d}{2} = \frac{0.0176 \text{ ft}}{2} = \frac{0.211 \text{ in}}{2} = 0.106 \text{ in}$$

Ts is the thickness of the wire lost during the structures design life due to corrosion of the carbon steel.

$$T_s = 0.47 \text{ mil/yr} (75 \text{ yr} - 13.9 \text{ yr}) = 28.7 \text{ mils} = 0.0287 \text{ in}$$

$$T_c = 0.106 \text{ in} - 0.0287 \text{ in} = 0.0773 \text{ in}$$

- The area of steel wire remaining after 75 years is A_C.

$$A_C = (T_c)^2 \pi = (0.0773 \text{ in})^2 \pi = 0.0188 \text{ in}^2$$

- According to the results shown in the table on page 292, the highest tensile force occurs in layer 9.

$$F_c = P_H / A_C = \frac{1043 \text{ lb/wire}}{0.0188 \text{ in}^2} = 55{,}479 \text{ lb/in}^2$$

- Check factor of safety against corrosion, FS_C.

$$FS_c = F_y / F_C = \frac{65{,}000 \text{ psi}}{55{,}479 \text{ psi}} = 1.17 \geq 1.0 \text{ (satisfy corrosion)}$$

Page 295 (7/99)

Change the fifth conclusion to the following;

Factor of safety against corrosion (FS_c) was 1.17 and was above the minimum value of 1.0.

Page 297 (2/01)

Change the equation for vertical spacing calculations (S_z) to;

$$S_z = \frac{F_a}{\sigma_x (FS_p)}$$

Page 298 (2/01)

Change the equation under the first bullet of Step 7.3 to calculate the tensile stress felt by the reinforcement to the following;

$$\sigma_x = K_o(\gamma_r Z + q)$$

Page 302 (2/01)

Change the equation under the second bullet of Step 7.1 to calculate active case for earth pressure to the following;

$$K_{ar} = \tan^2(45 - \phi_r / 2)$$

Page 302 (2/01)

Change the equation under the fifth bullet of Step 7.2 to check overturning to the following;

$$FS_{OT} = \frac{3(L^2)(\gamma_k)(H)}{K_{ar}(H^2)[\gamma_r(H) + 3q]}$$

Page 303 (2/01)

Change under the first bullet under EXAMPLE 3: KEYSTONE/TENSAR WALL to;

Try H = 2.0 ft (three blocks)

Page 305 (2/01)

Change the resultant for T_i after the first bullet under (for layer 1 at $z = 13.67$ ft)

$= 771$ lb/ft

FOREWORD

This study was funded as a part of the Coordinated Federal Lands Highway Technology Implementation Program. It is intended to serve the immediate needs of those who design and construct Federal Lands Highways, but it is also made available to all other interested parties.

This Design Guide streamlines and standardizes the art and science of slope management. It is intended to assist engineers and managers in planning and utilizing geotechnical information in the decision making process for selecting an appropriate type of retaining wall.

for Thomas O. Edick P.E.
Federal Lands Highway Program Administrator
Federal Highway Administration

NOTICE

Preface to the Second Edition

The second edition of the *Retaining Wall Design Guide* has been produced by the USDA Forest Service. The guide is an updated version of the original produced by Mr. David Driscoll of Geotechnical Resources, Inc., Portland, Oregon, in 1978. Funding for this revision was provided by the Federal Highway Administration (FHWA) through the Coordinated Technology Implementation Program (CTIP).

The principal changes to this edition are the emphasis on mechanically stabilized backfill (MSB), mechanically stabilized earth (MSE), or reinforced soil type wall systems, their design, application, and advantages. For the past decade the vast majority of new retaining structures constructed have been reinforced soil systems, such as welded wire and chainlink fencing walls, reinforced fills, reinforced earth or VSL walls, and geotextile walls. Geogrid and geotextile reinforced walls have used a variety of facing materials including timbers, modular concrete blocks, tires, or hay bales. They are generally cheaper to construct than traditional gravity structures because of their relatively low materials costs, easily transportable materials, and simple erection.

The design guide is divided into four chapters:

(1) Chapters 1 and 2 are intended to aid the engineering manager to plan and the management to use geotechnical engineering information in the decisionmaking process for selecting an appropriate type of retaining wall.

(2) Chapter 3 is a complete technical reference that summarizes the fundamentals of design for retaining walls used by low volume road agencies.

(3) Chapter 4 presents a compendium of standard designs that includes sample calculations, standards, and specifications.

Many people have assisted in the preparation of this design guide. The project leader for the Forest Service was John Mohney; contributors for the Forest Service were Doug McClelland, Gordon Keller, Richard Vandyke, Bob Young, Mark Truebe, Sandra Wilson-Musser, Rob Piehl, Karel Broda, and Mike Long. The section on reinforced soil walls was prepared under the direction of David Thielen of GeoEngineers, Portland, Oregon. Dr. Jonathan T. H. Wu of the University of Colorado, Denver, also contributed to the design of reinforced soil walls.

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Greek Symbols

α	Slope of back of wall with backfill.
β	Angle of backfill from horizontal.
β'	Effective backfill slope angle for transitioned backfill.
$\Delta\sigma H$	Change in magnitude of horizontal stress due to surcharge.
$\Delta\sigma V$	Change in magnitude of vertical stress due to surcharge.
δ	Friction angle between soil and wall.
δ_b	Friction angle of wall base and foundation material.
Υ	Unit weight of soil.
Υ_b	Buoyant unit weight of soil.
Υ_t	In-place density of backfill.
ϕ	Effective stress friction angle.
ϕ_a	Actual angle of internal friction of backfill material.
ϕ_r	Reduce internal friction angle for use in analysis of stack sack walls.
ϕ_r	Residual effective stress friction angle.
ϕ_t	Soil-tie friction angle.
ρ	Slope of soil failure plane.
ρ	Reinforcement ratio, A_s/bd.
ρ_i	Inclination of tangent to failure surface on slice i.
ρ_a	Active earth pressure.
σ_h	Horizontal earth pressure.
σ_{mom}	Moment-induced stress.

Preceding page blank

$\overline{\sigma_n}$	Effective normal stress on failure plane.
σ_p	Passive earth pressure.
σ_v	Vertical earth pressure at heel of wall.
τ	Shear strength along failure plane.

English Symbols

A_p	Tie rod force.
S_s	Area of reinforcing steel.
A_t	Cross-sectional area of tie.
a	Reinforced concrete design coefficient.
B	Foundation width.
B'	Width of gravity structure at a distance *h* from the top of the wall.
b	Width of concrete beam.
b	Point of zero shear below depth *m*.
C	Coefficient converting vertical stress magnitude to horizontal stress magnitude due to surcharge.
C	Ratio of effective width of soldier pile to pile spacing.
C_s	Coefficient of minimum reinforcement.
c	Soil cohesion or undrained shear strength.
c	Depth to zero shear in the upper portion of the retaining wall.
c_w	Cohesion between soil and wall.
$\bar{c}$	Effective stress cohesion.
$\overline{c_b}$	Cohesion or adhesion of the wall and foundation material.
D_f	Depth to base of shallow foundation.
D_{15F}, D_{85F}, D_{15B}, D_{50B}, D_{50F}	Effective diameter of the soil particles at which the numerical portion of the subscript passes that sieve. *B* or *F* refers to backfill or filter, respectively.
d	Effective depth of concrete section.
d	Free-standing height of retaining wall.

d Depth from top of wall to fabric layer.

d Depth from top of wall to level of anchors.

d Tie rod diameter.

d_f Depth from top of wall to overlap of fabric.

d_r Diameter of reinforcing steel bar.

e Eccentricity of resultant force on the base of a gravity structure from the centerline.

F Concrete section coefficient.

F_a Allowable axial stress.

F_b Allowable bending stress in steel member.

F_r Resistance of fabric pullout.

FS_y Factor of safety against yield failure of anchor tie material.

f_a Actual axial stress.

f_b Actual bending stress.

f_c Maximum concrete comprehensive stress.

f_t Maximum concrete tensile stress.

f_y Steel yield strength.

f_c' Ultimate concrete comprehensive stress.

H Effective wall height.

H_c Critical height of unsupported vertical cut.

H_1 Height of backfill slope above effective wall height.

h Height of wall above section at which stresses are being considered in a gravity structure.

h Location of resultant force P_h above depth m.

h Height of deadman.

h Depth of backfill surface to anchor level at anchor pile.

h_c Theoretical depth of tension crack in backfill.

h_p Pressure head.

h_w Height of perched water table above the heel of the wall.

h_z Depth of intersection of active and passive failure wedges.

I_f, I_e Influence factor.

jd Distance between resultant tensile and compressive forces in a reinforced concrete beam.

K Reinforced concrete design coefficient.

K Earth pressure coefficient.

K_H Backfill coefficient—horizontal component.

K_a Active earth pressure coefficient.

K_v Backfill coefficient—vertical component.

K_o Coefficient of at-rest earth pressure.

K_p Passive earth pressure coefficient.

kd Depth from top of beam to neutral axis—concrete compressional depth.

L Length of deadman.

L_e Embedded length of fabric to resist fabric pullout.

L_i Total length of stabilizer.

L_o Length of fabric overlap.

ΔL_i Length of failure for surface of slice *i*.

l_g Grouted length of rock anchors.

l_l Length of stabilizer beyond failure plane.

M Moment of resultant *N* about the base of the retaining wall.

M_A Bending moment at point *A*.

M_{max} Maximum bending moment.

m Ratio $\frac{x}{H}$.

m Depth from dredge line to point-of-zero earth pressure.

N Resultant force acting on the base of the retaining wall.

N.A. Neutral axis.

N_c, N_q, $N\gamma$ Bearing capacity factors.

n Ratio $\frac{z}{H}$.

o Perimeter of reinforcing bar.

P Soil pressure against skin wall—Stack Sack walls.

P_a Resultant active earth force.

P_h Resultant horizontal force acting on the back of the wall.

P_p Resultant passive earth force.

P_T Backfill thrust.

p $\frac{y}{x}$ equals ratio of lateral distance and distance to wall from point load to the point at which stresses are being computed.

p_a Equivalent fluid pressure—active state.

p_p Equivalent fluid pressure—passive state.

ΔP_h Change in magnitude of resultant horizontal force due to surcharge.

q Strip load intensity—force/area.

q Uniform surcharge load.

q_a Allowable bearing pressure.

q_d Ultimate bearing capacity.

q_d' Bearing capacity based on local shear.

q_l Magnitude of line load.

q_p Magnitude of point load.

q_u Unconfined compressive strength of rocks.

Δq Magnitude of uniform surcharge load.

R Individual design consideration rating.

R	Resultant frictional force on trial wedge.
R	Height of resultant above bottom of wall face.
R	Resultant load acting on the base of a gravity wall.
R	Anchor force per unit length of wall.
R_{max}	Maximum anchor tension per unit length of anchor wall.
R'_{max}	Maximum anchor tension reduced for interaction of active and passive failure wedges.
r	Radius of timber pile.
S	Section modulus of the wall section.
S	Vertical stabilizer spacing—Stack Sack walls.
S	Horizontal tie spacing—reinforced backfill walls.
S_f	Fabric strength.
S_{min}	Minimum section modulus of wall section.
s	Soldier pile spacing.
T	Horizontal resistance to backfill thrust.
T	Tensile stress in fabric.
T_f	Thickness of footing of concrete cantilever wall.
T_n	Resistance to pull out of stabilizer *i*.
t	Thickness of stem concrete cantilever wall.
t	Thickness of tie strip.
t	Thickness of lagging between H-piles.
U	Pore water pressure.
u	Allowable concrete bond stress.
V	Shear in skin wall.
V	Allowable shear stress in concrete.
W	Weight.

W Width of strip load.

W_e Effective width of solider pile.

W_n Weight of wedge of soil in failure wedge *n*.

X Horizontal distance from wall face to the location of a surcharge load.

X Vertical spacing of anchors.

x Depth of embedment of driven pile retaining walls (measured below point of zero earth pressure).

Δx_i Horizontal width of slice *i*.

Y Lateral distance along wall face to point at which stress is being computed.

Y Depth below dredge line to point of zero moment.

Z Depth from top of wall to point at which stress is being computed.

Z_i Depth from top of wall to stabilizer *i*.

z Height of earth pressure reversal at the tip of driven pile retaining walls.

z Depth below top of wall.

Chapter 1
Introduction

1A Purpose

This design guide has been developed in an effort to streamline and standardize the art and science of slope management. Specifically, it has been structured to accomplish the following tasks:

(1) To provide an organized procedure by which engineers and managers can select and design the most economical and appropriate structure to solve a given slope management problem.

(2) To help management better utilize geotechnical engineering input in selecting an appropriate retaining wall (see chapter 2).

(3) To provide a complete reference on the design of retaining walls used by low volume road agencies in steep terrain. Chapter 3 is directed towards the practicing civil or geotechnical engineer who will be responsible for the technical aspects of wall design. It includes discussions of earth pressures, backfill considerations, factors of safety, stability calculations, and material properties and behavior.

(4) To provide a compendium of standard designs and specifications commonly used by low volume road agencies. Chapter 4 can be used by the designer to complete wall designs according to the procedures presented in chapter 3. Sample designs given for selected wall types include calculations, standards of design, and specifications.

A major reason for this second edition is to emphasize the technology and advantages of mechanically stabilized backfill (MSB) type systems using both generic designs and those available from manufacturers which are commonly used today. A wide variety of designs and facing systems can be used with MSB systems. The understanding of and technology of earth reinforcement is rapidly changing, but use of these systems is well accepted and very cost effective today.

The first edition of this guide contained design information on walls that are no longer in use, which included, for example, Stack Sack, horizontal sheet pile, and vertical culvert. While introductory information on these walls was left in chapter 2, the design information has been deleted from chapter 3.

1B Slope Management

Effective slope management seeks economical and environmentally sound solutions to stability problems in steep terrain. Common slope management problems include maintenance of cut slope stability, improvement of slope stability, and control of fill stability. There are numerous approaches and combinations of approaches to the management of slopes, such as building retaining structures, rerouting, draining slopes (with surface, trench, and horizontal drains), cutting and reshaping slopes, filling, rock bolting, and consolidation grouting. In selecting the proper method of slope management, consideration of many factors must be considered, including engineering feasibility, long-term performance, economics, visual impact, intended use, future demands, environmental considerations, construction time and geologic and geotechnical considerations.

A comparison of hypothetical alternative solutions to three common steep terrain construction problems is presented in figure 1-1, "Slope Management." The problems considered are: construction of a cut section for a road across a hillside; stabilization of a landslide through an existing road, and construction of or reconstruction of a fill section for a road across a hillside.

A brief evaluation of the nonstructure and retaining wall solutions for each of the slope management cases presented in figure 1-1 leads to the following general conclusions:

(1) Retaining structures generally result in a smaller amount of disturbance to the landscape and environment than nonstructure treatment methods.

(2) Retaining structures generally require smaller amounts of excavation and filling than nonstructure treatments.

(3) Retaining structures and other slope management methods must be engineered.

Other considerations in selection of the most desirable method of solving slope stability problems are related to each site. A detailed discussion of the other selection criteria is presented in chapter 2.

The remainder of this design guide is predicated upon the assumptions that the use of a nonstructural solution to the slope management problem has been evaluated and eliminated, and that a retaining wall is required to meet project goals and objectives.

1C Lessons from Retaining Wall Failures

This section documents over 30 retaining wall failures that have taken place over the past 25 years (see table 1-1). These failures occurred on low volume road systems, primarily Federal and rural county systems in the northwestern United States, from the central Sierras to Montana This represents the experience of 10 individuals. Undoubtedly there

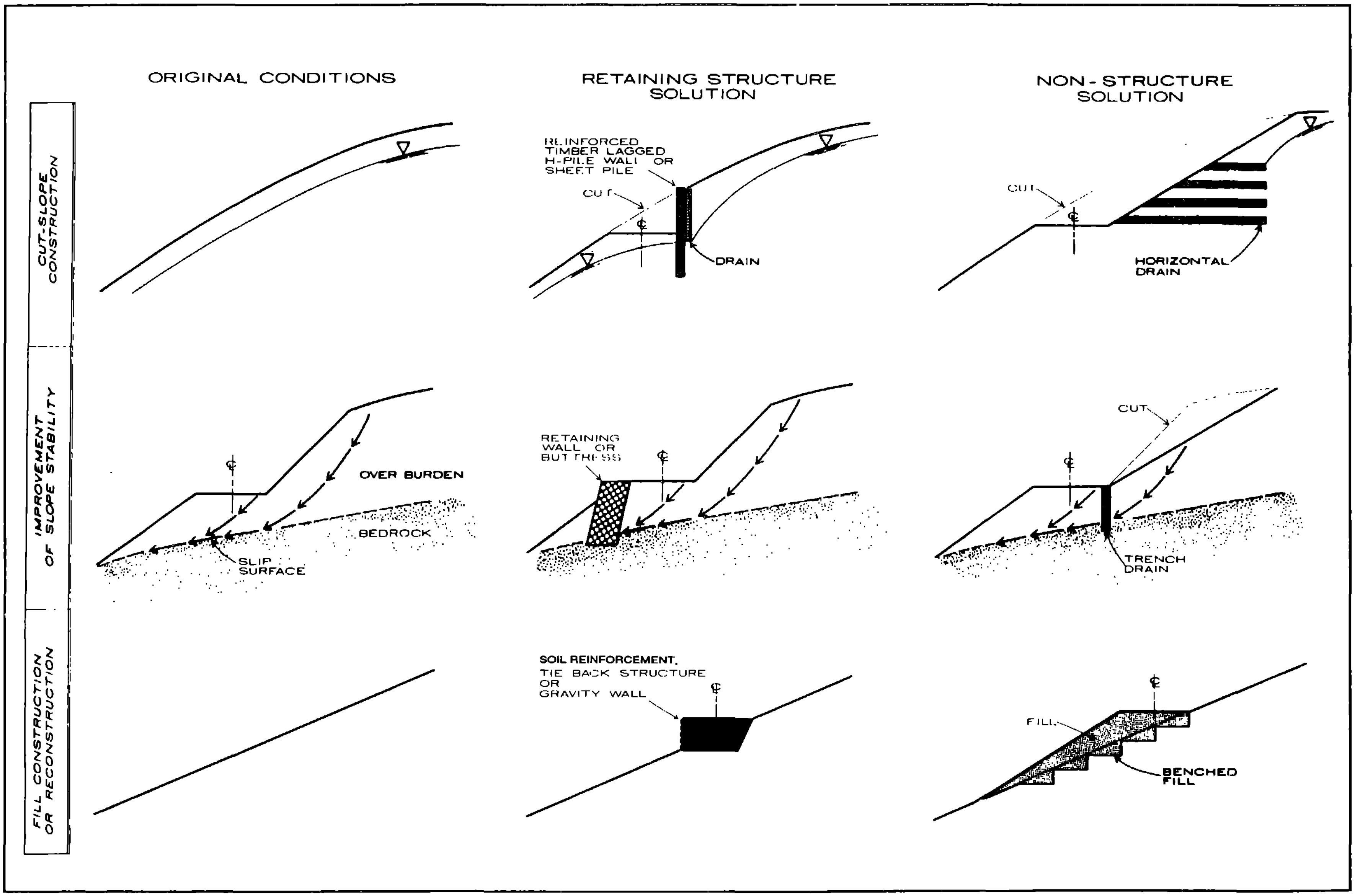


Figure 1-1.—Slope management.

have been many others in the northwest as well as the rest of the United States. Table 1-1 is a summary of the teams' experience.

A review of the table reveals the following reasons for the number of failures:

(1) Many walls are designed for steep, rugged terrain with no geotechnical investigation or involvement in construction. Most of these sites involve a combination of poor soils, wet conditions, steep slopes, and slope stability problems.

(2) Many designers are not familiar with requirements of walls in steep terrain. As a result, wall ends are not tied into solid natural ground, adequate bench width or embedment is not provided, and poor quality soils are used for backfill and walls are not properly drained.

(3) Adequate construction inspection is not provided. Many inspectors are not familiar with wall design and are unable to make design changes needed in the field. Often the contractor is left on his or her own, with no guidance from the agency. Important aspects of construction such as compaction, drainage, and review of foundation conditions prior to erection of the wall are unheeded.

(4) Many contractors for low volume road agencies are inexperienced in wall construction; therefore, some walls are designed and built by maintenance crews.

Walls do not fail due to some inadequacy in the wall system itself, particularly if it is a proprietary system. Proprietary systems are very conservatively designed and the failures are usually foundation, backfill, and drainage related.

Most low volume road agencies do not have geotechnical personnel on their staffs. When designing retaining walls for steep, wet, or unstable conditions the agency will save money by getting geotechnical help either from another agency or a consultant. Geotechnical help should be brought in at the beginning of the project to use their expertise throughout the design. Geotechnical experts should also be called on to review foundation conditions when they are exposed during construction and for any redesign decisions that needs to be implemented during construction.

Table 1-1.—Lessons from Retaining Wall Failures.

Wall Type	Type of Failure	Situation	Result
Bin wall	Foundation	The toe of the wall was undercut by the river. Poor location and no scour protection were provided.	The backfill emptied from the bins. The sheet pile was driven at the toe of the binwall at a cost of $100,000.
Bin wall	Raveling of backfill	Clean, fine sand was used as backfill. Traffic vibrations caused the sand to sift out over time.	Large voids formed inside the bins. The backfill was removed and replaced.
Concrete crib	Structural	There was no steel reinforcing in the ends of crib members. Members were a pirated design of a national brand produced by a nonlicensed supplier.	The ends of members cracked.
Bin wall	Structural	The backfill was dumped into bins without adequate compaction. It bridged, causing a void in the lower part of the bins.	The unsupported members were crushed.
Bin wall	Structural	The bins were installed upside down so light sections were on the bottom.	Bottom members failed. The wall had to be replaced.
Bin wall	Sliding/overturning	Stumps and boulders used in backfill were not compacted.	The wall tilted and moved laterally.
Bin wall	Structural	The wall was erected on a rock foundation, and the contractor later blasted a ditch.	All members were mangled and the wall was completely destroyed.
Bin wall	Foundation	The wall on a steep slope was not embedded deep enough.	Erosion on the slope caused the backfill to empty from the bins.
Gabion	Global stability	No geotechnical investigation was done, and the wall was built on an active slide.	The wall was replaced due to total wall failure.
Gabion	Foundation	No geotechnical investigation was done. The foundation conditions were unsuitable.	The wall settled and tilted, and it had to be replaced with more embedment.
Gabion	Foundation	No geotechnical investigation was done. The wall was founded on side cast material consisting of soil and logs.	As the side cast material settled, the wall slid laterally and tilted.

Table 1-1.—Lessons from Retaining Wall Failures (Continued).

Wall Type	Type of Failure	Situation	Result
Gabion	Foundation/global stability	The wall was built on a 30-foot, loosely compacted fill.	There was total wall failure.
Anchored H-pile	Structural/overturning	Due to the inexperience of the inspector, piles were bottomed in loose soil, rather than being driven to the recommended blow counts. Also, the tie rod connected to the deadman was poor.	Wall movement put extra stress in the ties, causing them to break. The wall toppled over and had to be replaced.
Anchored H-pile	Horizontal movement	The piles were extended during construction by an inexperienced inspector who did not recheck the design.	The wall had to be buttressed and the anchor replaced and extended.
Anchored H-pile	Structural	The wall had cables for the tie backs to an anchor pile. These cables corroded and broke in 16 years.	Though still in use, the anchors failed and moved slightly outwards. The anchors have to be repaired.
Cantilever H-pile	Overturning	Due to an inexperienced inspector, all piles were driven to the same elevation rather than to the depth required for stability. There was not enough embedment for the center piles.	There was total wall failure.
Cantilever H-pile	Vandalism	This wall was at the toe of a cut slope to catch slope ravel. Wood lagging in the flanges was not secured.	The lagging was stolen and had to be replaced. Clips were welded to the flanges to lock in the lagging.
Cantilever H-pile	Sliding	The wall was built on an active slide site.	The wall is functional, but it is moving with the slide.
Cantilever H-pile	Overturning	No geotechnical investigation was done. The wall was not embedded deep enough and did not have a thorough design.	Total wall failure within 1 year.
Cantilever H-pile	Overturning	Due to an inexperienced inspector, piles were placed in shallow dug holes rather than driven in.	The wall had to be replaced.
Welded wire	Sagging	Local soil rather than pea gravel was used in the wall face. The compaction could not be obtained.	The face has sagged, but the wall is functional.

Table 1-1.—Lessons from Retaining Wall Failures (Continued).

Wall Type	Type of Failure	Situation	Result
Welded wire	Foundation	The wall was built too close to the edge of a steep slope.	The wall is functional, but its face has sagged.
Welded wire	Bulge in the lower part of the wall	Poorly drained, clayey gravel backfill was used in wet conditions. Adequate drainage was not provided.	The top part of the wall was rebuilt. A bulge remains in the lower part.
Welded wire	Slump	The wall had poor quality backfill (15 percent clay), and the drain system was designed but not installed.	There was complete wall failure. It had to be rebuilt with quality backfill.
Welded wire	Foundation	The foundation was not inspected by the engineer prior to backfilling. The foundation failed draining backfill from the wall.	A concrete foundation was built under one end of the wall. The wall was rebuilt.
Welded wire	Foundation/sliding	No geotechnical investigation was done. The wall was built on loose side cast soil, and no drainage was provided.	The wall completely failed and it had to be rebuilt.
Reinforced backfill, geotextile reinforcement tire-faced	Loss of backfill through face	When tires are not staggered, and the batter is vertical, soil can be lost through the holes. Tires must be staggered, and the wall battered at 1/4:1.	Backfill could be lost.
Reinforced earth	Foundation	Inadequate geotechnical investigation. The hole was drilled 5 feet into a boulder, which was mistaken for bedrock.	The boulder moved and the entire wall failed.
Reinforced backfill, geotextile reinforcement gunite-faced	Erosion at the end of the wall	The designer did not "ground truth" the wall profile. Wall ends were not extended far enough into the ground.	The ends of the wall settled, causing cracks in the concrete facing.

Table 1-1.—Lessons from Retaining Wall Failures.

Wall Type	Type of Failure	Situation	Result
Reinforced backfill, several walls, geotextile and chain-link reinforcement	Secondary compression of backfill	The walls were backfilled with wood chips to reduce weight.	These walls are functioning; however, due to the continuing settlement of the wood chips, the walls need high pavement maintenance.
Reinforced backfill, chain-link reinforcement	Aesthetic	Backfill was dumped against the face rather than hand-raked.	Each face was pushed out over the lower layer, resulting in a reverse batter. The wall is functional, but it is not aesthetically appealing.
Reinforced backfill, geotextile reinforcement	Reduced safety factor	The supplier sent a lighter geotextile than was specified. This reduced the factor of safety to an unacceptable level.	The lower portion of the wall was buttressed to provide the required factor of safety.

Chapter 2
Wall Selection and Design Procedure

2A Introduction

A well-designed retaining structure should be safe, stable, economical, practical to build, and aesthetically and environmentally harmonious with its surroundings.

The process of selecting retaining structures that best fulfill the needs of a given situation consists of the following eight steps:

(1) The problem is identified.

(2) The problem is defined and reviewed by the geotechnical engineer or engineering geologist who completes a preliminary site investigation/field summary as outlined in appendix A and assesses potential solutions and costs.

(3) A preliminary site investigation/field summary is forwarded to the responsible engineering manager for review. The report details problems based upon information gathered in step 2, stressing the risks and uncertainties related to subsurface conditions and the relative urgency of the situation.

(4) The responsible engineering manager then develops a summary report based upon staff reports. The report includes economic and recreation values, desired permanency of solution, availability of funds, allowable construction methods, and environmental concerns.

(5) The geotechnical and design engineer, under the direction of the engineering manager, will either proceed with the process, with a preliminary design based upon presumptive design parameters, or expand the investigation to obtain additional information.

(6) The geotechnical and design engineer provides the responsible engineering manager with preliminary designs for several possible solutions. This report includes rough costs, relative risks, factors of safety, construction problems, required construction time, and other information about the possible wall types.

(7) The solutions are evaluated, and the management makes a final selection.

(8) The geotechnical and design engineer completes the final design, construction plans, standards and specifications, and submits the final design to the responsible engineering manager for review.

Note that some problems will not fit the process presented above. Due to constraints of time and economics, certain phases of the decision process may be simplified or eliminated.

The following subsections briefly describe the 14 wall types under consideration and present detailed discussions of the various aspects of design that management must consider. Emphasis will be placed upon geotechnical problems. Step 7, previously mentioned, will also be explained in detail in the final section of this chapter.

2B Wall Types

2B.1 Background

Based upon the mechanics of retaining wall performance, there are four basic wall classifications: MSB, tieback, driven cantilever pile, and gravity. Figure 2-1 shows schematic drawings of the mechanics of how each wall type develops resistance to react against the imposed lateral earth pressure. Example (a) shows an MSB wall. The force exerted on the wall facing is transferred into the backfill by the frictional resistance developed along the horizontal metal or fabric reinforcing strips. The gravity wall shown in example (d) must have sufficient gross weight, *W*, to develop the horizontal force, *T*, to resist the total thrust from the backfill, P_T. In the case of the anchored structure, example (b), the thrust from the backfill, P_T, is carried from the wall facing to the soil and/or rock behind the assumed failure plane by the anchor rods. Example (c) shows a cantilever pile wall. In this case, the force of P_T is resisted by the structural properties of the piles and the passive resistance of the soil. A combination of the four basic wall systems is also possible.

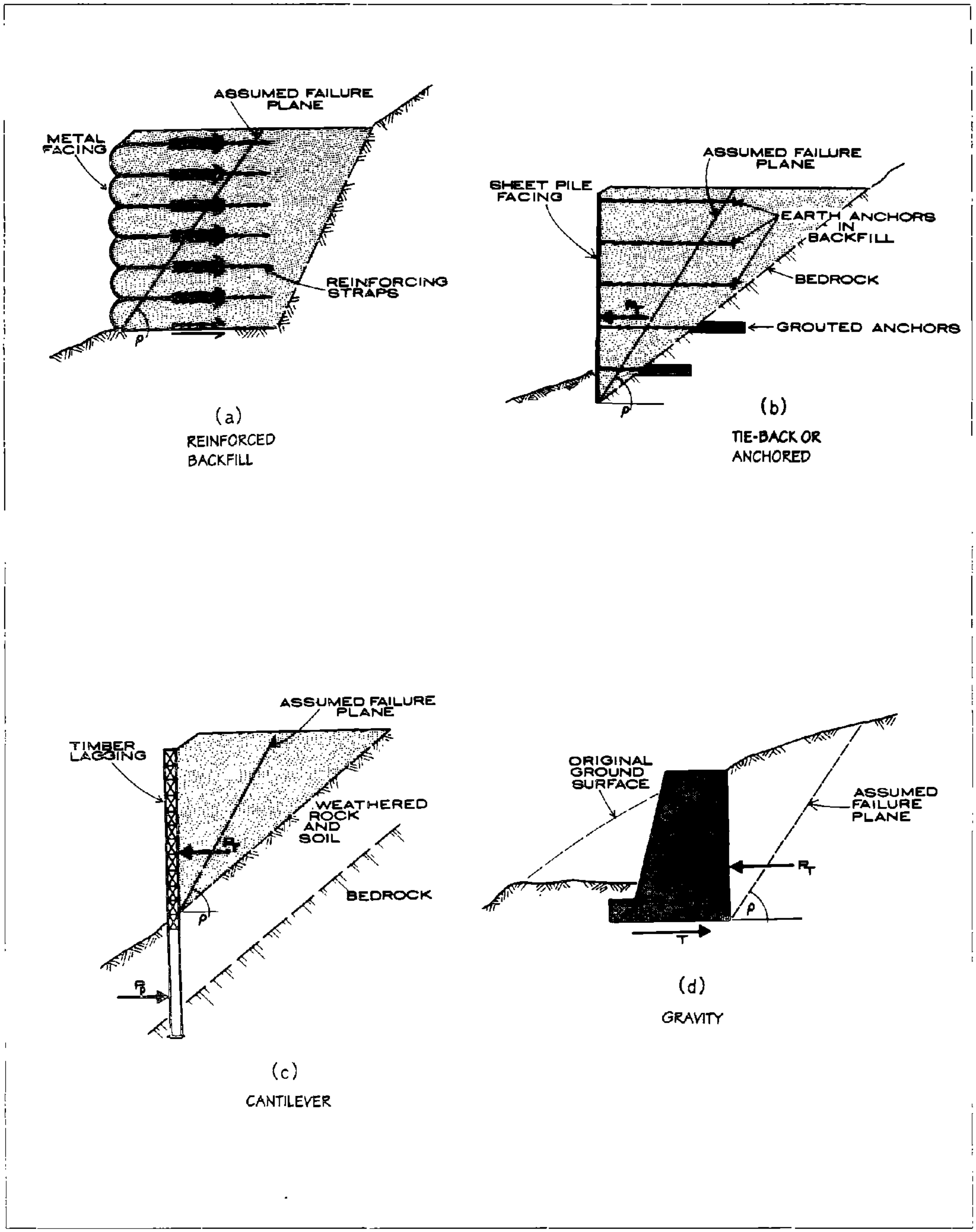


Figure 2-1.—Mechanics of wall systems.

Of the many types of retaining structures constructed by low volume road agencies in the past several years, 14 have been selected for evaluation in this design guide. Table 2-1 presents a tabulation of the 14 walls with respect to the mechanics of the design and the probable behavior of the construction medium. (Concrete cantilever walls have been classified as gravity structures for convenience.)

Table 2-1.—Wall classification.

Mechanically Stabilized Backfill (Reinforced Soil)	**Anchored**
Reinforced Earth	H-pile, timber lagged
VSL	Vertical sheet pile
Geosynthetic	Stack Sack
Stack Sack	All gravity structures
Modular block	
Welded wire	
Gravity	**Cantilever Piles**
Bin walls	Vertical sheet piles
Rectangular	H-pile, timber lagged
Circular	
Cross-tied	
Concrete crib	
Timber cribs	
Gabions	
Concrete gravity	
Concrete cantilever*	

Several of the anchored walls and the Stack Sack wall have received dual classifications in table 2-1. In the case of pile walls, this is because tied back designs are common in addition to cantilever designs. The Stack Sack wall is a hybrid design that develops its stability from a combination of backfill reinforcement and tied back anchoring. Essentially any gravity structure can be tied back to increase the factor of safety against overturning or sliding.

For purposes of discussion in this guide, retaining walls have also been classified as "standard" and "nonstandard." A standard wall is typically constructed from "off-shelf" materials according to preexisting designs developed by industry or State and Federal agencies. Standard wall systems have been used extensively by low volume road agencies.

A nonstandard design is one that requires a custom design. Table 2-2 shows the reclassification of the 14 walls evaluated in this guide.

Table 2-2.—Standard and nonstandard wall designs.

Standard Walls	**Nonstandard Walls**
Bin walls	H-piles, lagged
Rectangular	Geosynthetic
Circular	Vertical sheet pile
Cross-tied	Anchored (tied back)
Reinforced backfill	Stack Sack
Timber crib	
Concrete crib	
Gabions	
Concrete gravity	
Concrete cantilever	
Welded wire	
Modular block	

Although proprietary designs exist for many of the standard walls, all standard walls may have nonstandard applications that require substantial modification of the design. The following sections present brief descriptions of the 14 wall types evaluated in this manual.

2B.2 Mechanically Stabilized Backfill

2B.2.1 General

Earth reinforcement is a very simple concept to use for both the design and construction of retaining walls. Ease of use and economy of construction and materials are its major selling points. Mechanically stabilized embankments require minimal foundation preparation and can sustain large amounts of differential settlement without serious damage.

2B.2.2 Reinforced Earth

Figure 2-2 shows an example of a standard wall constructed in accordance with a patented process developed by the Reinforced Earth Company. This process uses thin metal reinforcing strips to develop the required frictional resistance to support a wall's facing.

Reinforced earth can be purchased with a variety of standard concrete facings. Hence, some architectural use can be made of the various facing systems.

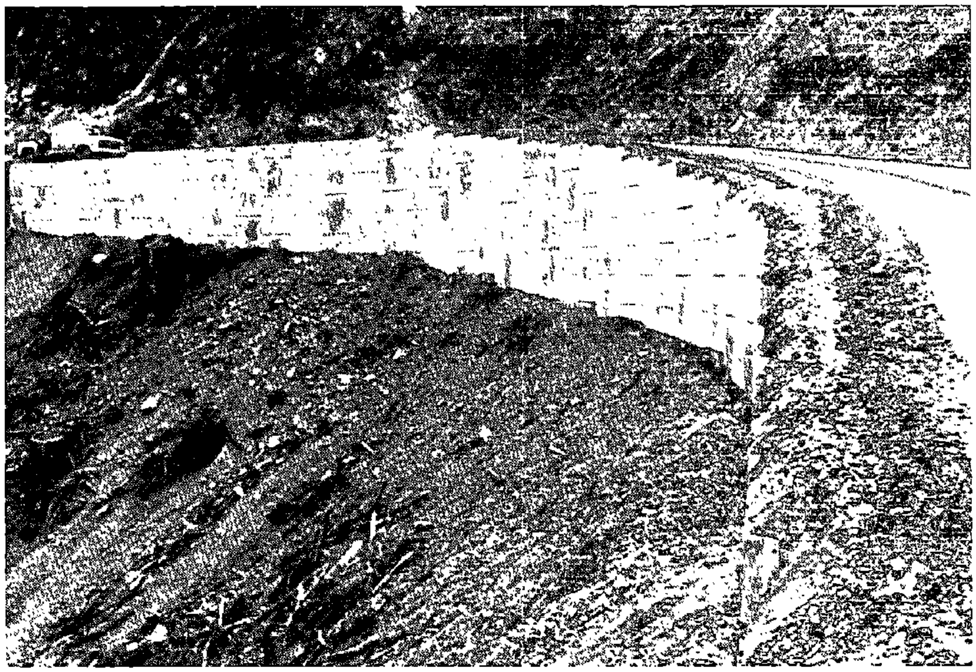


Figure 2-2.—Reinforced earth wall.

2B.2.3 Geosynthetic Reinforcement

Figure 2-3 shows a nonstandard reinforced backfill wall constructed of geosynthetic fabric. There are several advantages with geosynthetic reinforcement. It is a nonpatented process with low cost, ease of construction, and readily available construction materials.

The major shortcoming of geosynthetic reinforcement is the deterioration of the geosynthetic in sunlight, the susceptibility to vandalism, and the abrasion of geosynthetic by debris.

The geosynthetic facing shown in figure 2-3 has been coated with asphalt to protect it from deterioration in sunlight. Other walls have had shotcrete facing applied to protect the geosynthetic from sunlight and reduce vandalism. However, shotcrete raises the construction cost substantially and reduces the wall's ability to accept large amounts of settlement without damage. Timber also provides a good-looking, economical facing as shown in figure 2-4. Figure 2-5 shows a geotextile wall made with a used tire facing.

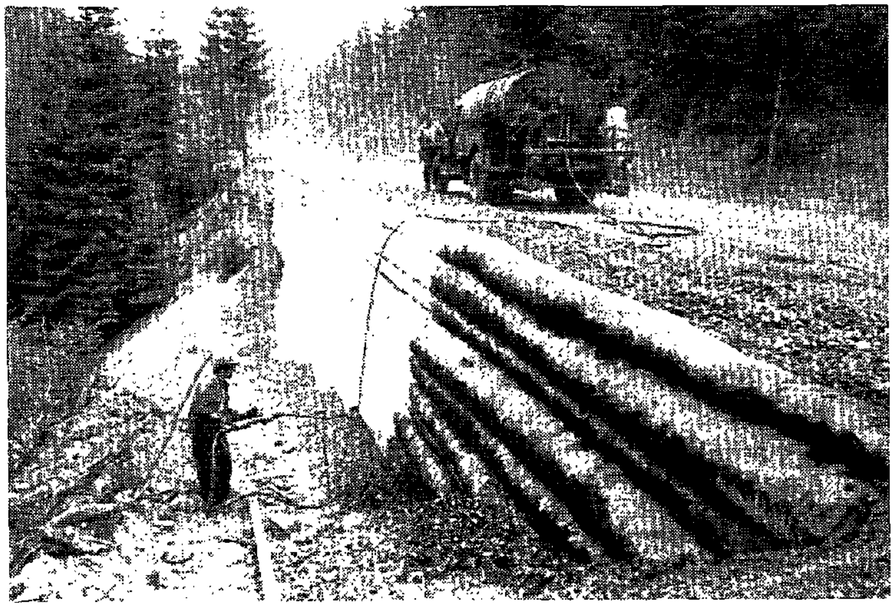


Figure 2-3.—Geosynthetic reinforced wall, Olympic National Forest.

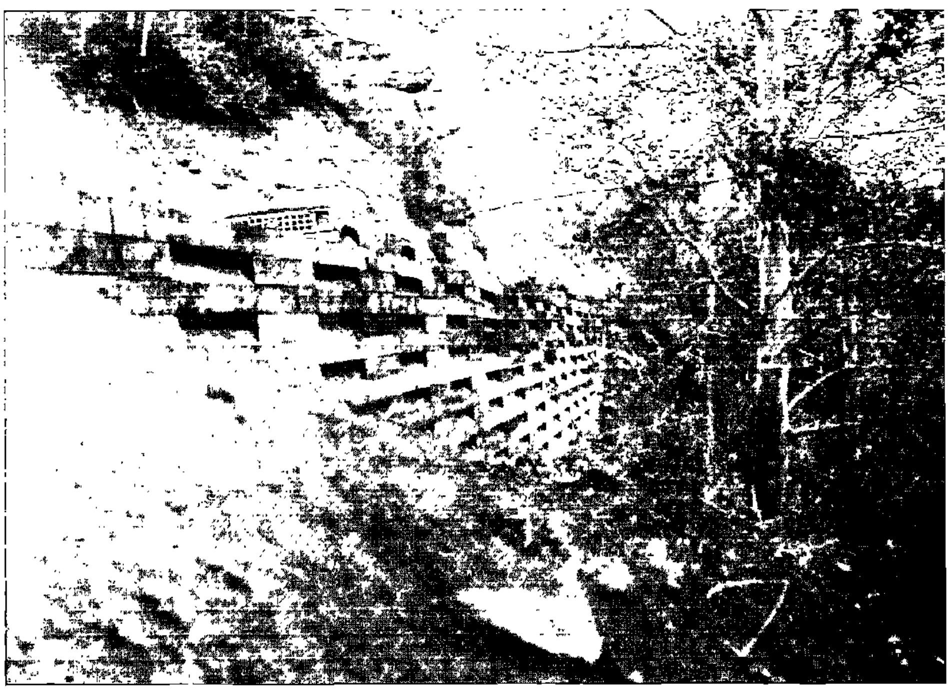


Figure 2-4.—Timber reinforced wall.

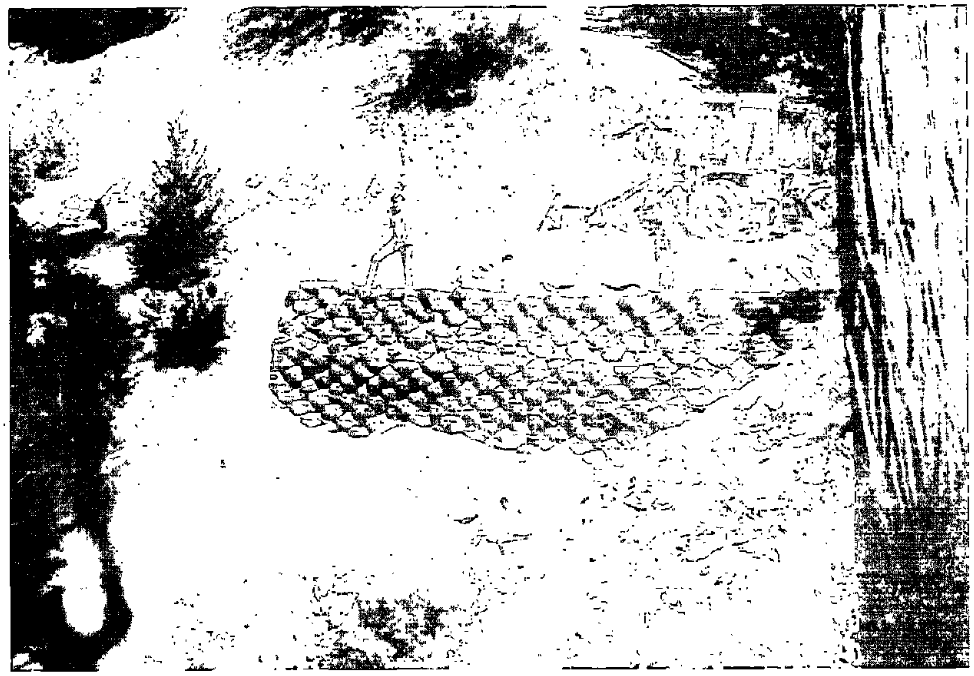


Figure 2-5.—Tire-Faced Geotextile Wall.

2B.2.4 Stack Sack Walls

Figure 2-6 shows a Stack Sack wall. The Stack Sack wall is a non-standard wall system, although it can be made with standard designs. Limited in use, the design is based upon a combination of tieback and backfill performance criteria. Thus, the designs should be carefully checked for each design situation.

Stack Sack walls will endure minor differential settlement. They typically require smaller amounts of excavation than true reinforced embankment walls, but they also require additional foundation preparation using an unreinforced working mat. Stack Sack walls have the same general ease of construction as reinforced embankment walls. Because of this low cost, Stack Sack walls are excellent for long, low walls or a series of walls in one general area. However, for isolated, short or high walls, the cost is high because construction can proceed at a rate of only about 2 feet of wall height per day.

Figure 2-6.—Stack Sack wall.

2B.2.5 Welded Wire Walls

Figure 2-7 shows a standard Hilfiker welded wire wall. This wall is made up of preformed wire components that are lightweight, transportable, and easy to handle. While the standard wire facing is mostly used, a variety of facings such as timber, concrete panels, shotcrete, and stone are available.

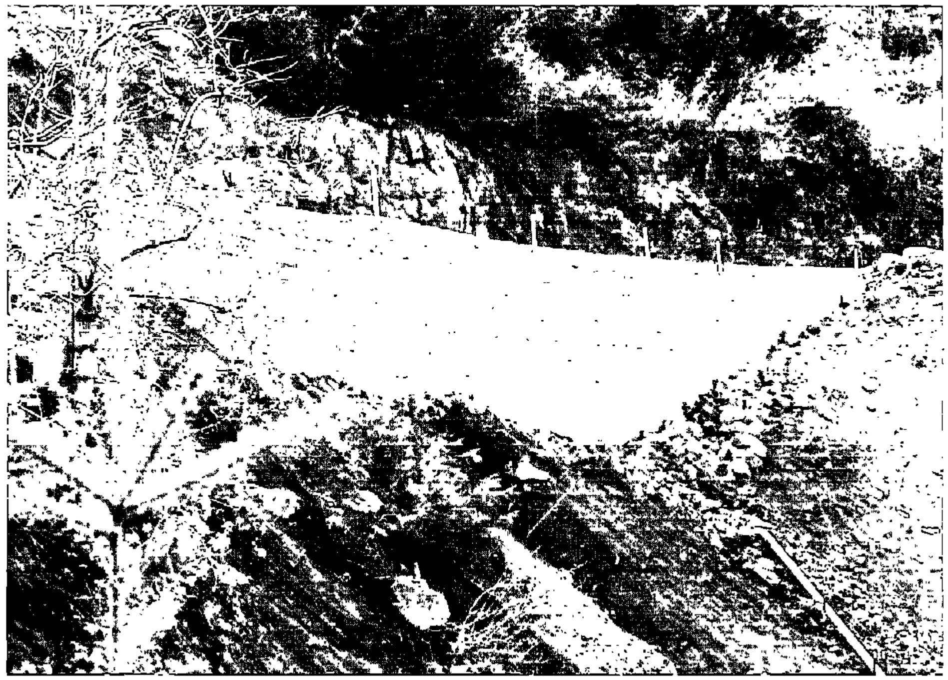


Figure 2-7.—Hilfiker welded wire wall.

2B.2.6 Chain Link Walls

Chain link fencing has been used as reinforcing material in reinforced backfill walls. Figure 2-8 shows an example of this type of wall.

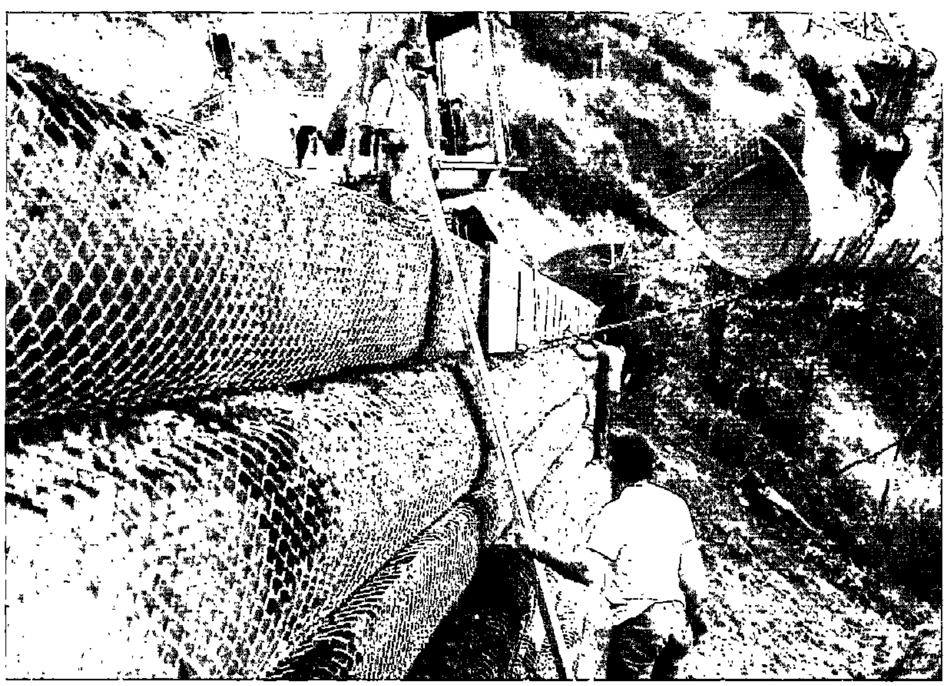


Figure 2-8.—Chain link wall.

2B.2.7 Modular Block Walls

Figure 2-9 is a modular block wall made by Keystone®. These walls are reinforced backfill walls where the reinforcements are Tensar® geogrids. A variety of architectural facings are available.

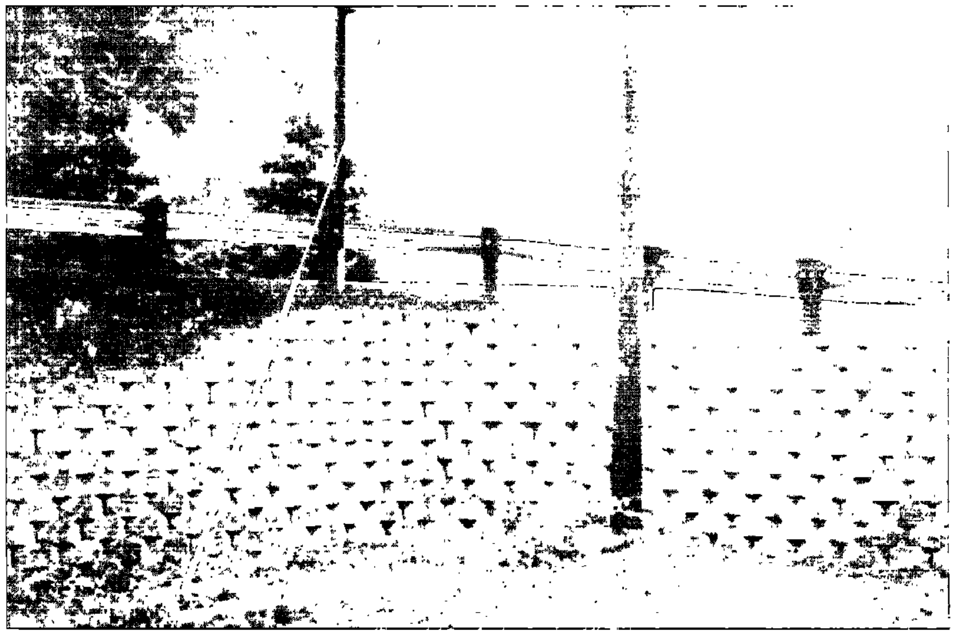


Figure 2-9.—Modular block wall.

2B.3 Gravity Walls

2B.3.1 Bin Walls

Figures 2-10 through 2-12 shows examples of circular, rectangular, and cross-tied bins. Steel bins are generally considered standard gravity structures, although cross-tied bins develop a portion of their stability from the tensile forces induced in the tie strap. Typically, steel bins are proprietary designs.

Some advantages of steel bins are their availability, ability to accept minor differential settlement, low maintenance, ease of assembly, appearance, and durability. Open-faced bin walls can also be constructed to make them blend in with their surroundings.

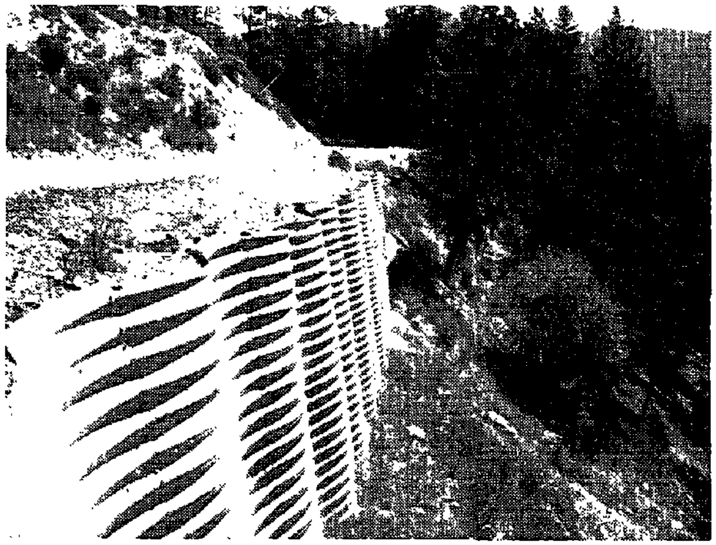


Figure 2-10.—Circular metal bin wall, Fall Creek Road, Okanogan National Forest.

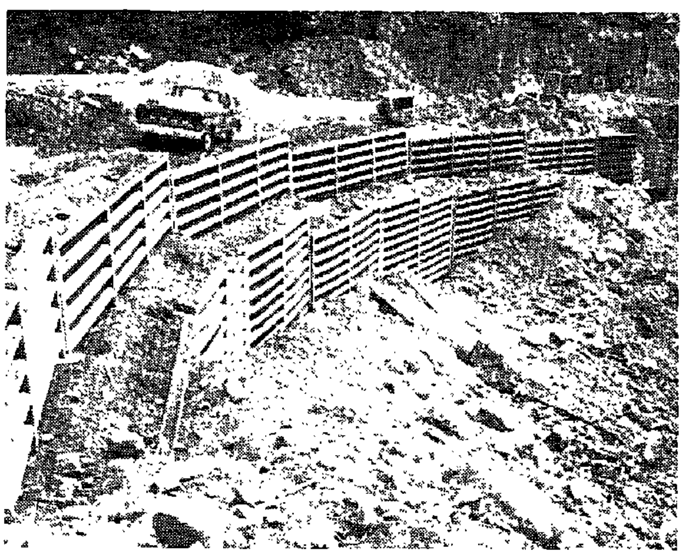


Figure 2-11.—Rectangular steel bin wall, Mary's Peak Road, Siuslaw National Forest.

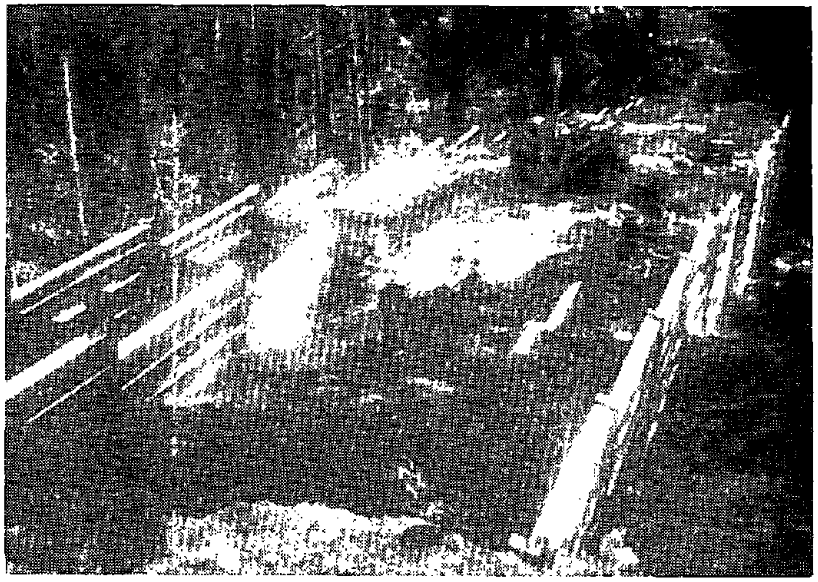


Figure 2-12.—Rectangular cross-tied bin walls, Stoney Mountain Road, Siuslaw National Forest.

Disadvantages of prefabricated steel bins are the difficulty and cost of modifying the design in the field to meet changing subsurface conditions, time lost waiting for factory-modified components, and limited space within the bins requiring the use of hand-compaction equipment. An important consideration when designing cross-tied bins is allowing sufficient space to maneuver a small tractor.

2B.3.2 Concrete and Timber Cribs

Figures 2-13 and 2-14 show examples of standard design concrete and timber crib walls. Similar to steel bins, crib walls are designed as gravity structures that rely on the stabilized fill within the cells for mass. Crib walls are generally built in a "log cabin style" with openings alternating with the cribbing; however, concrete cribs can also be built with a smooth face.

Timber cribbing does not possess the permanence that concrete and steel do, and, consequently, it requires more maintenance. Another disadvantage of open-faced cribs is the tendency of the backfill material to run out.

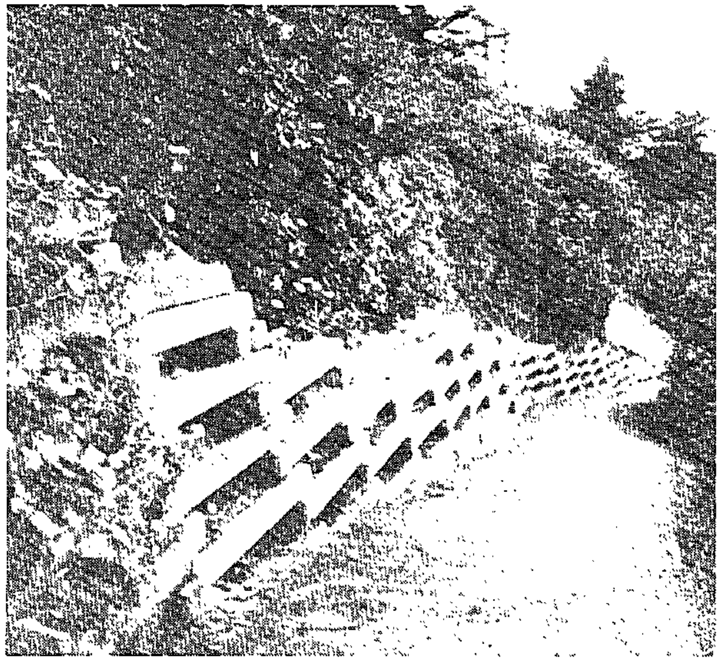


Figure 2-13.—Concrete crib wall, Cape Perpetua Visitor's Center, Siuslaw National Forest.

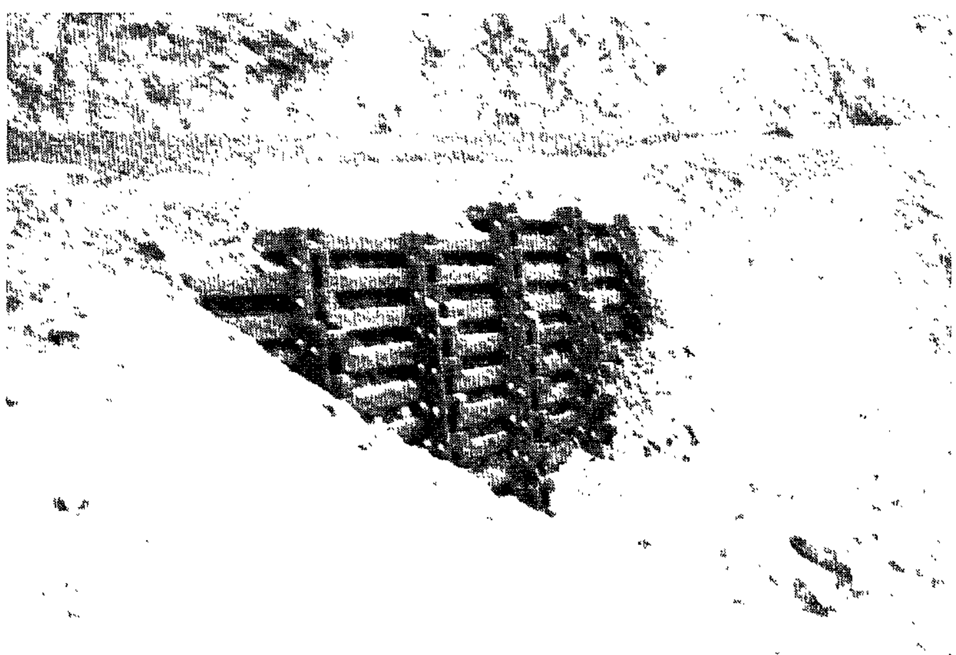


Figure 2-14.—Timber crib wall, White River Road, Mt. Hood National Forest.

2B.3.3 Gabions

Figure 2-15 shows an example of a gabion wall. Considered a standard type, gabion wall is a flexible, gravity structure that is constructed from preformed wire baskets filled with clean rock. Of all of the flexible gravity structures, gabion walls typically require the least amount of foundation preparation, and they can sustain the greatest amount of differential settlement without serious distress.

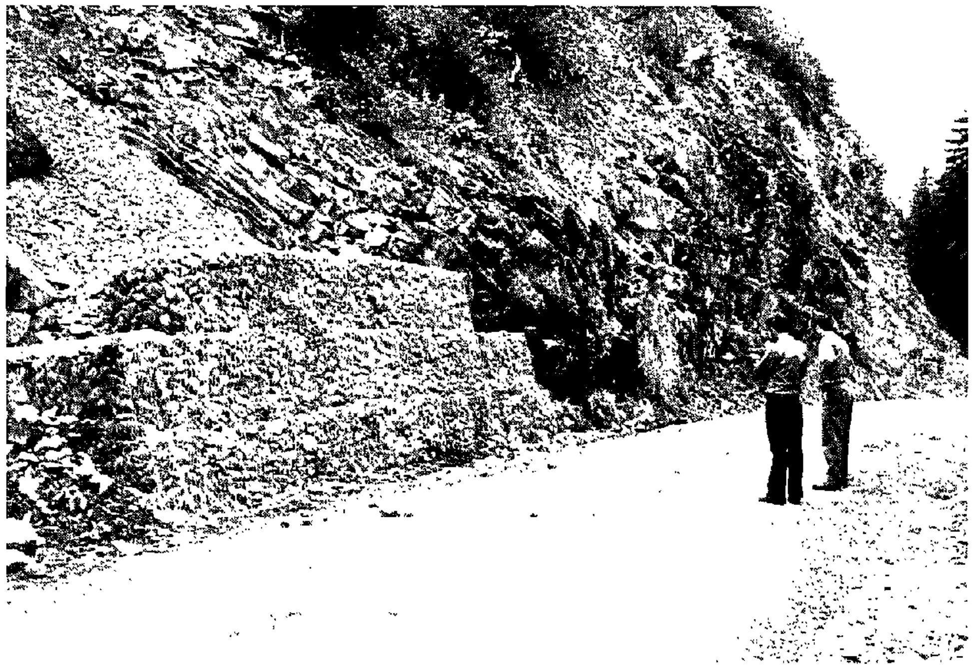


Figure 2-15.—Gabion wall, Quartzville Road Reconstruction, Willamette National Forest.

The advantages of gabions walls are similar to those of bins and cribs. Constructed of clean rock, gabion walls blend in very well with their surroundings. The preformed baskets are also light and easy to work with during construction.

One of the major shortcomings of gabion walls is that free-draining rock is required to fill the wire baskets. If quality backfill material is not available within a reasonable distance, gabion walls may be economically less desirable.

Experience shows that rupture of the wire baskets may be a problem for walls in the 25- to 30-foot-height range. This problem may be the result of localized over-stressing due to excessive differential settlement or to wire damage during construction caused by dropping rock fragments. Postconstruction damage resulting from stream load abrasion or vandalism may also be a problem. Therefore, gabion walls in the 20- to 30-foot range should be closely monitored during and after their construction.

2B.3.4 Concrete Structures

Figures 2-16 and 2-17 show examples of cantilever-reinforced concrete and unreinforced mass concrete retaining walls. Standard designs for concrete structures are available from many sources. Typically, concrete walls require better foundation conditions and more foundation preparation than the more flexible bins and cribs. With adequate foundation conditions, a concrete wall makes an excellent installation. Concrete walls can be formed, textured, and colored to make them more aesthetically appealing. If cast in place, they are more adaptable to changes in subsurface conditions than some prefabricated types. Due to the cost of materials, however, the construction of concrete walls may be very expensive in remote areas.

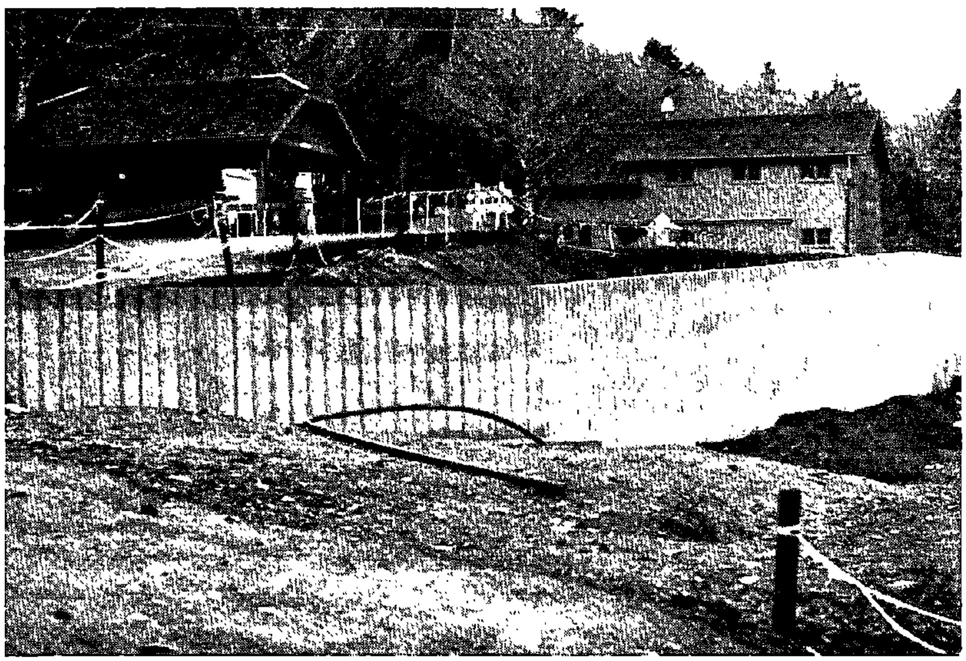


Figure 2-16.—Cantilever reinforced concrete wall, Hebo Ranger Station, Siuslaw National Forest.

Figure 2-17.—Unreinforced mass concrete wall, Interstate 5, Marquam Hill, Portland, Oregon.

2B.4 Cantilever Pile Walls

2B.4.1 General

Eliminating or greatly reducing the amount of excavation required, driven pile walls may be economical solutions to retaining wall design in excessively steep ground. Since essentially no excavation is required, there is virtually no disturbance to the downslope hillside, thus providing some environmental advantages. In addition, if the overburden is sufficiently thick for the piles to be driven, cantilever pile walls are easy to construct.

The major disadvantage of driven pile walls in remote areas is the need for special equipment, such as pile drivers and cranes. Depending on the job size and location, the costs for special equipment frequently constitute most of the project cost. Frequently in steep mountainous terrain, hard bedrock occurs at shallow depths and it may be difficult to drive piles to the necessary depth.

2B.4.2 Cantilever Sheet Piles

Figure 2-18 shows an example of a cantilever sheet pile wall. One of the many advantages of this type of wall is the speed of construction. Once the sheets are driven, the wall is complete except for backfill. The cost of the sheets is a major drawback in cantilever wall construction. The section required for a cantilever and, consequently, the costs can be greatly reduced by using soldier piles or tieback anchors. The latter will be discussed in a following section.

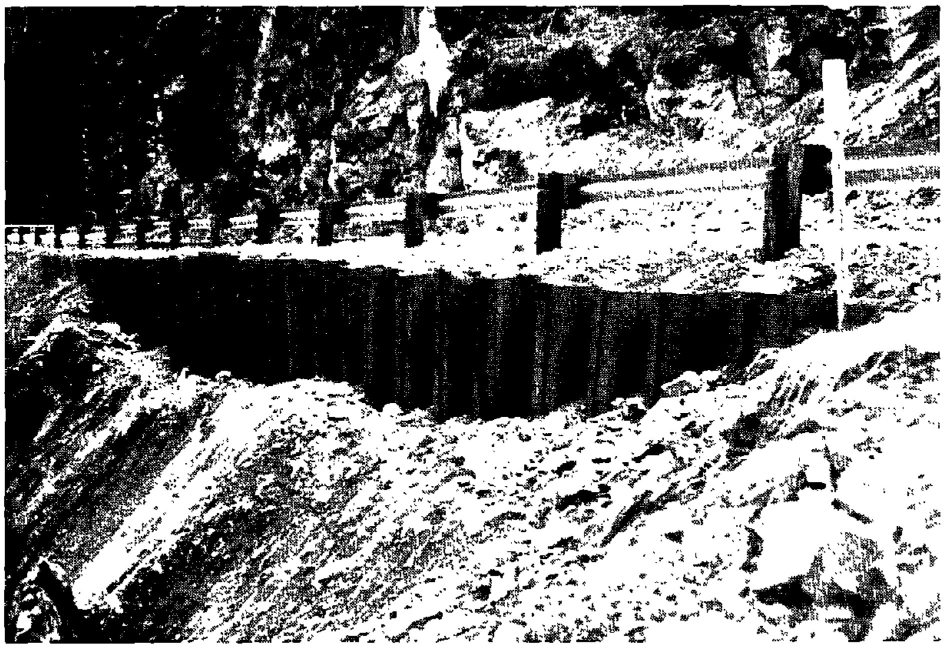


Figure 2-18.—Cantilever sheet pile wall.

2B.4.3 Cantilever H-Piles

The cantilever H-pile wall shown in figure 2-19 has timber lagging to support the backfill. Horizontal sheet piling and precast panels are other forms of lagging that have been used. Lagged H-pile walls typically represent an economical and easily constructed means of earth support and retention. An advantage of H-piles over sheets is that fewer of them are driven. Since there are fewer of them, they are generally heavier in section and can take more abuse during driving. Tip protectors can also be attached to the piles to reduce the probability of damage during heavy driving.

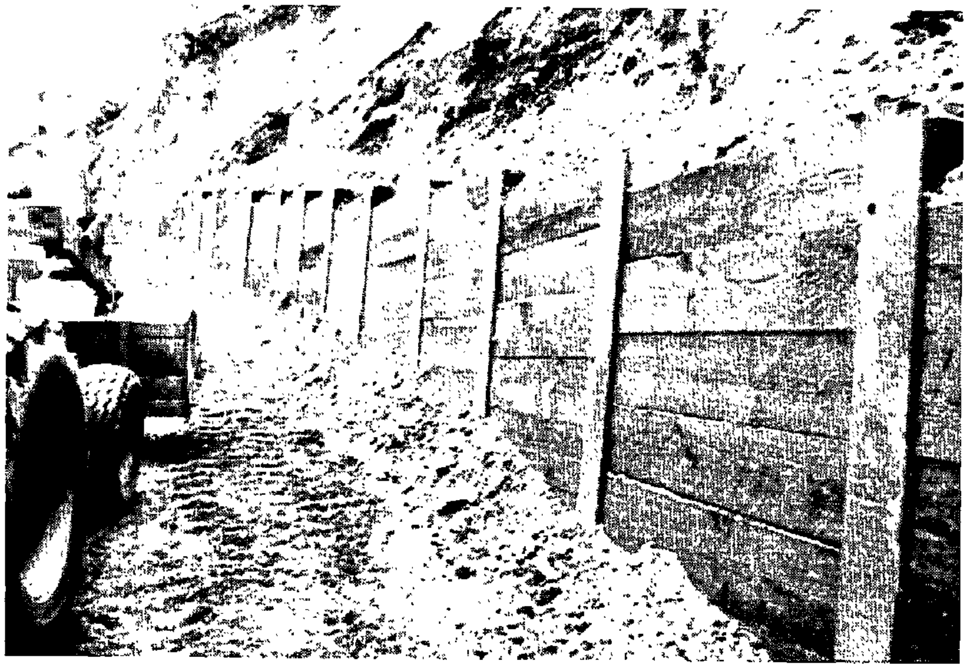


Figure 2-19.—Cantilever H-pile wall with timber lagging.

2B.5 Anchored Walls

2B.5.1 General

There are two basic types of anchoring systems. The traditional form of anchor is a protected steel tendon or bar that has been fixed in the soil or rock behind the assumed failure plane by a mechanical anchoring device or a column of grout. Another common form of anchor involves the use of a deadman and a tieback system. The deadman can be a gravity or partial gravity device, or some type of soil anchor that develops its capacity by mobilizing the passive resistance of the soil. Generally, deadman anchors are used for soil-related problems and are commonly placed in fill or undisturbed ground.

Recent developments in the use of anchored walls include the Stack Sack wall, which develops its resistance by a combination of tieback anchors and earth reinforcement. The use of tie back anchors generally allows the designer to reduce the required structural section of the facing.

2B.5.2 Anchored H-Piles

Figure 2-20 shows an example of an anchored H-pile wall with precast concrete panels for lagging. The H-piles have been cast into a reinforced concrete foundation because shallow bedrock prevented driving them to an adequate depth. The wall was anchored with rock bolts. For shallow bedrock conditions, the wall shown in figure 2-20 is quite economical. Similar walls may be constructed using driven H-piles provided that sufficient embedment can be achieved to fix the pile tips in the soil and/or weathered rock. Also, steel sheet pile or timber may be used as lagging.

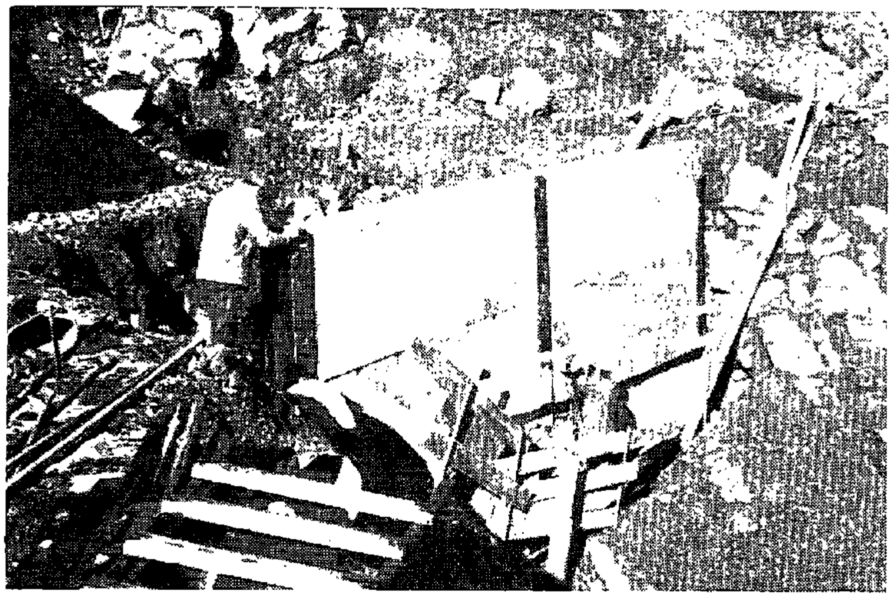


Figure 2-20.—Anchored H-pile wall—precast concrete lagging, Willamette National Forest.

2B.5.3 Anchored Sheet Piles

Figure 2-21 shows a driven sheet pile wall that has been tied back using a system of H-pile anchors and soldier piles. Sheet piles can also be tied back using a system of walers placed horizontally across the front of the wall.

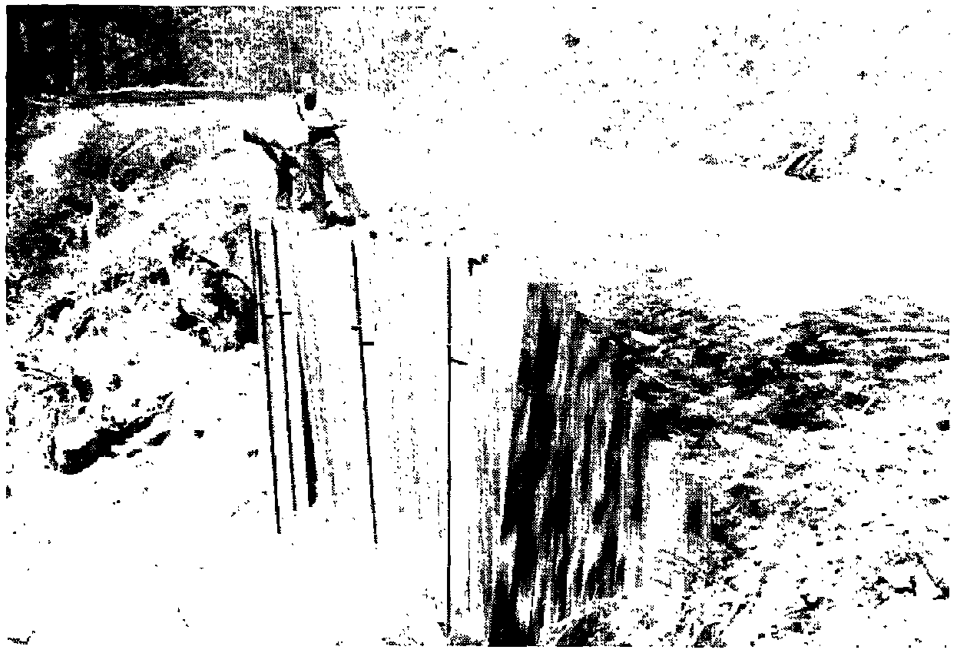


Figure 2-21.—Anchored sheet pile wall, Siuslaw National Forest.

2B.6 Standard Walls

The primary advantage of standard walls is that, typically, the components are prefabricated and readily available as shelf items. In addition, a minimum amount of preconstruction engineering is required except for excavation and backfill quantity estimates. An adequate geotechnical investigation, as outlined in appendix A, is required to evaluate the probable adequacy of foundation support and the resistance to overturning and sliding. Typically, standard walls are overdesigned with respect to internal stress and, therefore, do not require a rigorous internal structural analysis as long as the backfill soil strength properties equal or exceed those used by the manufacturer for design.

2B.7 Nonstandard Walls

Use of a nonstandard or custom wall design is required when geotechnical and/or topographic conditions necessitate. Geologic conditions that might require a nonstandard design would include weak foundations, shallow bedrock, landslide conditions, or deep soils on steep slopes. The most common topographic features requiring nonstandard designs are slopes generally in excess of 67 percent and narrow rock chutes.

The primary advantage of a custom design is the ability to design a wall to fit the slope, rather than designing the slope to fit the wall. Cost savings can be realized if this approach is prudently used.

2C Wall Dimensions

The physical dimensions of the wall are important from a standpoint of safety and economics. Walls less than about 20 feet in height and 2500 square feet in area may be designed according to standard design charts similar to those presented in appendix E of this guide. Although conservative, the use of the design charts is justified because increased preconstruction engineering and management costs could offset the increase in construction cost related to a conservative design. Design of retaining structures in excess of about 20 feet in height or about 2500 feet in area should be based upon comprehensive engineering analyses. The height restriction is related to both safety and economics, while the area restriction is based entirely on economics.

There is a direct relationship between total design thrust and construction costs. The design thrust depends on both the wall height and the friction angle of the soil. For a given wall height, therefore, variations in the assumed soil strength, ϕ, may change the value of the horizontal force by as much as 50 percent. Thus, standard designs based on wall height alone are likely to be substantially overdesigned for all but extreme conditions; the overdesign translates directly into increased construction cost.

Savings in construction costs for custom-designed walls less than 20 feet in height may be small and might be offset by preconstruction engineering costs. However, savings afforded by custom design of higher and longer walls accrue rapidly. Consequently, both economics and safety are major considerations in deciding whether to complete a limited or an indepth engineering evaluation of a given problem.

2D Geotechnical Considerations

The first and probably the most important contribution made to the wall design and selection process is the preliminary site review. Therefore, it is important that the site review be done thoroughly and comprehensively by a qualified engineering geologist or geotechnical engineer. Appendix A shows suggested outline for completing a site review.

The site review should provide documented information about slope stability, locations of surface cracking, probable depth of the failure surface (if there is landsliding), probable depth and slope of bedrock, foundation materials (soil and rock), estimated or allowable soil and/or rock bearing pressure, ground water conditions, local and regional faulting, seismic conditions, availability of construction materials, and probable anchor locations and capacities. Topographic data relating to the slope and physical configuration of the site should also be provided.

The geotechnical engineer or the engineering geologist is responsible for evaluating all of the preliminary information obtained in the light of his or her general knowledge of the fundamentals of soil mechanics, engineering geology, and his or her experience with the performance of various types of retaining structures and construction methods. Subsequently, the person should make recommendations about the most suitable types of retaining wall. These recommendations may be used for preliminary cost estimating or, depending upon the magnitude of the job, as a part of the final selection and design process.

If the problem under study warrants additional geotechnical study and investigation, it is the obligation of the geotechnical engineer or engineering geologist to make those needs known and to take action to undertake these studies.

2E Management Considerations

2E.1 Background

The current concern and requirements for protection of the environment have placed new constraints on construction. No longer are roads constructed for the sole purpose of economics. Therefore, the engineer and the engineering manager must not only consider the immediate utility of the road or structure, but also its economics, relative safety, visual qualities, long-term performance, future demands and required construction time. The following paragraphs present a more complete discussion of the management considerations introduced above.

Some types of operations require extra right-of-way or an easement. An example of this is installation of tiebacks or anchors when they extend outside the right-of-way.

2E.2 Future Demands

Two of the most important questions that management must answer are: What are the future demands on the road and structure? Will the structure have a short design life, or will it become a permanent structure open to the traveling public? Will the structure be designed for light traffic or will it have to support 10- to 12-foot-wide loads weighing up to 130,000 pounds?

Having established the intended long-term use, management should attempt to evaluate and satisfy the following objectives:

(1) Functional adequacy of the transportation system—limitations, speed, road surface, etc.

(2) Needs and demands for public safety—fencing, road width, paving, etc.

(3) Compatibility with all environmental objectives.

(4) Economy of construction, operation, and maintenance.

(5) Public acceptance of the whole project, such as tradeoffs that must be made to meet all objectives and requirements.

2E.3 Environmental Concerns

There are many environmental concerns that may influence the selection of one type of wall over another. Among the most commonly considered factors are visual impact, fisheries and wildlife impact, and long- and short-term effects on soil and water. The consideration of visual impact is of such importance that it is discussed separately in the following subsection.

Following visual impact, the next most commonly considered environmental concern is water quality and stream encroachment. The primary concern is the long-term maintenance of water quality. The wall and backfill must be designed to prevent stream siltation resulting from discharge of eroded soil through surface drains or the structure itself. Some "nonpoint source pollution," such as siltation during construction, is generally tolerated, provided that it does not harm downstream fisheries or water supplies.

Stream encroachment can present problems from the standpoint of fisheries, streambed and bank erosion, and politics. If a substantial increase in water velocity occurs due to encroachment, fish migration is generally negatively impacted. This becomes a problem if the stream flow is passed through a long culvert. In cases of severe encroachment, fish ladders should be supplied. Often encroachment further affects fisheries and water supplies by forcing the relocated stream to cut a new channel, thereby increasing the quantity of suspended particulate matter to unfavorable levels. Compliance with the U. S. Army Corps of Engineers or other stream encroachment limitations should be evaluated.

Siltation following erosion can be a serious problem if large cut slopes are required for wall construction. These areas should be treated either during construction or immediately thereafter to prevent deterioration of the visual resource and water quality. Slope protection can take the form of vegetation or, in the case of a streambank, riprap.

Construction of retaining structures may also have the potential for blocking traditional migration routes used by big game animals, particularly in excessively steep terrain. In addition, there is the chance that the construction might dislocate the nesting or breeding ground of endangered species. In these situations the advice and assistance of a fisheries and wildlife biologist should be sought.

2E.4 Visual Impact

People today are concerned about the visual impact that highways have on the landscape. Early in the design process, input should be sought from those in charge of the visuals. This may be a site and design review board for the agency or a landscape architect responsible for visual management on the project.

A review of the photos throughout this chapter shows the different looks that can be obtained. Also, many walls are available that have spaces for plantings to be incorporated into the wall face. With this concept it is possible to completely disguise a wall with vegetation where a structure "visual" is not wanted.

2E.5 Construction Timing

Timing is critical in the construction of any structure in a road system. It is critical in attempting to select a favorable construction season because it affects the total length of construction time required.

It is standard practice in most parts of the country to schedule all major construction projects for the dry summer months. This is particularly important if large volumes of earthwork are required, as is typically the case in retaining wall construction. This aspect of construction timing becomes even more important if quality backfill is not available in the immediate area. Long hauls and redesign or over-design to accommodate wet, low quality backfill greatly increase construction costs.

The concept of total construction time becomes a consideration from the following standpoints:

(1) If construction cannot be completed during one construction season, it may be necessary to remobilize the contractor the following spring at a substantial increase in total project cost.

(2) Interrupted road use becomes important if the structure under construction or repair is serving a major rural route or a residential area.

In situations similar to those already described, the structure selected should be simple in design and construction. Simplicity generally reduces the time required for construction, and minimizes the probability of unforeseen and time-consuming construction delays.

Most agencies today are required to comply with the National Environmental Policy Act (NEPA). This and other local requirements may place restrictions on when work can be performed. For example, work in anadromous streams may not be allowed during spawning seasons. Also, noise from blasting or pile driving may be prohibited during nesting season.

2E.6 Safety

Safety is everybody's business, and, according to the Occupational Health and Safety Administration (OSHA), construction safety is "not just a good idea, it's the law." Retaining wall construction in steep ground presents many hazards to a work force. Good common sense is probably the most effective weapon against construction accidents.

However, certain aspects of steep ground construction frequently defy even the clearest thought processes, e.g., rock falls, backslope failure, rolling and falling rock, high traffic density in confined areas during construction, and poor footing in wet or snowy weather.

The OSHA's *Construction Safety and Health Regulations*, title 29 CFR, part 1926, present the Federal standards for construction safety. Overall, this is an excellent set of regulations that should be closely followed by the contractor. Unfortunately, it is frequently the small contractor who has difficulty meeting and understanding the safety standards. It is, therefore, the obligation of engineering management to strictly enforce the requisite safety standards with the realization that some increase in construction costs will result.

One way to increase construction safety is to select a wall type that presents a minimum of risk to the work force. For example, a typical driven sheet pile wall requires minimal excavation and little or no above-ground work; however, a cast-in-place wall may require considerable excavation and a substantial amount of high form work.

The safety of the completed structure is also a serious consideration for management. The location of the structure in question is extremely important. For example, a tall, vertical wall near a high-use recreation area should receive more attention with respect to pedestrian safety than the structure on a rural road that will be used occasionally. The use of fencing and guardrail results in additional costs, but they are excellent means of reducing risk to pedestrian and vehicle traffic.

2E.7 Inventory and Condition Surveys

Condition surveys of retaining structures should be part of any highway maintenance program. However, because much of the wall is embedded in soil or rock, only the wall face is subject to rigorous examination and evaluation, unless the wall was instrumented with field measurement devices during construction. In the case of permanent retaining structures, it is recommended that a condition survey be completed biannually, unless the structure appears to be deteriorating at a rate that warrants closer scrutiny.

It is recommended that the inspecting engineer or technician has the as-built plans, including the as-built grading, any available geotechnical information, and previous condition reports. It is recommended that the inspector complete the standard inspection report presented in appendix B. The completed report should be circulated to key personnel members for evaluation.

2F Economic Considerations

2F.1 Background

Economics traditionally has been the controlling consideration in the design and selection of retaining structures for low volume roads. However, in light of safety and environmental concerns, the cheapest construction costs are no longer necessarily the best solution. Nonetheless, cost-effective engineering and management are still prime concerns of any agency.

The costs of construction of retaining structures on low volume roads are relatively high. Typically, this is due to the remoteness of the project, small project size, and steep, rugged terrain. Table 2-3, from Schwarzhoff, 1975, summarizes the cost implications of these three factors. The table shows that the only money-saving factors related to remote, rugged terrain construction are the possibility of reduced traffic control problems and the probability that construction tolerances may be less stringent. All other cost implications generally increase construction expenditures.

Table 2-3.—Low volume road cost factors (from Schwarzhoff, 1975).

Factors			
Remoteness	Small project	Rugged terrain	Cost Implications
x			Increases labor costs.
x			Higher materials transportation costs.
x	x		Equipment mobilization could be a high percentage of construction costs.
x			May decrease cost of features for traffic control during construction.
x	x	x	High engineering costs, particularly foundation investigations.
	x		Special equipment requirements increase cost.
	x		High costs when special tools are required to form or fabricate materials.
		x	High cost of construction safety features.
		x	High elevations often mean short construction seasons, resulting in need for "crash-type" construction which results in higher costs.
		x	Different wall types may influence other construction costs because of influence on overall project progress.
x		x	Highly precise construction tolerances may not be necessary, therefore allowing some decrease in assembly costs.

For purposes of discussion, the expenses related to a given project have been divided into preconstruction, initial construction, and maintenance. In addition, a fourth category, potential modification for future use, should also be considered for certain projects. The significance of the expenses associated with each of the above categories will be discussed in the following subsections and summarized in the accompanying outlines, tables, and figures.

2F.2 Preconstruction Costs

Preconstruction costs include all expenditures prior to the start of construction including topographic surveys, geotechnical investigations, engineering design of the structure, evaluation of the designs, and all managerial and overhead costs related to the problem. Typically, the preconstruction costs are directly related to the physical size of the proposed structure, the degree of complexity of the problem, and the political and economic sensitivity of the problem.

It is the responsibility of management, with the assistance of engineering and the other related disciplines, to analyze the economics of each site. These analyses should attempt to define the savings in construction costs and reductions in damage to the environment that can be achieved by accomplishing a detailed engineering analysis, or by using a nonstandard wall design instead of a standard design. Further, the cost of the engineering studies should balance with the savings from reducing the contractor's degree of uncertainty and risk.

2F.3 Initial Construction Costs

Initial construction costs include all expenditures for materials, labor, and overhead related to the actual construction of the retaining wall. Overhead costs refer to agency costs for administration of the construction contract. In addition, if the contractor is using agency-provided materials sources, for example, rock quarries or gravel pits, an accounting of these items should also be included.

Estimating initial construction costs, even when the project is reasonably well-defined, is difficult. A spread of 30 to 50 percent or more in bid prices is common on major construction projects.

Table 2-4 shows a comparison of costs for various walls. Costs of these walls, updated to 1992, appear reasonable, especially considering the remote locations.

Table 2-4.—Summary of Earth Retaining Structures.

	(Ft) Height	(Ft²) *Cost $	Comments
Reinforced fills	15–50	5–14	Less expensive than walls where they fit; slope typically 1:1; 1/2:1 slope with extra measures
Tire-faced walls	10	14	Significate face settlement; visually questionable
Timber-faced walls	1–8	16–22	Good wall considering cost, durability, and aesthetics; easy to construct
Geotextile-faced walls	1–20	15–29	Temporary structures; irregular and nondurable face unless covered or treated
Lightweight walls	28	16–25	Special geotextile walls suited for landslide terrain; moderate settlement with sawdust
Chain link fencing walls	22	23–29	Require a custom design; accommodate face settlement
Welded wire	6–30	23–34	Good construction support from manufacturer; standard designs available
Modular block	5–30	14–25	Masonry block facing; very aesthetic and durable; standard designs available; used for landscaping
Reinforced concrete cantilever level backfill 1-3/4:1 backfill	 10–20 10–20	 33–54 25–36	Standard designs available; long history of use
Reinforced earth	10–30	20–27	Good customer service
Metal binwall	10–30	35–45	Difficult to make height adjustments during construction
Concrete crib	10–20	25–32	Difficult to make height adjustments during construction
Geotextile or geogrid, shotcrete face	10–20	11–21	
Gabions	10–25	22–42	
Anchored H-pile	1–40	45–60	Requires a specialist design
Cantilever soldier pile with wood lagging	5–15	15–22	Requires a specialist design
Cantilever sheet pile	5–15	15–22	Requires a specialist design
Permanent tied back walls used for landslide retention		100–125	Requires a specialist design
Launched soil nails	5–30	8–15	Rapid stabilization of shallow slope failures with minimum site disturbance; new technology, may not be locally available

*1992 costs that typically include drainage, excavation, and backfill. Total wall cost can increase significantly depending on wall size, site difficulty, and other road repair work.

Note: 1 ft = 0.3048 m

The following paragraphs outline an approach to estimating initial construction costs that will provide a reasonable assessment of expenditures and method of comparing the relative costs of various wall systems for most projects.

The following assumptions have been made in order to simplify the remainder of this section:

(1) The average slope angle of the original ground surface will be 50 percent, 67 percent, or 100 percent.

(2) The retaining wall will be constructed below the road as a combination cut and fill.

(3) A spoil area and borrow source will be nearby and available for use at no cost to the contractor.

(4) The required excavation and fill for given types of wall systems, for example, gravity or anchored walls, has been assumed to be equal for a given wall height.

The approach used for estimating construction costs is to evaluate the project costs of (1) structural materials, (2) required excavation, (3) fabrication, (4) backfill, (5) overhead (contracting agency), and (6) contractor profit and risk.

Costs of the first four items have been summarized in tabular form on tables 2-5 through 2-9. Cost data for common excavation rates (soil and rippable rock) of 60 and 120 cubic yards per hour and fill placement rates of 60 to 200 cubic yards per hour are shown. These rates of excavation and fill correspond to those commonly achieved by experienced contractors. Typically, the lower rates correspond to steep-sided, confined sites that do not provide adequate access to operate equipment; the higher rates correspond to sites that have good access. Generally, the 60 and 200 cubic yard per hour rates for backfilling are associated with cellular crib or bin walls and reinforced backfill, respectively.

Inflation and cost increases are accounted for on all of the cost estimating figures by the inflation correction ratio, which is based upon data that is published in the *Engineering News Record.*

Soil and rock increase in volume (swell) when they are removed from their original locations because voids are created during the handling process. When using tables 2-5 through 2-9 for estimating excavation costs, it must be realized that no swell factors have been applied. Consequently, the estimated in-place excavation must be increased according to soil or rock type to estimate the actual truck volume using table 2-10. The use of the swell factor is demonstrated in chapter 4.

Table 2-5.—Costs of a 10-foot-high wall—$/ft² of Face.

Wall Type	Materials	Erection*	Backfill Excavation Rate, cy/hr 60.00	120.00	200.00
Gabions	2.51	5.18	2.94	1.69	1.28
Reinforced backfill	5.01	3.73	4.73	2.73	2.06
Concrete cantilever	6.26	15.84	2.94	1.69	1.28
Timber crib	14.20	5.18	2.94	1.69	1.28
Steel bins	16.28	5.18	2.94	1.69	1.28
Cantilever H-Pile, timber lagging	16.28	18.11	4.00	2.30	1.73
Cantilever sheet pile, soft or loose soil	28.64	18.11	4.00	2.30	1.73
Cantilever sheet pile, dense soil	18.79	18.11	4.00	2.30	1.73
Reinforced earth	10.50	3	4.73	2.73	2.06

* Note: Erection costs do not include filling cribs, bins, gabions, or earth reinforcement. These costs are included in backfill.

Table 2-6.—Costs of a 10-foot-high wall—$/ft² of face.

	Excavation Costs ($/ft² of face)					
	50% Slope		67% Slope		100% Slope	
Wall Type	60 cy/hr	120 cy/hr	60 cy/hr	120 cy/hr	60 cy/hr	120 cy/hr
Gravity walls (gabions, concrete cantilever, bins, and cribs)	1.18	0.68	1.40	0.81	1.63	0.94
Reinforced backfill	3.92	2.26	4.08	2.35	*	*
All piled walls	0.03	0.02	0.03	0.02	0.03	0.02

Table 2-7.—Costs of a 20-foot-high wall—$/ft² of face.

Wall Type	Materials	Erection	Backfill		
			60 cy/hr	120 cy/hr	200 cy/hr
Gabions	2.51	5.18	5.71	3.29	2.5
Reinforced backfill	11.69	3.73	6.45	3.73	2.81
Concrete cantilever	9.60	26.91	5.71	3.29	2.50
Timber crib	16.70	5.18	5.71	3.29	2.50
Steel bins	21.88	5.18	5.71	3.29	12.50
Cantilever H-pile, timber lagged	32.73	11.39	4.57	2.63	1.99
Cantilever sheetpile, soft or loose soil	35.24	11.39	4.57	2.63	1.99
Cantilever sheetpile, stiff or dense soil	16.10	11.39	4.57	2.63	1.99
Reinforced earth	10.00	3.00	3.43	1.99	1.51

Table 2-8.—Costs of a 20-foot-high wall—$/ft² of face.

	Excavation Costs, $/ft² of face					
	50% Slope		67% Slope		100% Slope	
Wall Type	60 cy/hr	120 cy/hr	60 cy/hr	120 cy/hr	60 cy/hr	120 cy/hr
Gravity walls (gabions, concrete cantilever, bins, and ribs)	1.47	0.87	1.96	1.13	3.43	1.99
Reinforced backfill	3.18	1.83	4.24	2.45	*	*
All piled walls	0.03	0.02	0.03	0.02	0.03	0.02

* Required excavation becomes excessive.

Table 2-9—Inflation factors.

Materials inflation correction ratio	Present ENR "Materials" Cost Index 1952.4*
Excavation backfill inflation correction ratio	Present ENR "Construction" Costs Index 5263.5*
Erection inflation correction	Present ENR "Common Labor" Cost Index 10,665.9*

* Engineering News Record Cost Indexes for October 18, 1993.

Table 2-10.—Estimated swell factors.

Material	Condition	Percent Swell
Clay or silt, with or without gravel	moist	40
Sand and/or gravel	wet or dry	10–15
Loam, topsoil	dry	15–35
Loam, topsoil	wet	25
Cemented soil		50
Rock (ripped or well blasted)	wet or dry	50–65

Similarly, when loose material is recompacted as fill, shrinkage (reduction in volume) occurs. Shrinkage factors have not been included in tables 2-5 through 2-9; consequently, the estimated cost of fill will have to be increased appropriately in accordance with the material being placed. Table 2-11 presents shrink factors for commonly used fill material. Use of shrink factors is also demonstrated in chapter 4.

Table 2-11.—Estimated shrink factors for embankment.

Material	Condition	Percent Shrink
Silt or clay, with or without gravel	moist	30–40
Sand and/or gravel	wet or dry	25–40
Loam, topsoil	moist	25–40
Cemented soil		30–40
Rock (ripped or well-blasted)	wet or dry	10–15

Overhead costs in retaining wall construction are associated primarily with construction control, supervision, and managerial costs of administering the contract. These costs are included in the outline presented in appendix C.

On a project that is well-defined, a contractor will typically operate with a projected profit roughly 15 to 20 percent, including all contingencies. However, as the contractor's risk exposure increases, so does the contingency portion of his total fee. On a high risk project, the contingency is frequently more than 30 percent. The simplest method of reducing the contractor's risk to a tolerable level is to provide a simple, well-defined design for construction. This requires a reasonable evaluation of the subsurface conditions.

The approach to cost estimating that has been shown in the tables and figures is based on the limited number of cost studies that are available, contractor experience, and the past experience of the writers. At present, this method appears to produce reasonable estimates of construction costs. In order to verify the future accuracy of the method, all users are urged to maintain records of construction estimates for comparison with actual construction costs.

2F.4 Maintenance

Maintenance on most retaining structures constructed on low volume roads should be small. However, this does not preclude completing condition surveys on a regular basis. Maintenance includes clearing of drainage facilities and repair of accidental damage or vandalism. No attempt will be made to quantify accidental or vandalism-related damage.

Drainage facilities must be maintained on a periodic basis and, in particular, during periods of heavy runoff. Assuming that no major changes occur near a retaining structure, for example, a change in land use has not occurred uphill, most serious drainage problems will be identified within a year of the wall's construction. These areas should be monitored very closely. The cost of maintaining drainage for retaining structures is generally included in the operating budget of the road system.

Excessive maintenance problems of low volume road retaining walls generally imply that the wall is poorly constructed or under designed, overloaded, or physically incompatible with its environment.

If maintenance or maintenance-related safety becomes a problem, it is recommended that the wall be evaluated for major upgrading or possible replacement.

2F.5 Projected Future Use

One final consideration in assessing construction costs for retaining structures is their future use. If, at the time of planning and initial construction, future use is anticipated, additional economic considerations must be assessed. Three questions need to be evaluated:

(1) Is the proposed structure, for instance, structurally or aesthetically adequate for the projected future use?

(2) If the proposed structure is not adequate, what costs would be associated with upgrading it? Would another structure be better suited or upgraded more economically?

(3) Would it be more economical to construct a temporary structure and replace it in the future with the required final design?

The solution to the future use problem is often very closely tied to economics and is most easily evaluated by comparing present needs and construction costs with those projected by management. These comparisons can be most effectively accomplished by using a selection process similar to the one described in this guide.

2G Evaluating Alternatives—Final Selection

2G.1 Approaches

Three approaches for selecting appropriate wall designs have been suggested in step 7 of the decision process. The approaches are:

(1) A consensus of engineers, related discipline specialists, and engineering management.

(2) A review by engineering management with the aid of choosing by advantages (CBA).

(3) An engineering staff, in case of an emergency.

Depending on the project size, sensitivity and timing, all the following suggestions are valid methods of selection. The following subsections contain brief discussions of each of the suggested methods for evaluating alternatives and the appropriate conditions for use. It has been assumed in these discussions that all parties are well-informed and understand the ramifications of the problem.

2G.1.1 Approach A

All major projects and any other projects that will have a significant impact on any of the interrelated disciplines should benefit from a consensus evaluation by a team whose members represent the various disciplines. Each of the participating team members should be prepared to present a summary statement about the project and its influence on their area of concern. Subsequently, the team should make a decision based upon the information provided. Quite possibly, the use of CBA by the team members might help to point out the advantages or disadvantages of a given method of solution. (A discussion of CBA is presented in the following section.)

2G.1.2 Approach B

2.G.1.2.1 Background

In evaluating alternatives, the purpose and circumstances of the decision must be defined: Why is the wall needed and what is it to accomplish? At this point, other alternatives that would provide for the intended uses without using a wall would have been considered and a wall is the selected course of action.

The process of alternative analysis included in this chapter is based on principles of sound decisionmaking from the CBA system of decisionmaking. This decisionmaking system was developed by Jim Suhr while working for the Forest Service in Region 4. It stresses using applicable criteria, appropriate viewpoints, relevant facts, and sound methods as the basis of sound decisionmaking. The system stresses weighing only the importance of advantages and not weighing factors or rating alternatives against the factors, which introduces error into decisionmaking. Weighing only advantages, and not advantages and disadvantages or pros and cons, eliminates double counting and lessens the chance of omission of exaggeration. Contact Jim Suhr, Decision Innovations, 801-782-6168 for more information.

2.G.1.2.2 Alternative Analysis

The alternative analysis will consist of:

(1) Identifying the needs and preferences of the stakeholders, and determining the decision factors relevant to the analysis.

(2) Identifying the applicable criteria (rules, regulations, guidelines) that relate to the analysis.

(3) Formulating a full range of alternatives.

(4) Assembling the attributes for each alternative that relate to each decision factor.

(5) Alternative analysis and tentative selection.

(6) Reconsideration of the decision.

(7) Implement the selected alternative, and evaluate the process and results.

Each of these subjects will be discussed below.

(1) Stakeholders and decision factors: The stakeholders are those that have an interest in, or are affected by, the decision.

Ideally, all of the stakeholders would be involved in making the decision. This usually is not possible or practical. The stakeholders need to be represented, however, so that their viewpoints are visible in the process. After identifying the purpose and circumstances of the decision, the stakeholders need to determine what factors are relevant to the analysis. Factors are containers of data (criteria, attributes, and advantages), and as such carry no "weight" or value relative to the decision themselves. They are categories in which data are organized. Some factors that may be relevant are: geotechnical, environmental impact, visual impact, future use, construction problems, maintenance, management, safety, and topographic. Project cost is considered later in the analysis.

(2) Criteria: Criteria need to be developed for each factor that is used in the analysis. This may come from rules, regulations, or guidelines, or may be decisions that guide further decisionmaking. The criteria are generally in the form of "must" or "want" statements. They are statements that allow the decisionmakers to judge the alternative's attributes within each factor.

(3) Formulating alternatives: Consider the walls that meet all of the "must" criteria in (2) above.

(4) Assembling attributes: Attributes for each alternative for each factor should be documented. The attributes are the characteristics of the alternatives that describe how that alternative contributes to each factor. These may be quantitative statements (facts and figures) or they may be qualitative statements describing how the alternative relates to each factor.

(5) Alternative analysis: The following steps of the CBA process are illustrated in the tabular format displayed in table 2-12.

(a) Alternatives are listed across the top of the matrix.
(b) Factors are listed down the left side.
(c) Attributes for each alternative for each factor are summarized in the top of each corresponding box.
(d) The least preferred attribute within each factor is determined and underlined.
(e) Using the least preferred attribute within each factor as a base, advantages are described for each of the other alternatives in the bottom of each corresponding box.
(f) The most important advantage within each factor is determined and circled.
(g) Comparing the most important advantages for each factor, determine which is the most important or "paramount" advantage for all factors.

(h) Assign the paramount advantage a convenient number of importance (100), and assign relative importance points to each of the other advantages. At this point be sure that there is a level of comfort with the points assigned.
(i) Total the importance points for each alternative.

Example Situation:

A road requiring the retaining wall is a single-lane logging access road. The structure is needed to support the roadway across a rock chute in a tight horizontal curve (radius = 70 feet). The wall is expected to be about 100 feet long and 25 feet high at the center. Foundation conditions are solid rock with highly weathered rock in the center 10 feet of the chute. The major construction problem is envisioned to be containment of the structural excavation. The primary road use will be for administrative and logging traffic, and no special safety problems are expected. The road traverses some rugged scenic terrain in an area of weathered granite. In the vicinity of the proposed retaining wall, the road is visible from a major visitor information center on an adjacent ridge. Thus, major management concerns are keeping land disturbances to a minimum and selecting a structure that blends well with the natural environment. Granular backfill material is available at a high cost. Local material with a low plasticity index is available with 3-inch maximum size.

The relevant factors and associated criteria for this situation follow:

(1) Land disturbance.
  - Less disturbance is desirable.
(2) Visual impacts.
  - Minimize visual disturbance.
  - Allowing for alternate treatments to blend with natural environment is desirable.
(3) Geotechnical considerations.
  - Minimize required geotechnical investigations.
  - "Common practice" methods are desirable.
(4) Design and construction flexibility.
  - It is desirable to be able to adjust to fit site during construction.
(5) Design life.
  - Minimize maintenance.
  - Minimize corrosion effects.

Table 2-12 shows the layout of the CBA decision table with the attributes described for each alternative. Table 2-13 is the same as table 2-12 with the advantages described, the most important advantage for each factor circled, and the "paramount" advantage identified. Table 2-14 is the completed CBA decision table for the project.

Table 2-12.—CBA decision table with ATTRIBUTES described for each alternative.

\ Factor\Alt		A Tied back concrete	B Anchored H.Pile	C Geosynthetic	D Welded wire	E Steel binwell
Land disturbance	Att	6 ft lateral excav at toe	4 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe
	Adv					
Visual impacts	Att	Can vary texture and color of face	Some variation, show vertical H pile	Wood, concrete, or painted face	Wire and rock face	Galvanized or painted face
	Adv					
Geotechnical	Att	Requires testing for rock bolts	Requires testing for rock bolts	Determine excav char of material, local common practice	Determine excav char of material, local common practice	Determine excav char of material, very common practice
Considerations	Adv					
Design and	Att	Very flexible	Inflexible w/ conc, flexible w/ wood	Very flexible, comes on roll	Somewat flesible, comes in sheets	inflexible, pre-manufactured
Construction Flexibility	Adv					
Design Life	Att	50+ years	50+ years	Projected at 50+ years	< 50 years using local material	50+ years
	Adv					
	Total Cost	$100,000	$80,000	$50,000	$60,000	$70,000

Table 2-13.—CBA decision table with attributes and advantages for each alternative. The most important advantage for each factor is circled and the "PARAMOUNT" advantage is identified and assigned 100 points. Alternatives with no advantage are assigned 0 points.

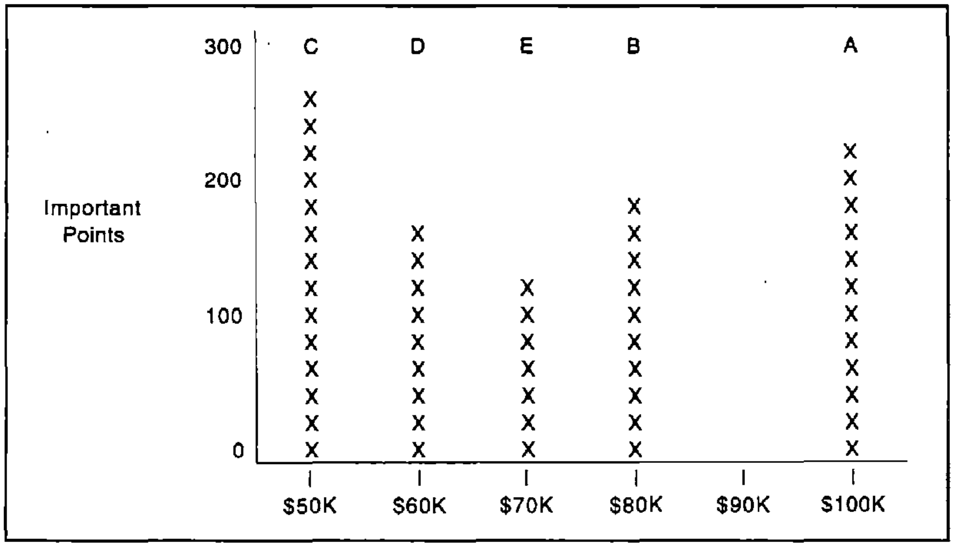
\ Factor\Alt		A Tied back concrete	B Anchored H.Pile	C Geosynthetic	D Welded wire	E Steel binwell
Land disturbance	Att	6 ft lateral excav at toe	4 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe
	Adv	9 ft less disturbance	11 ft less disturbance	0	0	0
Visual impacts	Att	Can vary texture and color of face	Some variation, show vertical H pile	Wood, concrete, or painted face	Wire and rock face	Galvanized or painted face
	Adv	Flexible in forming	Flexible color & texture of panel	Flexible color & texture 100	Color of rock backfill	
Geotechnical Considerations	Att	Requires testing for rock bolts	Requires testing for rock bolts	Determine excav char of material, local common practice	Determine excav char of material, local common practice	Determine excav char of material, very common practice
	Adv	0	0	Less testing, flexible	Less testing, flexible	Less testing but rigid
Design Construction Flexibility	Att	Very flexible	Inflexible w/ conc, flexible w/ wood	Very flexible, comes on roll	Somewat flesible, comes in sheets	inflexible, pre-manufactured
	Adv		0	Easily re-designed	Must order new sheets for cnge	Must re- order for diff sizes
Design Life	Att	50+ years	50+ years	Projected at 50+ years	< 50 years using local material	50+ years
	Adv	Longer design life	Longer design life	Projected longer design life	0	Longer design life
	Total Cost	$100,000	$80,000	$50,000	$60,000	$70,000

Table 2-14.—The completed CBA decision table.

\ Factor\Alt		A Tied back concrete	B Anchored H.Pile	C Geosynthetic	D Welded wire	E Steel binwell
Land disturbance	Att	6 ft lateral excav at toe	4 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe	15 ft lateral excav at toe
	Adv	9 ft less disturbance 40	11 ft less disturbance 50	0	0	0
Visual impacts	Att	Can vary texture and color of face	Some variation, show vertical H pile	Wood, concrete, or painted face	Wire and rock face	Galvanized or painted face
	Adv	Flexible in forming 90	Flexible color & texture of panel 80	Flexible color & texture 100	Color of rock backfill 50	0
Geotechnical Considerations	Att	Requires testing for rock bolts	Requires testing for rock bolts	Determine excav char of material, local common practice	Determine excav char of material, local common practice	Determine excav char of material, very common practice
	Adv	0	0	Less testing, flexible 60	Less testing, flexible 60	Less testing but rigid 50
Design Construction Flexibility	Att	Very flexible	Inflexible w/ conc, flexible w/ wood	Very flexible, comes on roll	Somewat flesible, comes in sheets	inflexible, pre-manufactured
	Adv	40	0	Easily re-designed 70	Must order new sheets for cnge 60	Must re- order for diff sizes 20
Design Life	Att	50+ years	50+ years	Projected at 50+ years	< 50 years using local material	50+ years
	Adv	Longer design life 50	Longer design life 50	Projected longer design life 40	0	Longer design life 50
	Total	220	180	270	170	120
	Cost	$100,000	$80,000	$50,000	$60,000	$70,000

Since each of the alternatives has different project costs, another step is needed to make the selection of the best alternative. This is done by plotting importance points against project costs as shown in table 2-15.

Table 2-15.—CBA decisionmaking.



Alternative C is the obvious choice because it has the greatest total importance for the least cost. This is not always the case. For example, if alternative C was not feasible and a choice had to be made from the other four alternatives, the decision would be much more complicated. It is obvious that alternative E would not be selected because it offers less importance for a higher cost than alternative D. The question that must be answered (if we are not considering alternative C) is: Is the gain in importance of 10 points (180–170) between alternatives B and D worth the additional investment of $20,000 ($80K–$60K) between alternatives B and D? Or, is the gain in importance of 50 points (220–170) between alternatives A and D worth the investment of $40,000 ($100K–$60K) between alternatives A and D? Alternative A does have a higher incremental importance/cost than does alternative B. Depending on the circumstances and the availability of funds, B or A may be possible selected alternatives. In making this type of decision, the team must decide if the additional importance gained is worth the additional investment.

(6) Reconsideration: It is important that the stakeholders represented in the decisionmaking process are comfortable with the recommended alternative. They should review the tentative decision to make sure their viewpoints have been considered. Since this is a very visible system, anyone reviewing the process should be able to see what the thinking

of the analysis team was, and if they are not in agreement they should be able to show visibly where they would make changes in the decision information.

(7) Implementation and evaluation: Follow up on the project during the implementation to see if the assumptions that were made were accurate and evaluate the soundness of the decision. This will provide feedback for future designs and increase the reliability of design and cost data.

2G.1.3 Approach C

The final method of wall selection is used in emergencies when it is expedient and relates to politics, economics, or safety. Under these conditions, it is recommended that the engineering staff be allowed to evaluate the problems from a primarily technical standpoint and to make a recommendation for construction directly to management for approval. This should not be used as an excuse to bypass the legal requirements of NEPA but should only be used in an emergency.

Chapter 3
Design Considerations

3A Introduction

The first step in solving any retaining wall problem is defining the problem. This entails: (1) identifying the soil types; (2) describing the local geology; (3) defining the topography and critical cross-sections at the site; (4) estimating the loading conditions, that is, backfill and surcharge; (5) studying the overall stability of the area; and (6) establishing soil and ground water parameters for preliminary and noncritical situations. Completion of the preliminary site investigation form, outlined in appendix A, should provide much of the required information.

Having collected the information above, the procedure for the design of retaining walls consists of two steps: selection of tentative dimensions of the structure, and analysis of the ability of the design structure to resist the internal and external forces. The process is repeated until a stable, economically feasible structure has been designed. The external analyses should include evaluations of sliding, overturning, bearing capacity, and overall slope stability. The internal analyses should consider shear, bending moment, tensile forces, and differential settlement (depending on the wall type). Since all forces acting on the structure are transmitted by soil, geotechnical information should be incorporated in all retaining wall designs. For each wall, the designer must decide on the amount and type of geotechnical information to include in the analysis and the level of accuracy required. The cost of the investigations can be justified with projected savings in construction costs or a higher degree of confidence in the computed factor of safety.

3B Geotechnical Investigation

3B.1 General

A well-planned geotechnical investigation is essential to the economical and safe design of a retaining structure. The problem is evaluated and the boundary conditions are established in the field by the geotechnical investigations. The geotechnical investigation should be accomplished under the direction of an experienced engineering geologist or geotechnical engineer with a good knowledge of soil mechanics.

Generally, the main areas of concern are the lateral earth pressure, ground water, foundation stability, stability of retained soil and appurtenant slopes. In all cases, both the undrained (end of construction) and drained (long-term) performance should be evaluated.

A geotechnical investigation typically consists of three sections: field investigations, laboratory studies, and engineering analyses. The latter, having been thoroughly discussed in other portions of the guide, will not be considered in this section.

3B.2 Field Investigations

The exploration phase of retaining wall design must consider a range of design elements and retaining systems prior to and during the investigation phase. This is necessary because as information is gathered and the subsurface model developed, conditions that are conducive to the most efficient and cost-effective design will become apparent. If the design is constrained due to right-of-way, for instance, exploration methods can be chosen to meet the design criteria for that particular type of wall.

The geotechnical specialist must realize that some uncertainties can be handled during design and construction. The feasible designs for a given project will be developed as the geologic model is developed during the investigation. One of the most important consideration to keep in mind is that design alternatives must be kept in perspective as the geologic model is developed.

Comprehensive field investigations should include surface and subsurface evaluations of the site. The investigations should be initiated with a preliminary site visit and the completion of the form in appendix A. The remainder of the field investigations should be planned on the basis of the preliminary site visit. Generally, the field investigation will include some form of subsurface exploration, such as drilling, test pits and trenches, and refinement of the preliminary geologic surface mapping or geophysical surveys.

3B.3 Essential Information

The following list of exploration essentials and techniques is offered for investigation decisions. The reader should refer to section 3 in the *Slope Stability Reference Guide for National Forests in the United States* for an expanded discussion.

(1) Scope of project
Locations, size, risk, and budget

(2) Project objectives
Will the proposed wall be designed to retain:
Fill slopes
Cut slopes
Landslide forces
Presently failed cut or fill slope conditions
Wall location
Road width

(3) Project constraints
    Right-of-way (for construction or tiebacks)
    Environmental impact
    Visual impact
    If the design requires a retained cut slope, will tieback or staged excavation be necessary?

(4) Wall system considerations
    Gravity structures
    Reinforced soil walls
    Driven sheet piles
    Driven H-piles
    Socketed H-piles
    Soil nails
    Tied back structures

3B.4 Surface Exploration

Obtain controlled survey plan and section profiles through wall area.

Use the field-developed cross-section method (see appendix 3.5 in the *Slope Stability Reference Guide for National Forests in the United States*) to develop geologically interpreted sections and tie to controlled survey.

Map and field classify soil and rock units according to the Unified Soil and Rock Classification Systems (see appendices 3.2 and 3.4 in the *Slope Stability Reference Guide for National Forests in the United States*).

Determine mass attitude of rock units from outcrop mapping and three-point projection.

3B.5 Subsurface Exploration

3B.5.1 Hand Tools Only

Use drive probe (see appendix A), hand auger, and shovel to obtain subsurface profile information from soil, rock, and ground water regime. Information is needed upslope, downslope, and along the grade line of the proposed structure.

Develop initial approximations of vertical and horizontal distribution of soil and rock units and ground water model. Drive probe pipe may be left in place as open standpipe piezometer.

3B.5.2 Geophysics

Seismic refraction surveys along surveyed sections and grade line location will help to confirm hand tool exploration results.

3B.5.3 Backhoe Trenching

This is another method for determining subsurface information that is about half the cost of drilling. The other advantage of this method is the use of excavation characteristics in determining cost estimation.

Use an Ely volumeter to obtain soil unit weight values.

Use a vane shear to obtain undrained shear strength values in cohesive soils in test pit sideslopes or large excavated blocks.

Obtain lab samples to test index properties (Atterberg limits, moisture content, unit weight, and gradation). Undisturbed samples for shear strength and consolidation testing may be retrieved by pushing Shelby tubes with the backhoe bucket.

3B.5.4 Drilling

If drilling is necessary, the following techniques should be considered:

(1) Continuous standard penetration tests using hollow stem augers through the soil profile to a depth equal to at least two times the wall height, or until refusal is encountered. If refusal is encountered, then core-drill a minimum of 10 feet beyond.

(2) Obtain undisturbed soil samples in representative layers.

(3) Use torvane and pocket penetrometer to obtain undrained shear strength values in cohesive soil in the end of the Shelby tubes. Transport the tubes secured in a vertical position.

(4) Consideration may be given to using the Dutch cone penetrometer, bore hole shear, or pressure meter for in situ tests.

(5) Install, as a minimum, open standpipe piezometers in all drill holes.

(6) When operating drilling machines or excavators are near streams, require the operator to have spill containment and clean up equipment available onsite.

3B.6 Laboratory Studies

Like the field investigation, the laboratory studies and the data obtained should be planned and conducted under the direction of an experienced geotechnical engineer. Experience and familiarity with soil behavior will permit the designer to optimize the laboratory budget through the judicious use of classification and index property tests. The following paragraphs describe the minimum index property soil tests commonly used in the geotechnical aspects of retaining wall design. Index properties are used to group soils into major classifications with respect to soil type and probable behavior. Prudent use of index property tests may greatly reduce the number of strength and consolidations tests required to design the wall.

3B.6.1 ASTM D2216 Natural Moisture Content

This test should be performed on all samples obtained from the subsurface investigation. The correlation between Atterberg limits and optimum moisture content may be used to deduce probable soil behavior with respect to relative stability and workability. A relationship also exists between natural water content and compression index for normally consolidated soils.

3B.6.2 ASTM D423 Atterberg Limits

This test should be performed on selected samples to aid in classification and for correlation with their test data. For example, the plastic limit frequently approximates the optimum moisture content as determined by ASTM D698 (AASHTO T-99). Correlations can also be developed between residual friction angle and plasticity index, and liquid limit and compression index. Generally, such relationships should be developed on a local basis to be most valuable.

3B.6.3 ASTM D1556/D2937 Unit Weight

These tests should be performed to aid in classification and to provide data for earth pressure and stability calculations. Although in-place density tests may be performed, unit weight determinations are generally made on Shelby tube samples. Typically, in-place density testing is used for fill control.

3B.6.4 ASTM D422 Grain Size Analyses

These tests are performed for soil classification and filter design.

3C Soil Properties

The influence of pore water pressure will be discussed in the drainage subsection. The following tables and charts present typical values of the soil parameters presented above. These values are provided for use in preliminary designs and certain phases of semiempirical designs, as described in Section 3D.2, "Semiempirical Methods." Note that for theoretical designs as shown in Section 3D.3, "Theoretical Methods," all soil properties should be verified by field and/or laboratory testing.

Table 3-1 shows empirical relationships between standard penetration resistance, relative density, friction angle, and unit weight for granular soils. Make sure that the values presented in tables 3-1 through 3-3 and figure 3-1 are average values. The actual in situ values may vary from those presented by ±20 percent.

Table 3-2 presents empirical relationships between standard penetration resistance, unconfined compressive strength, and unit weight.

The identification characteristics shown at the bottom of table 3-2 are useful guidelines when classifying soils.

Figure 3-1 shows approximate relationships between dry unit weight and angle of internal friction for cohesionless backfill. The soil types shown on this figure (e.g., GW and SP) refer to the Unified Soil Classification System, a complete description of which appears in all basic texts on soil mechanics.

Table 3-1.—Granular soil (after Teng, 1962).

Compactness		Very Loose		Loose		Medium		Dense		Very Dense	
Relative density, D_d	0		15%		35%		65%		85%		100%
Standard penetration resistance, N = no. of blows per foot.	0		4		10		30		50		
ϕ (degrees)*			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	

*highly dependent on gradation and particle angularity

Table 3-2.—Cohesive soil (after Teng, 1962).

Consistency		Very Soft		Soft		Medium		Stiff		Very Stiff		Hard
q_u = unconfined compression strength, tons per square foot.	0		0.25		0.50		1.00		2.00		4.00	
Standard penetration resistance, N = no. of blows per foot.	0		2		4		8		16		32	
Unit weight, pcf (saturated)			100-120			110-130			120-140			130+
Identification charateristics		Exudes from between fingers when squeezed in hand		Molded by light finger pressure		Molded by strong finger pressure		Idented by thumb		Indented by thumb nail		Difficult to indent by thumb nail

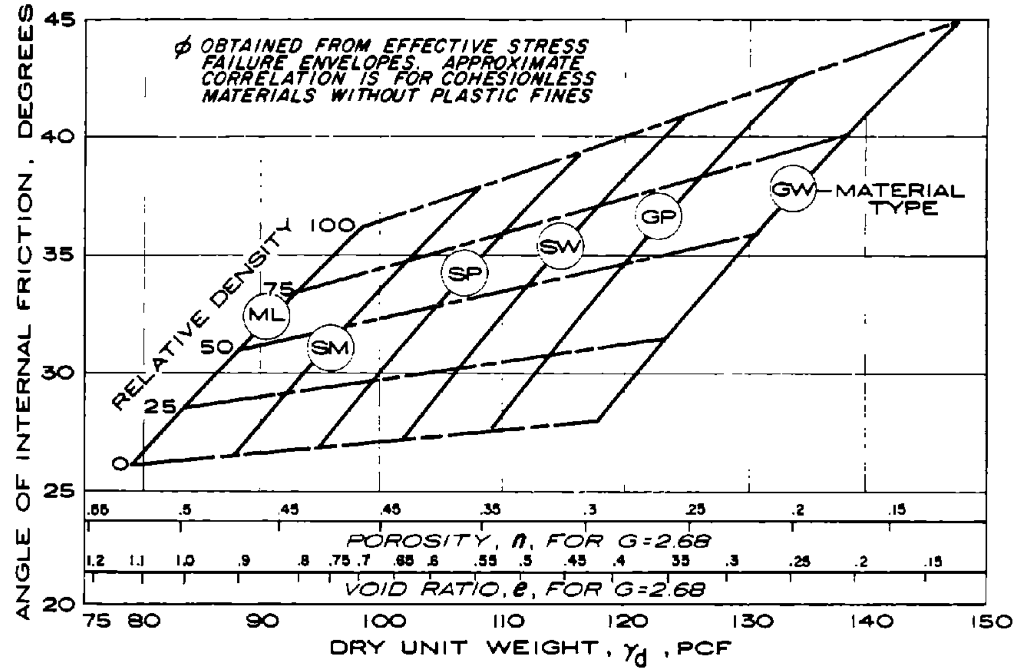


Figure 3-1.—Angle of internal friction versus dry unit weight for granular soils (after NAVFAC DM-7, 1971).

Figures 3-1A, B, and C are additional charts that provide correlations between soil classification and soil parameters. These figures show a method of determining soil strength values for design. In many cases these will provide values that can be used for final design without shear strength testing. Where risks are high or significant cost savings can be made, actual soil strength tests may need to be performed.

Table 3-3 shows a summary of soil properties for potential backfill materials that have been compacted to the maximum dry density, as determined by AASHTO T-99. For low to moderately plastic clays, limited information implies that, due to creep effects, the long-term values of $\bar{c}$ decrease to 0 and ϕ decreases to 20 to 30 degrees. Higher plasticity clays probably exhibit even more reduction of frictional characteristics in the long term. Hence, although cohesion has a tendency to lower the active earth pressure and raise the passive earth pressure, it is frequently omitted for design. Likewise, for clay of high plasticity, the peak friction angle is seldom used for long-term analyses.

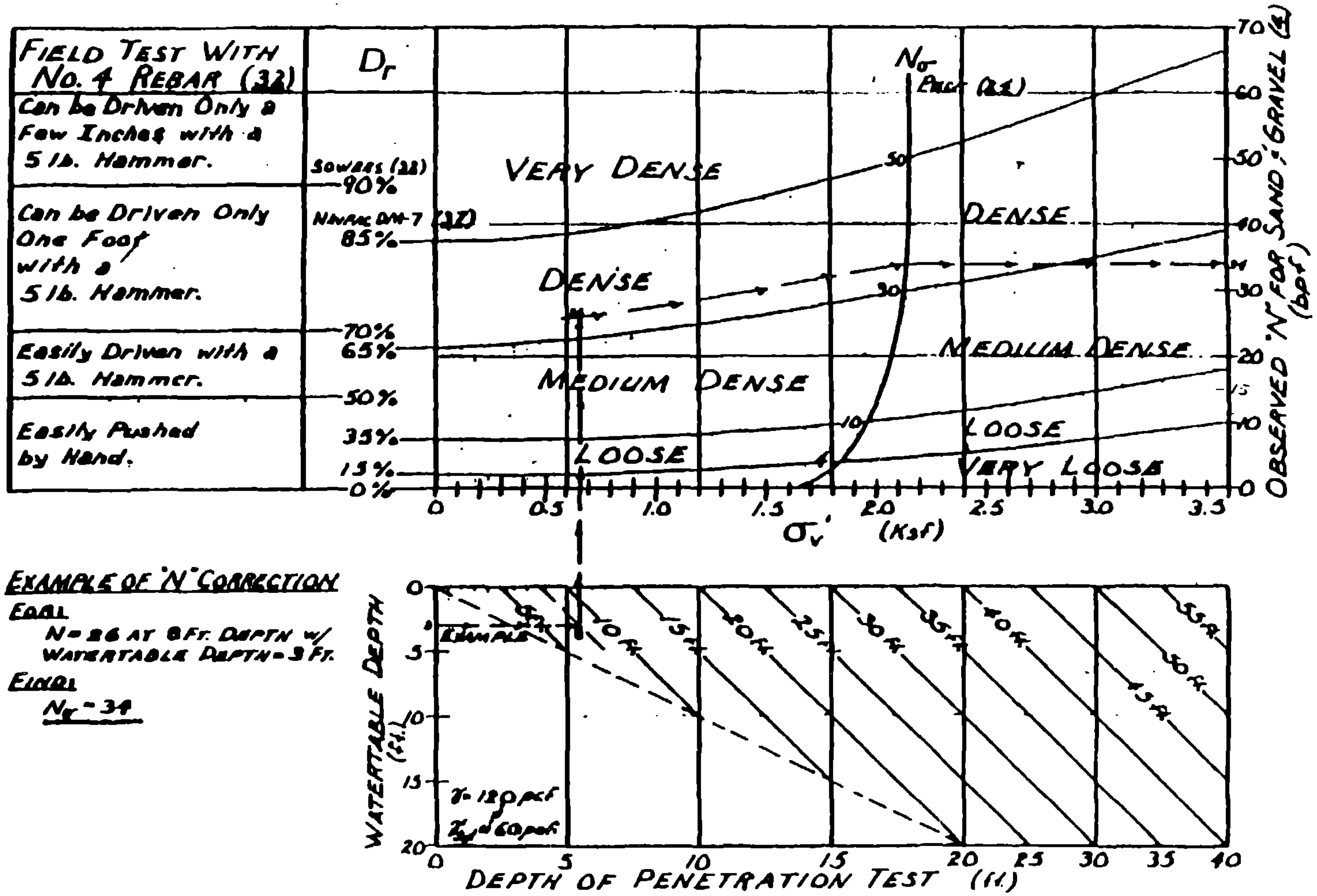


Figure 3-1A.—Cohesionless soil σ_v' versus N and D_r (from Transportation Engineering Handbook, 1981).

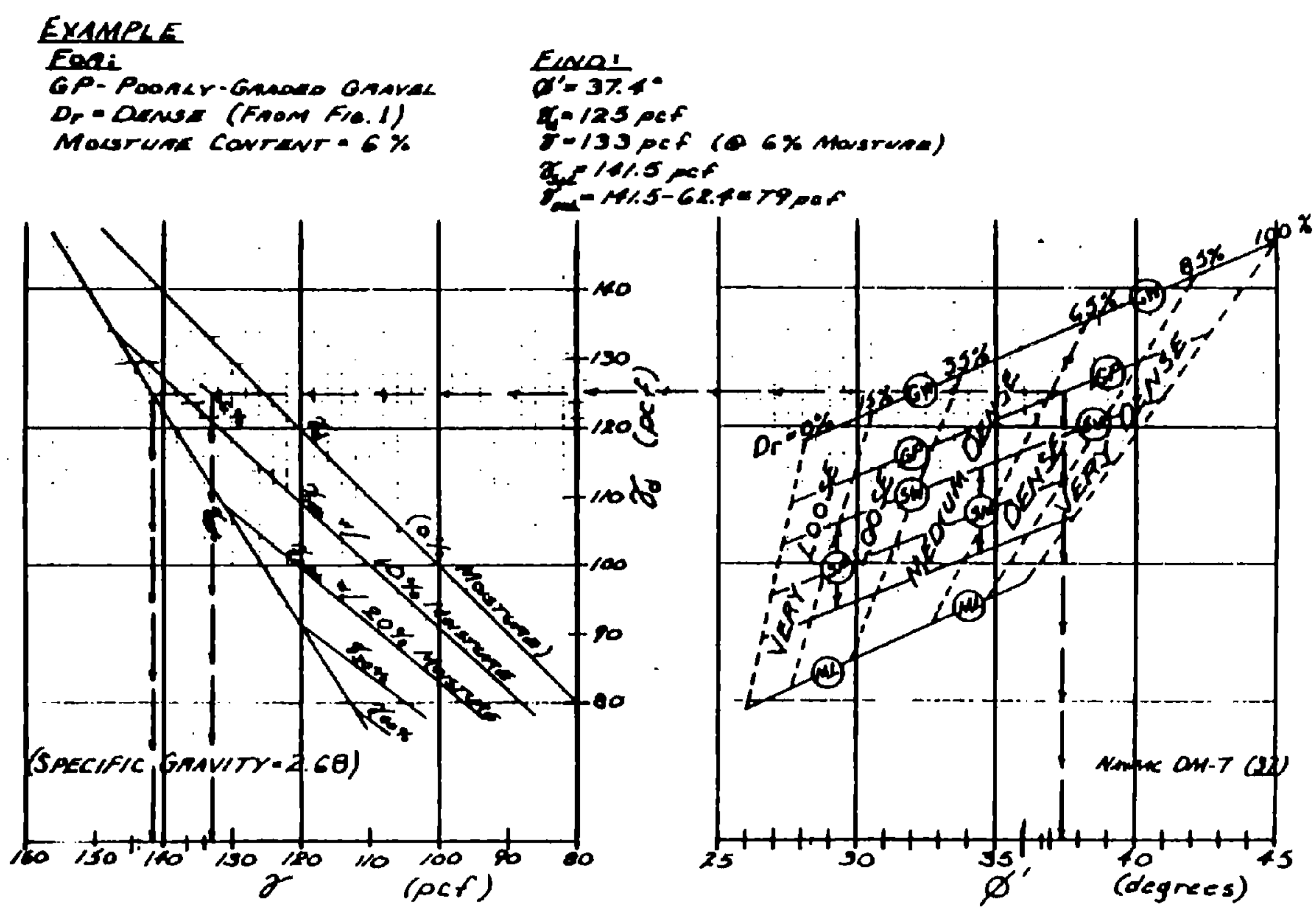
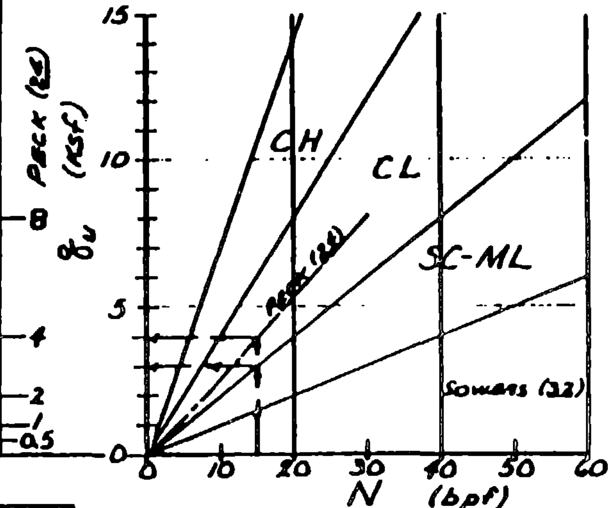


Figure 3-1B.—Cohesionless soil D_r versus γ and ϕ' (from Transportation Engineering Handbook, 1981).

Field Test Sowers (32)	Consistency
Dented Only Slightly by Pencil Point.	HARD
Dented Only Slightly by Finger Pressure.	VERY STIFF
Dented by Strong Pressure of Fingers.	STIFF
Molded by Strong Pressure of Fingers.	FIRM
Easily Molded by Fingers.	SOFT
Squeezes Between Fingers w/ Fist Closed.	VERY SOFT



Estimated Density Hough (14)				
Cohesive Soil	γ (pcf)		γ_{sub} (pcf)	
	Loose	Dense	Loose	Dense
Uniform Inorganic Silt	81	136	51	73
Sandy or Silty Clay	100	147	38	85
Skip-Graded Silty Clay w/ Gravel	115	151	53	89
Clay	94	133	31	71
Colloidal Clay	74	128	8	66
Organic Silt	87	131	25	69
Organic Clay	81	125	18	62

EXAMPLE

Given:
$N = 15$
Unified Class - CL-ML

Find:
Consistency - Stiff
Est. q_u = 3 ksf (Sowers)
= 4 ksf (Peck)
Use q_u = 3 ksf, Then:
Est. Cohesion $C_u = q_u/2$ = 1.5 ksf
(Assumes $\phi = 0$)
Est. Density: γ = 110 pcf & γ_{sub} = 50 pcf

Figure 3-1C.—Cohesive soils consistency versus N and q_u (from Transportation Engineering Handbook, 1981).

Table 3-3.—Typical properties of compacted materials.

Group symbol	Soil type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi)	Typical value of compression: At 3.6 tsf (50 psi)	Typical strength characteristics: Cohesion (as compacted) psf	Cohesion (saturated) psf	ϕ (Effective stress envelope) degrees	Tanϕ	Typical coefficient of permeability, k ft/min.
				Percent of original height						
GW	Well-graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}
GP	Poorly graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	>0.74	10^{-1}
GM	Silty gravels, poorly graded gravel-sand-silt.	120-135	12-8	0.5	1.1	...	...	>34	>0.67	$>10^{-6}$
GC	Clayey gravels, poorly graded gravel, sand, and clay	115-130	14-9	0.7	1.6	...	...	>31	>0.60	$>10^{-7}$
SW	Well-graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	0.79	$>10^{-3}$
SP	Poorly graded clean sands, sand-gravel mix.	100-120	21-12	0.8	1.4	0	0	37	0.74	$>10^{-3}$
SM	Silty sands, poorly graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	0.67	5×10^{-5}
SM-SC	Sand-silt clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	0.66	2×10^{-6}
SC	Clayey sand, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	0.60	5×10^{-7}
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	0.62	10^{-5}
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	0.62	5×10^{-7}
CL	Inorganic clay of low to medium plasticity	95-120	24-12	1.3	2.5	1800	270	28	0.54	10^{-7}

Table 3-3.—Typical properties of compacted materials (cont'd.).

Group symbol	Soil Type	Range of maximum dry unit weight, pcf	Range of optimum moisture, percent	Typical value of compression: At 1.4 tsf (20 psi) — Percent of original height	Typical value of compression: At 3.6 tsf (50 psi) — Percent of original height	Typical strength characteristics: Cohesion (as compacted) psf	Typical strength characteristics: Cohesion (saturated) psf	Typical strength characteristics: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tanϕ	Typical coefficient of permeability, k ft/min.
OL	Organic silts and silt-clays, low plasticity	80-100	33-21	...	...	...	...	...	...	...
MH	Inorganic, clayey silts, elastic silts	70-95	40-24	2.0	3.8	1500	420	25	0.47	5×10^{-7}
CH	Inorganic clays of high palsticity	75-105	36-19	2.6	3.9	2150	230	19	0.35	10^{-7}
OH	Organic clays and silty clays, etc.	65-100	45-21	...	...	...	...	...	...	...

Notes: (NAVFAC DM-7.2, 1982)
1. All properties are for condition of "standard Proctor" maximum density, except values of k which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown. (...) indicates insufficient data are available for an estimate.

Table 3-4 presents friction factors and adhesion values for interfaces of various structural and foundation materials. The values indicated are approximate and should be verified by field tests and/or previous experiences.

3D Earth Pressures

3D.1 General

Three states of stress within the backfill are of general interest to the retaining wall designer: (1) the active, (2) the passive, and (3) the at-rest. The at-rest case is associated with the backfill forces acting on a nonyielding retaining wall and on any retaining wall that has not yielded, for example, by the end of construction conditions. The active state of stress is associated with the failure conditions when all available shear strength is mobilized in the backfill, and is related to the wall moving away from the retained soil, as shown in example (a) of figure 3-2. The passive state of stress is associated with the failure condition in the backfill, and is related to the wall moving into the retained soil, as shown in example (b).

Figure 3-3 illustrates the relative amount of movement required by the wall to develop the various states of stress and the relative magnitude of the resulting lateral pressure.

Table 3-4.—Friction factors and adhesion for dissimilar materials.

Interface materials	Friction factor tan ϕ	Friction angle, ϕ, degrees	Adhesion C_A, psf
Mass concrete on the following foundation materials:			
Clean sound rock	0.70	35	
Clean gravel, gravel-sand mixtures, coarse sand	0.55–0.60	29–31	
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45–0.55	24–29	
Clean fine sand, silty or clayey fine to medium sand	0.35–0.45	19–24	
Fine sandy silt, nonplastic silt	0.30–0.35	17–19	
Very stiff and hard residual or preconsolidated clay	0.40–0.50	22–26	
Medium stiff and stiff clay and silty clay (Masonry on foundation materials has some friction factors.)	0.30–0.35	17–19	
Steel sheet piles against the following soils:			
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22	
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17	
Silty sand, gravel, or sand mixed with silt or clay	0.25	14	
Fine sandy silt, nonplastic silt	0.25	11	
Soft clay and clayey silt			100–600 600–1,200
Formed concrete or concrete sheet piling against the following soils:			
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40–0.50	22–26	

Table 3-4.—Friction factors and adhesion for dissimilar materials (cont'd.).

Interface materials	Friction factor tan ϕ	Friction angle, ϕ, degrees	Adhesion C_A, psf
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30–0.40	17–22	
Silty sand, gravel, or sand mixed with silt or clay	0.30	17	
Fine sandy silt, nonplastic silt	0.25	14	
Soft clay and clayey silt			200–700 700–1,200
Various structural materials:			
Masonry on masonry, igneous and metamorphic rocks:			
Dressed soft rock on dressed soft rock	0.70	35	
Dressed hard rock on dressed soft rock	0.65	33	
Dressed hard rock on dressed hard rock	0.55	29	
Masonry on wood (cross-grain)	0.50	26	
Steel on steel at sheet pile interlocks	0.30	17	

Note (NAVFAC DM-7.2, 1982):

Numbers shown are ultimate values and require sufficient movement for failure to occur. Where friction factor only is shown, the effect of adhesion is included in the friction factor

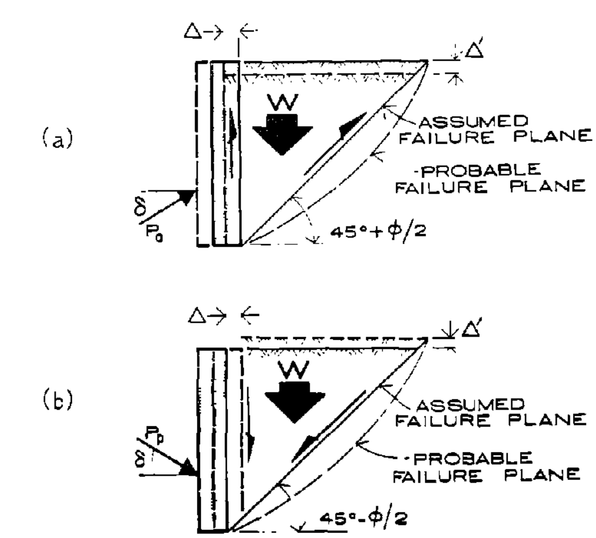


Figure 3-2.—Wall movement—(a) active pressure; (b) passive pressure.

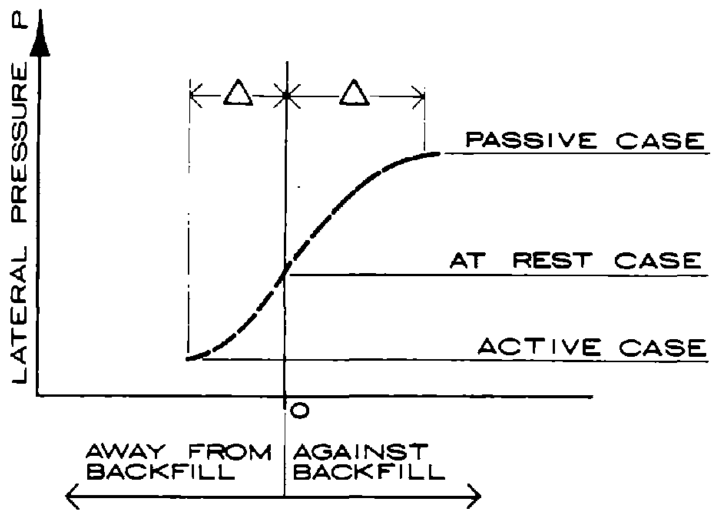


Figure 3-3.—Relative magnitude: active and passive pressures.

Table 3-5 presents a summary of minimum wall displacements required to develop the active and passive states of stress for various soil types as a function of wall height, *H*. The displacements are measured at the top of the wall.

Table 3-5.—Minimum required wall displacement (after Sowers and Sowers, 1970).

Soil	Active State	Passive State
Dense cohesionless	0.0005 H	0.005 H
Loose cohesionless	0.002 H	0.01 H
Stiff cohesive	0.01 H	0.02 H
Soft cohesive	0.02 H	0.04 H

After achieving the active or passive state of stress, soft cohesive soils will tend to creep (yield slowly). As creep proceeds, the soil approaches the at-rest state of stress. Consequently, soft cohesive soils in the active stress state will tend to increase in stress intensity and those in the passive state of stress will tend to decrease in stress intensity. Likewise, after achieving the active or passive states of stress, backfills of stiff clay of moderate-to-high plasticity tend to develop slicken sides and lose strength when allowed to strain. This loss of strength generally results in an increase of stress intensity for soils in the active state of stress and a decrease in stress for those in the passive state of stress.

Two commonly accepted approaches for determining the design earth pressure acting on a retaining wall are the semiempirical methods and the theoretical methods (see sections 3D.2 and 3D.3, respectively). In general, the use of the semiempirical method of analysis is reserved for standard retaining walls that are low (generally less than 20 feet in height) and have relatively small surface areas. Typically, these methods result in a conservatively designed wall. It is the conservatism of design and the attendant increase in construction costs that generally justify a comprehensive design, based upon theoretical aspects of soil mechanics, for larger and more complex problems and some smaller projects. In addition to the cost and time factors of designing high walls, safety is an important concern for walls greater than about 20 feet in height. The following subsections discuss semiempirical and theoretical design approaches.

3D.2 Semiempirical Methods

Since empirical design methods are based on a combination of the earth pressure theory and the observed performance of completed structures, the following assumptions have been made in the development of the semiempirical design approach:

(1) The backfill is drained.

(2) The wall will yield away from the backfill a sufficient amount to develop the active state of stress.

(3) The backfill will settle more than the wall.

(4) The earth pressure increases hydrostatically from the ground surface to the base of the wall.

The semiempirical design method consists of classifying the proposed project in terms of the backfill type, backfill slope, surcharge loading, and selecting the appropriate design parameters from the following tables and graphs.

The first step in the analysis consists of classifying the backfill material using soil descriptions presented in table 3-6.

Having determined the type of backfill material to be used, the geometry and surcharge conditions of the backfill are classified using table 3-7.

Table 3-6.—Types of backfill for retaining walls (after Terzaghi and Peck, 1967).

(1)	Coarse-grained soil without admixture of fine soil particles; very permeable (clean sand or gravel).
(2)	Coarse-grained soil of low permeability due to admixture of particles of silt size.
(3)	Residual soil with stones, fine, silty sand, or granular materials with conspicuous clay content.
(4)	Very soft or soft clay, organic silts, or silty clays.
(5)	Medium or stiff clay deposited in chunks and protected so that only a negligible amount of water enters the spaces between the chunks during floods or heavy rain. If this condition cannot be satisfied, the clay should not be used as backfill material. With the increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly.

Table 3-7.—Backfill geometry and surcharge conditions classification.

(1)	The surface of the backfill is plane, i.e., horizontal or sloped upward from the crest of the wall.
(2)	The surface of the backfill is sloped upward from the crest of the wall, but it becomes leveled at some elevation above the crest.
(3)	The backfill is surcharged.

Examples (a) and (b) in figure 3-4 show walls with plane-sloped backfills, as described in classification 1, table 3-7. The earth pressure distribution may be estimated with the aid of example (c), which presents K_v and K_h as a function of β, the slope angle. Using the values of K_v and K_h obtained from example (c), the horizontal and vertical pressures σ_h and σ_v are

$$\sigma_h = K_h H \tag{3-1}$$

and

$$\sigma_v = K_v H. \tag{3-2}$$

Similarly, the resultant of the total horizontal and vertical pressures are

$$P_h = 1/2 K_h H^2 \tag{3-3}$$

and

$$P_v = 1/2 K_v H^2. \tag{3-4}$$

The resultant, P_h, is assumed to act at a point $H/3$ above the base of the footing in a horizontal direction. As shown in examples (a) and (b) of figure 3-4, the effective height of the wall, H, is measured vertically through the heel of the wall from the base of the footing to the overlying ground surface. Terzaghi and Peck, 1967, suggested that for type 5 soils, the value of the effective wall height, H, be reduced by 4 feet for calculating the total pressure. However, they recommend that the point of application of the resultant be maintained at $H/3$ above the base, in which H is the unreduced effective height.

Examples (a), (b), and (c) in figure 3-5 show classification 2 backfill geometries. Classification 2 backfills slope upward from the crest of the wall and become level at some higher elevation. Charts (d) through (h) present K_h and K_v as a function of slope angle, soil type, and the ratio of H_l/H. As (a) through (c) show, H_l is the truncated slope height above the effective wall height, H. The calculation of stress and total load to any depth are similar to those for plane slopes, as shown in equations 3-1 through 3-4. The point of application of the total load is also at the lower third point of H. For type 5 soils, H, the effective wall height, should be reduced by 4 feet to calculate the magnitude of the resultants P_h and P_v. The point of action of P_h is assumed to be located at the third point of the unreduced height, H.

The surcharge loadings mentioned in classification 3 include point loads from the wheels of heavy vehicles and various loading patterns from stockpiling materials (e.g., logs or aggregate) such as line, strip, and aerial loads. The stress distribution resulting from the imposed surcharge is added vectorially to the earth pressures developed by the backfill to model the design pressures.

For a uniform surcharge, Δq, on a horizontal backfill the change in horizontal stress, $\Delta\sigma_h$, on a unit area of wall is

$$\Delta\sigma_h = C\Delta q \tag{3-5}$$

in which C is a coefficient based on soil type (see table 3-8). The resultant of the increase in stress is obtained from

$$\Delta P_h = \Delta\sigma_h H \tag{3-6}$$

in which H is the effective wall height and $\Delta\sigma_h$ is obtained from equation 3-5. The resultant of the increase in stress, ΔP_h, acts at a point $H/2$ above the foundation base. The increase in vertical stress resulting from a uniform surcharge is equal to the applied surcharge.

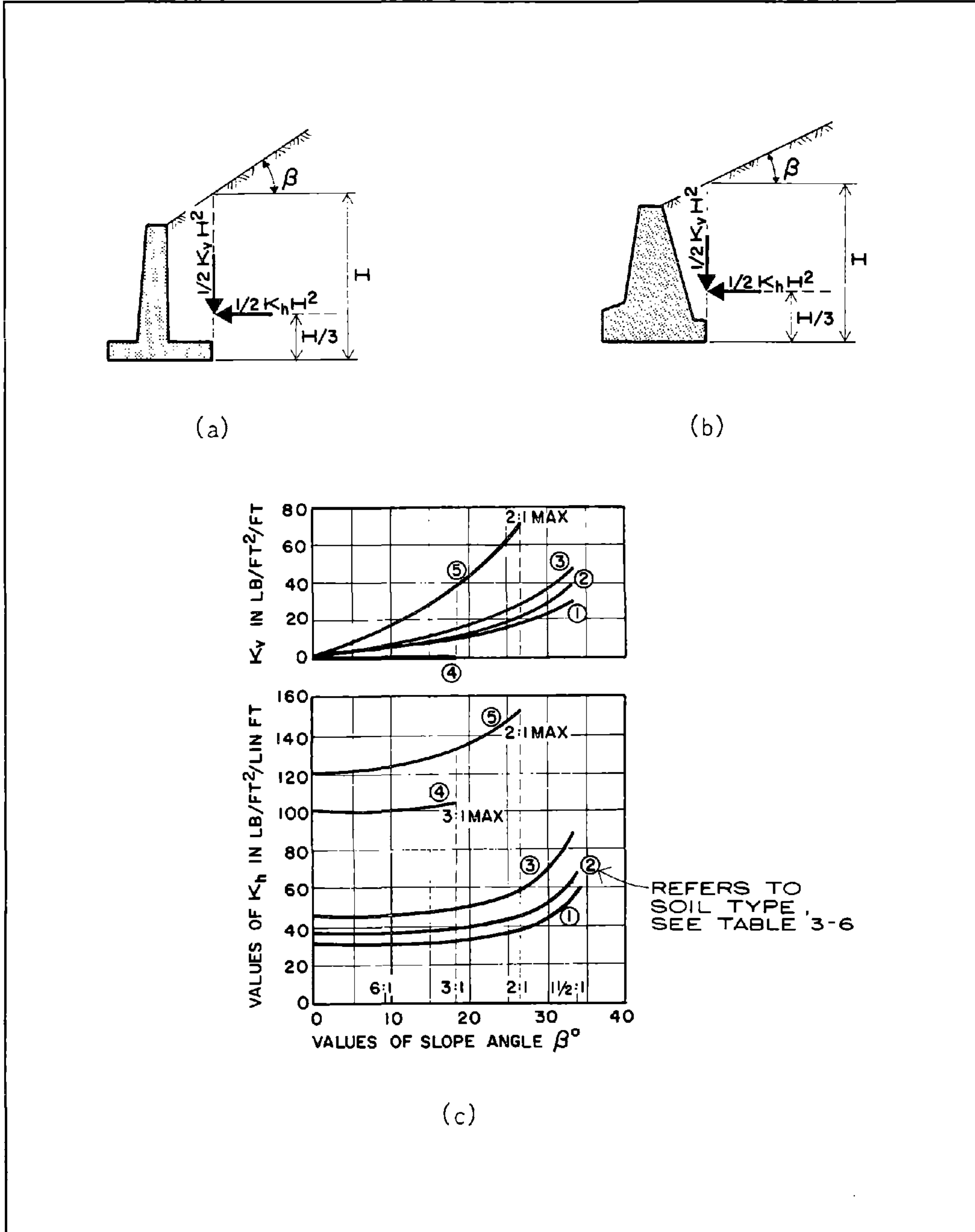


Figure 3-4.—Backfill coefficients for a plane backslope (after NAVFAC DM-7.2, May 1982).

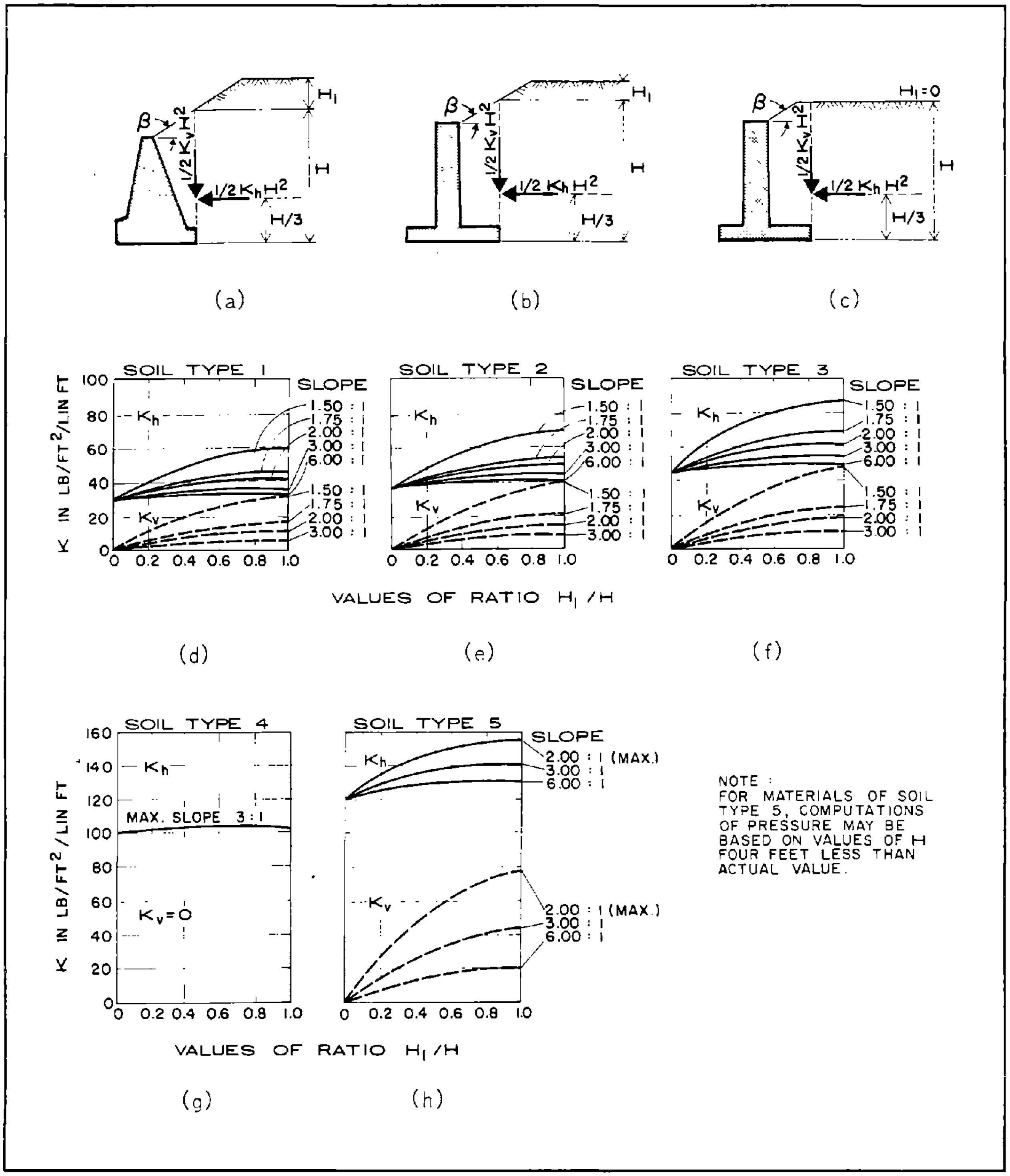


Figure 3-5.—Backfill geometries and coefficients for a transitioned backfill (after NAVFAC DM 7.2, May 1982).

The influence of a line load, q', per unit length parallel to the crest of the wall, as shown in example (a) of figure 3-6, can be computed using

$$\Delta P_h = Cq' \tag{3-7}$$

in which ΔP_h is the resultant horizontal force and C is obtained from table 3-8. As shown in example (a) of figure 3-6, the point of application of the change in horizontal force, ΔP_h, is at point d. Point d can be established by constructing a line from c, the point of application of the line load, downward at an angle of 40° to the horizontal until it contacts the wall at point d_1. The point of application, d, is then located on the vertical plane, ab, at height d_1. This rule for locating the point of application of the resultant horizontal force is independent of the location of the parallel line load. If the point of application, d, is located below the base of the wall, the influence of the line load can be disregarded.

Table 3-8.—Values of C (after Terzaghi and Peck, 1967).

Type of Soil*	C
1	0.27
2	0.30
3	0.39
4	1.00
5	1.00

* *Taken from table 3-6.*

A line load will also result in an increase in the vertical stress on the heel of the wall. This stress can be estimated by distributing the line load over an equilateral triangle with its apex located at point c (see example (b) in figure 3-6).

The load is distributed to the depth of the top of the footing. When computing the increase in vertical force, only that portion lying over the heel of the wall should be considered. Hence, in example (b) of figure 3-6, the net increase in vertical stress is

$$\Delta\sigma v = \frac{q'}{\overline{ef}} \tag{3-8}$$

and the net increase in vertical force on the heel of the wall is

$$\Delta P_v = (\Delta\sigma_v)\ (\overline{eg}). \tag{3-9}$$

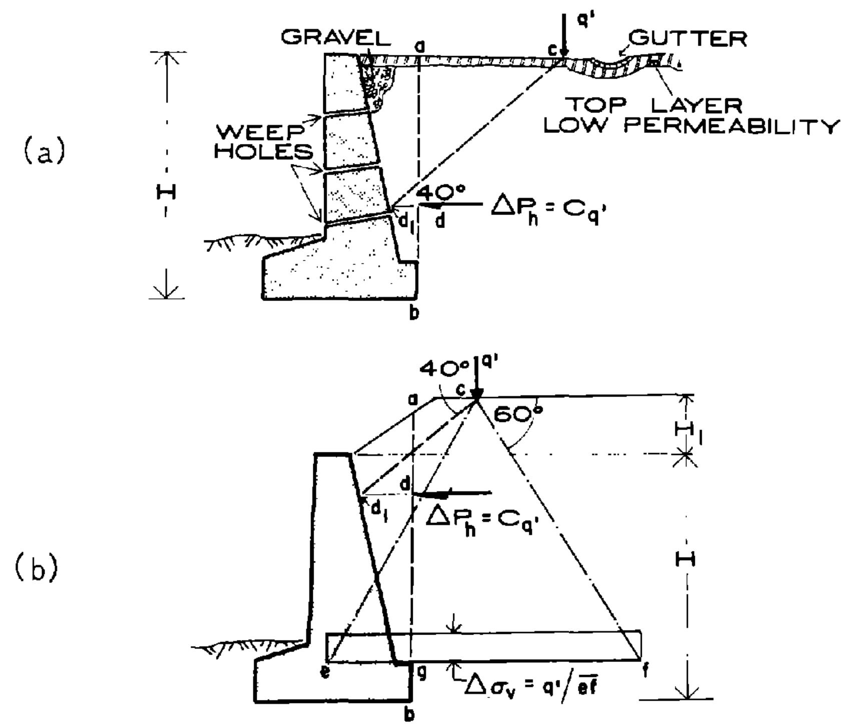


Figure 3-6.—Line loads—magnitude and action of resultant (after Terzaghi and Peck, 1967).

Semiempirical designs are sufficiently conservative to provide an adequate factor of safety against failure for all point loading conditions which are legal highway loads (except by special use permit). For high intensity point loads, such as the wheel loads for heavyweight equipment, it is recommended that a comprehensive analysis of the problem be completed as will be described in Section 3D.5, "Surcharge Loadings by the Theory of Elasticity."

3D.3 Theoretical Methods

The resultant of the lateral earth pressure, P_h, acting on a retaining structure, regardless of the state of stress, is a function of the following variables:

Υ = unit weight of backfill

$\bar{c}$ = effective stress cohesion intercept of backfill

ϕ = effective stress friction angle of backfill

δ = soil wall cohesion

C_w = soil wall cohesion

H = effective wall height

β = slope of backfill

α = slope of back of wall with backfill

ρ = slope of failure plane

U = pore water pressure in backfill

A number of analysis methods have been developed to evaluate lateral earth pressure, each of which makes different assumptions about the combinations of the variables listed above and the development of the failure plane through the retained soil. For given sets of conditions, each of the theories is an acceptable method of analysis. Three of the most commonly accepted methods of analysis are the Rankine, Coulomb, and log spiral. Of the three, the log spiral is generally considered the most accurate.

The following recommendations are made for computing passive earth pressures:

(1) The Coulomb method should only be used for $\beta < 10°$ and $\delta < \phi/3$.

(2) The Rankine method should be used for projects that can afford a conservative design, or when the assumed strength parameters are utilized.

(3) The log spiral method should be used in all cases that require a refined design from the standpoint of engineering or economics.

The following paragraphs will show equations and charts to aid the engineer in the development of earth pressure diagrams for the design of earth and rock retaining walls. These charts and tables will be referred to in Chapter 4, "Specific Wall Designs." The development of these equations, tables, and charts appears in all texts on the fundamentals of soil mechanics.

3D.3.1 The Rankine Theory

The following assumptions are made in the Rankine theory:

(1) The soil is homogeneous and isotropic, in a state of plastic equilibrium, and possesses internal friction.

(2) The failure surface within the backfill is planar.

(3) The shear strength is mobilized uniformly on all planes throughout the backfill.

(4) The presence of the wall does not influence the state of stress in the backfill.

(5) The failure is a two-dimensional problem.

(6) The resultant, P_a, is inclined at angles to the wall.

Example (a) in figure 3-7 shows the soil structure system for the Rankine active case; and (b) and (c) show the resulting force polygons for the cohesionless and cohesive cases, respectively. Equations 3-10 through 3-15 summarize the Rankine active case.

$$\sigma_a = \Upsilon h K_a, \tag{3-10}$$

$$P_a = 1/2\ \Upsilon H^2 K_a \tag{3-11}$$

in which

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \tag{3-12}$$

Equation 3-12 reduces to

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45 - \phi/2) \tag{3-13}$$

for a level backfill; that is, $\beta = 0$.

Figure 3-8 shows that the stress distribution and the resultant of the stresses, P_a, developed in the Rankine active case are oriented at angle β parallel to the surface of the backfill. Also shown on the figure is the location of the resultant, P_a, at a point $H/3$ above the base of the wall. A common and slightly more conservative approach is to assume that the stress distribution and resultant act horizontally.

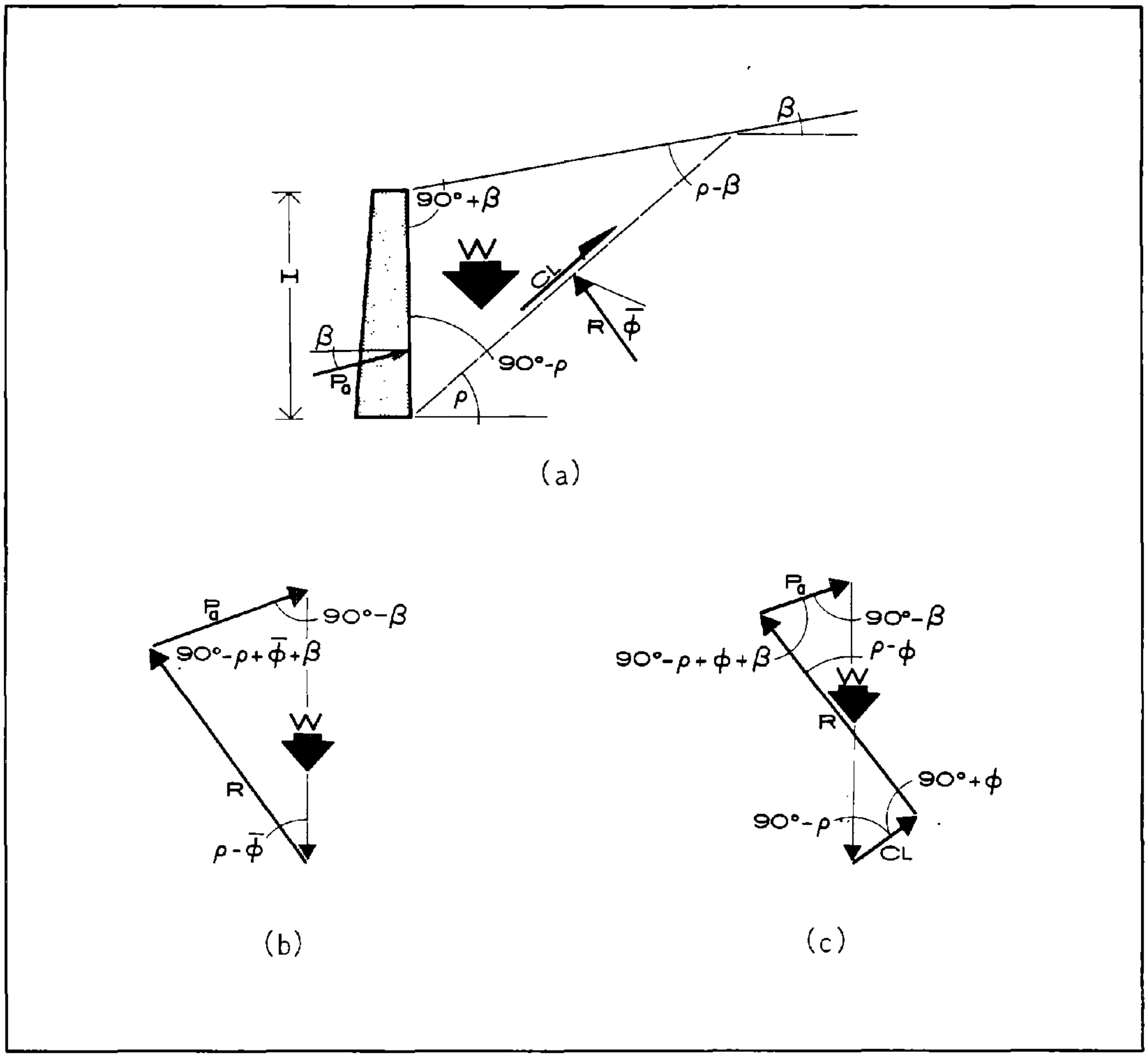


Figure 3-7.—Rankine active case.

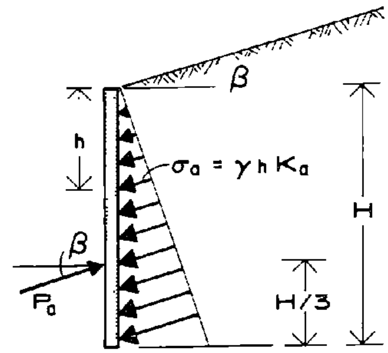


Figure 3-8.—Earth pressure diagram of the Rankine active case.

Figure 3-9 shows a method for determining the location and orientation of the resultant, P_a, using the Rankine earth pressure solution for a backfill with a transition in the backslope.

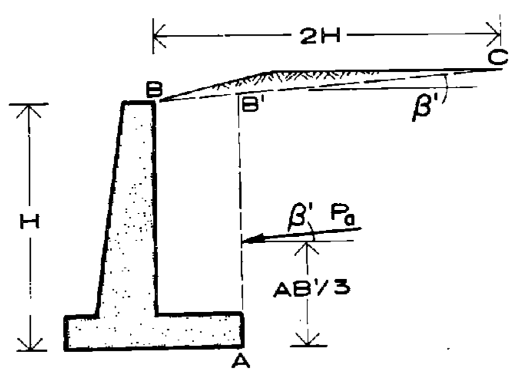


Figure 3-9.—Transitional backfill for the Rankine active case.

Cohesion, c, can be included in the Rankine theory by use of equations

$$\sigma_a = \gamma HKa - 2c\sqrt{K_a} \tag{3-14}$$

and

$$P_a = 1/2\gamma H^2 K_a - 2cH\sqrt{K_a} \tag{3-15}$$

in which K_a is obtained from equation 3-12. A comparison of equations 3-11 and 3-15 shows that the presence of cohesion reduces the Rankine active pressure.

The theoretical, unsupported height of a vertical cut, H_c, can be obtained from the Rankine equations by solving equation 3-15 for $P_a = 0$, resulting in

$$H_c = \frac{4c}{\gamma\sqrt{K_a}} \tag{3-16}$$

Similarly, the solution of equation 3-14 for $\sigma_a = 0$ reveals that the theoretical depth of a tension crack developing in the backfill is

$$h_c = \frac{2c}{\gamma\sqrt{K_a}} \tag{3-17}$$

If the backfill does exhibit cohesion, the total height, H, must be adjusted because of the reduction in cohesion due to a tension crack and the potential for developing hydrostatic pressure in the tension crack in the backfill. Example (a) in figure 3-10 shows a wall with a tension crack. Example (b) in figure 3-10 shows the resulting Rankine active stress distribution for a level backfill with cohesion but no wall friction or ground water table. If the tension crack were filled with water, an equivalent triangle of pressure would be distributed along *ab*, as shown in example (b) of figure 3-10.

Example (a) in figure 3-11 shows the soil structure system for the Rankine passive case. Example (b) in figure 3-11 shows the resulting force polygon for the cohesionless case. Example (c) in figure 3-11 presents the force polygon for the cohesive case.

The equations for the Rankine passive case can be presented in the following forms:

$$\sigma_p = \gamma H K_p \tag{3-18}$$

and

$$P_p = 1/2\ \gamma H^2 K_p \tag{3-19}$$

in which

$$K_p = \cos\beta\ [\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\overline{\phi}}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\overline{\phi}}}] \tag{3-20}$$

When the backfill is at level, $\beta = 0$, equation 3-20 reduces to

$$K_p = \frac{1 + \sin\overline{\phi}}{1 - \sin\overline{\phi}} \tag{3-21}$$

Similarly, for the Rankine passive case with cohesion, equations 3-14 and 3-15 may be rewritten as

$$\sigma_p = \gamma H K_p + 2c\sqrt{K_p} \tag{3-22}$$

and

$$P_p = 1/2\gamma H^2 K_p + 2cH\sqrt{K_p} \tag{3-23}$$

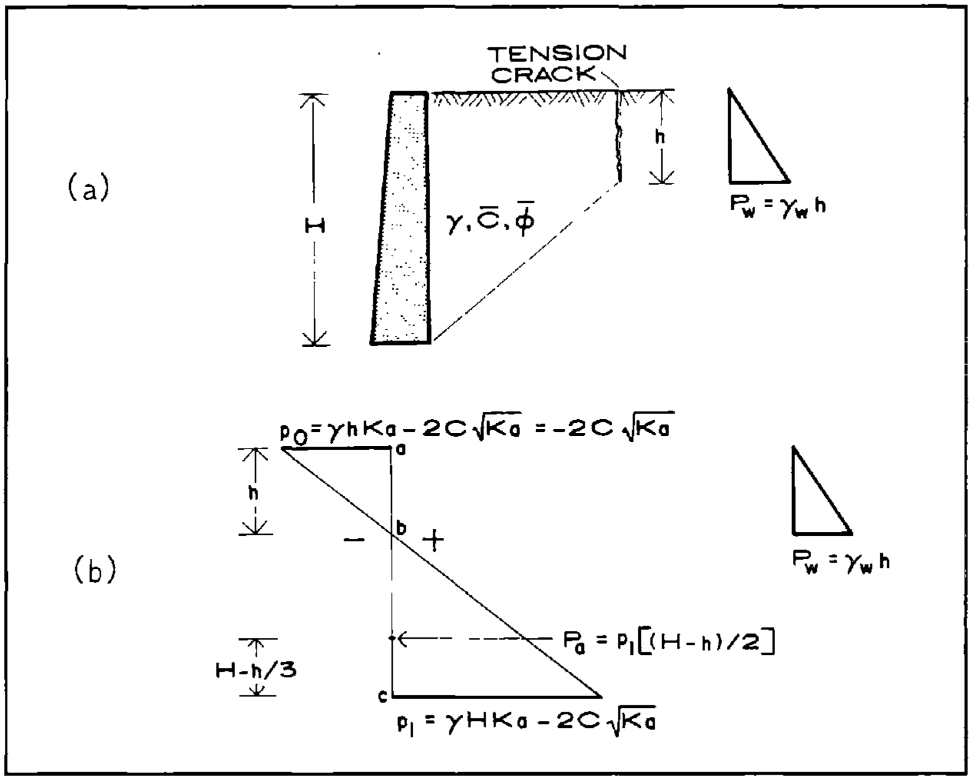


Figure 3-10.—Rankine active case with tension crack.

3D.3.2 The Coulomb Theory

The following basic assumptions are made in the Coulomb analysis:

(1) The soil is homogeneous and isotropic, in a state of plastic equilibrium, and possesses internal friction and cohesion.

(2) The failure surface in the backfill is planar.

(3) The shear strength, τ, is mobilized uniformly along the failure plane, and $\tau = \bar{c} + \bar{\sigma}_n \tan\bar{\phi}$.

(4) The failure wedge is a rigid body.

(5) There is wall friction; that is, as the failure wedge moves, the shear strength along the soil wall interface is mobilized.

(6) The failure is a two-dimensional problem.

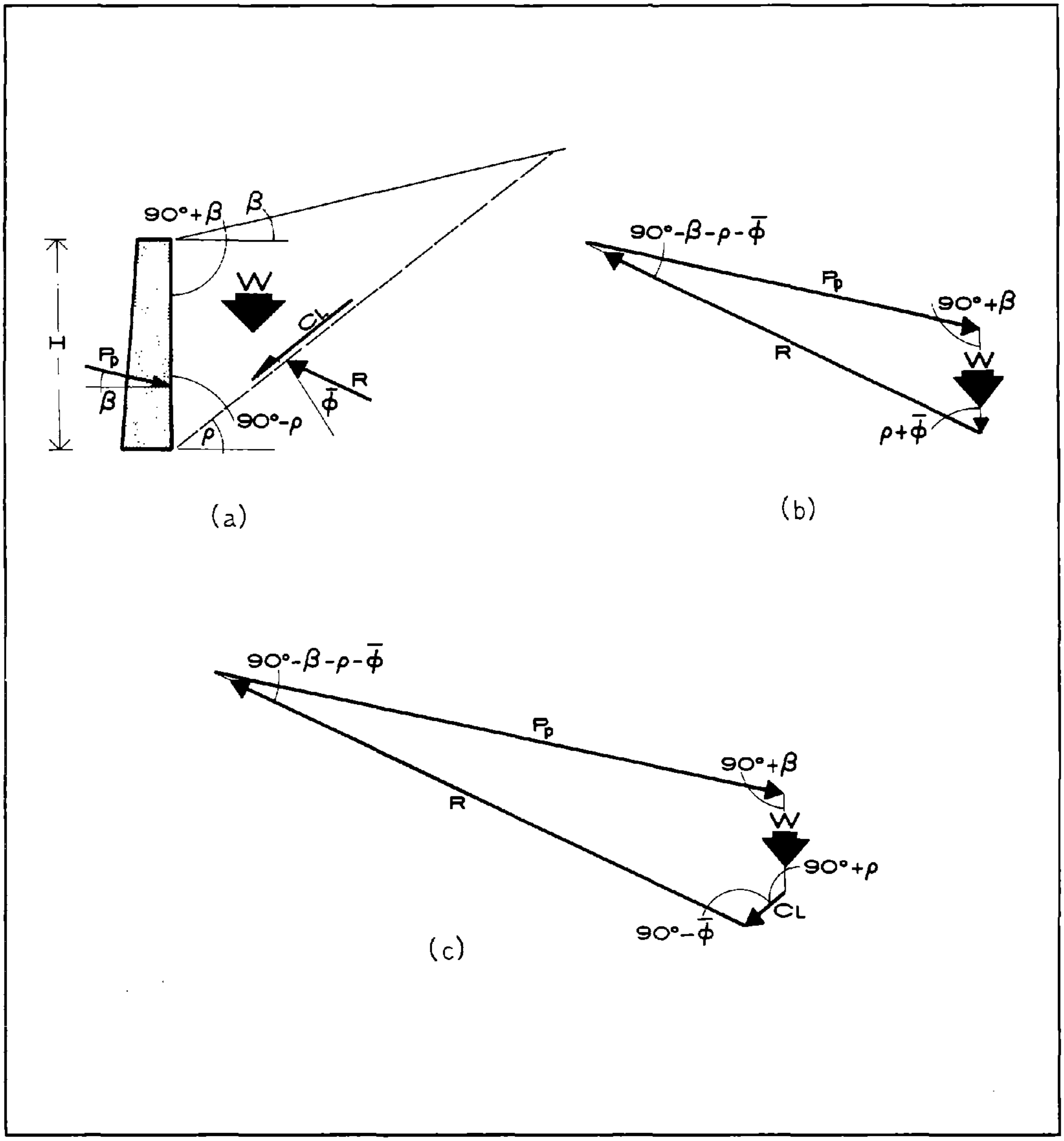


Figure 3-11.—Rankine passive case.

Example (a) in figure 3-12 shows the soil structure system for the Coulomb active case. Examples (b) and (c) in figure 3-12 show the resulting force polygons for cohesion, $\bar{c}$, equal to zero and a value greater than zero, respectively. A comparison of these example shows that the presence of cohesion reduces the value of, P_a, the total active thrust. However, time-dependent characteristics of the soil frequently cause $\bar{c}$ to approach zero with an attendant increase in P_a

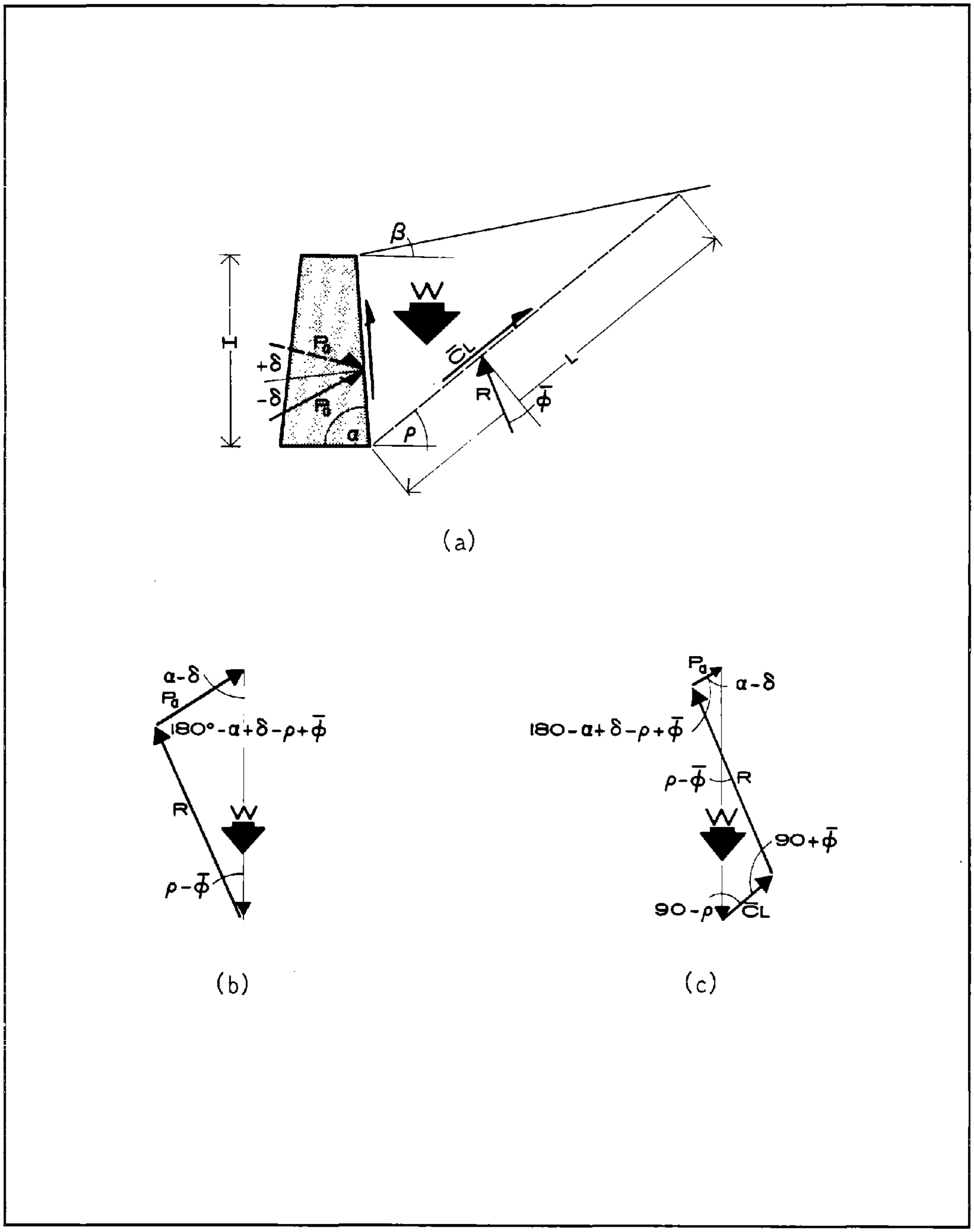


Figure 3-12.—Coulomb active case.

Equations 3-24 through 3-26 summarize the Coulomb active case for noncohesive soils.

$$\sigma_a = \gamma H K_a \tag{3-24}$$

and

$$P_a = 1/2\ \gamma H^2 K_a \tag{3-25}$$

in which

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2\alpha\ \sin(\alpha - \delta)\ [1 + \sqrt{\dfrac{\sin(\phi + \delta)\ \sin(\phi - \beta)}{\sin(\alpha - \delta)\ \sin(\alpha + \beta)}}\ ]^2} \tag{3-26}$$

For a smooth, vertical wall with a horizontal backfill, that is, $\beta = \delta = 0$ and $\alpha = 90°$, equation 3-25 simplifies to

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} \tag{3-13}$$

Referring to equations 3-24, 3-25, and 3-13, it becomes apparent that the Rankine theory is a special case of the Coulomb theory.

Example (a) in figure 3-13 shows the soil structure system for the Coulomb passive case. In the same figure, examples (b) and (c) show the resulting force polygons for cohesion equal to zero and a value greater than zero, respectively. A comparison of these examples shows that the presence of cohesion increases the value of P_p, the total passive thrust.

The following equations summarize the Coulomb passive case:

$$\sigma_p = \gamma H K_p \tag{3-27}$$

and

$$P_p = 1/2 \gamma H^2 K_p \tag{3-28}$$

in which

$$K_p = \frac{\sin^2(\alpha + \phi)}{\sin^2\alpha \sin(\alpha + \delta)\left[1 - \sqrt{\dfrac{\sin(\phi + \delta)\ \sin(\phi + \beta)}{\sin(\alpha + \delta)\ \sin(\alpha + \beta)}}\right]^2} \tag{3-29}$$

When using the Coulomb analysis, recall that for values of $\delta > 10°$ and $\delta > \phi/3$ the computed values of passive earth pressure become nonconservative.

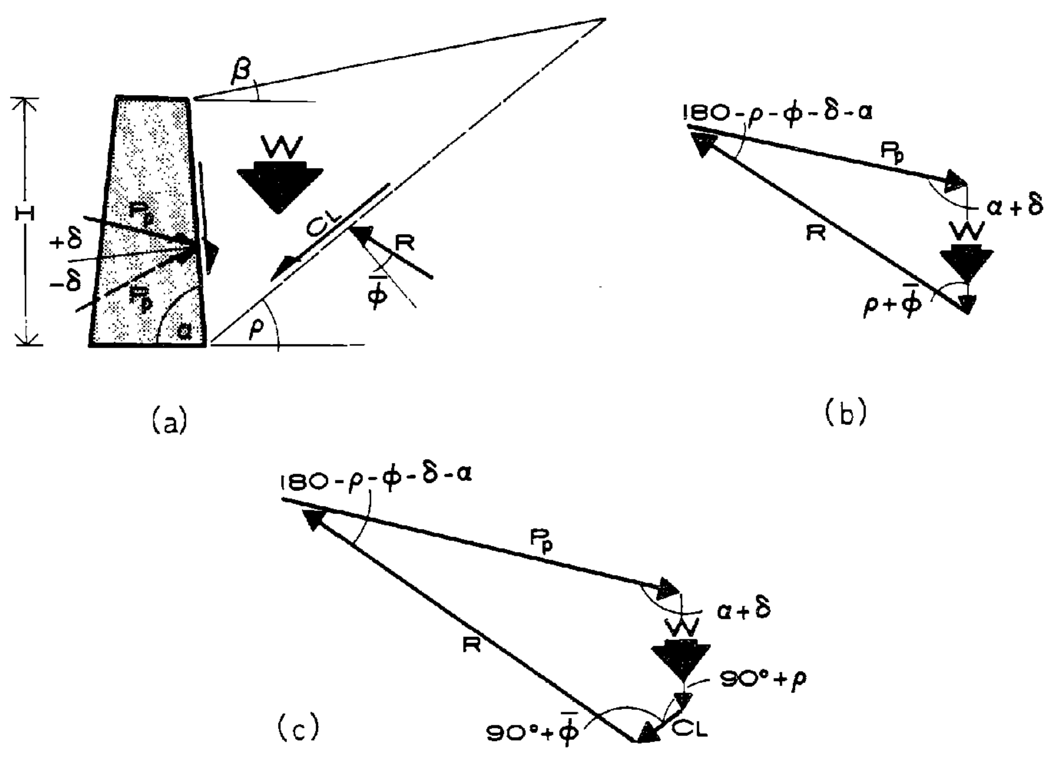


Figure 3-13.—Coulomb passive case.

3D.3.3 The Log Spiral Theory

A plane failure surface has been assumed in the Rankine and Coulomb methods of analysis: In fact, the failure surface observed in the field is curved as a result of the increase in vertical stress close to the wall from the downward-acting tangential shearing forces of the soil on the wall (see figure 3-14). The curvature degree of the failure surface increases with increasing values of δ, the wall-soil friction angle. Since the log spiral method of analysis more closely represents the actual failure surfaces, particularly in the case of passive resistance (Terzaghi and Peck, 1967), it is recommended for all cases requiring a refined design from the standpoint of engineering and/or economics.

Figure 3-15 shows the earth pressure coefficients, reduction factors, and equations required for computation of lateral earth pressure for cohesionless soil.

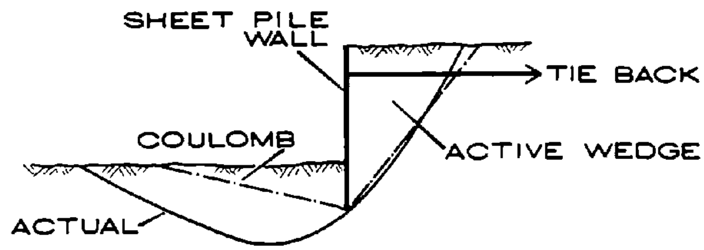


Figure 3-14.—Comparison of Coulomb and actual failure surfaces.

3D.3.4 At-Rest Conditions

Experience shows that under certain conditions it is prudent to design retaining structures for lateral earth pressures that are in excess of the active state of stress. Typically, this is done to minimize the deflections that the wall will realize under long-term operating conditions. Casagrande (1973) has indicated that only under ideal conditions, such as uncompacted, free-draining granular backfill, will the wall maintain the minimum active state of stress. In all other cases the earth pressure will be greater, approaching or exceeding the K_0, or at rest. Lee (1975) has proposed to use K_0 stresses in the design of reinforced earth walls.

For design considerations, K_0 can be computed using

$$K_O = 1 - \sin \overline{\phi} \tag{3-30}$$

The coefficient of earth pressure at rest, K_0, may be used in either the Rankine or Coulomb equations for the computation of the lateral earth pressure (see equations 3-11 and 3-25). Depending upon the assumed boundary conditions, the resultant should be located and oriented in accordance with the guidelines presented in the preceding sections on Rankine and Coulomb theories.

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

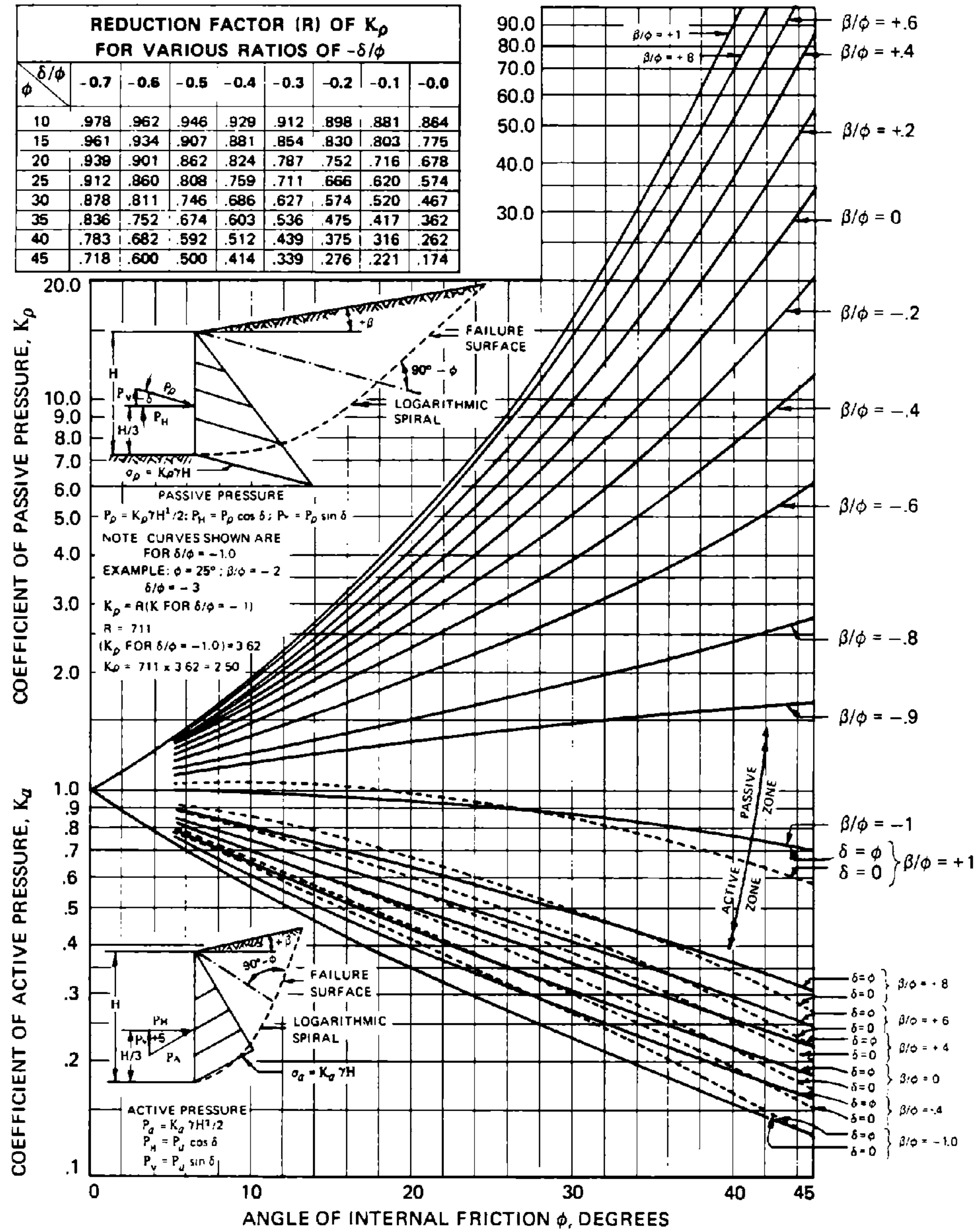


Figure 3-15.—Sloping backfill active and passive coefficients with wall friction (after NAVFAC Dm-7.2, May 1982).

3D.4 Graphical Solutions

3D.4.1 Culmann's Method

Culmann's method uses the Coulomb assumptions and is the simplest of all the graphical solutions. The method has the capability to account for wall friction, irregularity of the backfill, surcharges (concentrated or distributed), and the angle of internal friction. The method can be adapted for stratified deposits of varying density; however, the angle of internal friction must be assumed to be constant throughout the backfill. The Culmann method may also be used to analyze the passive case; however, for values of δ greater than $\phi/3$ and β greater than 10 degrees, the log spiral or the generalized method presented here is recommended. As shown in figure 3-16, the steps for the Culmann method for the active case are as follows:

(1) Draw a cross-section of the retaining wall to a convenient scale, including the ground surface, point loads, surcharges, and the geometry of the wall.

(2) Construct line *AC* at angle $\bar{\phi}$ to the horizontal. Note that for a cantilever wall, *A* is located at the heel of the wall (see example (c) in figure 3-16).

(3) Construct line AD at angle θ to AC.

(4) Develop trial wedges ABC_1, ABC_2, . . . , ABC_n using the surface of the backfill as a guide.

(5) Find the weight, W_n, of each wedge; consider the soil stratification and ground water. Use simple geometric shapes to compute areas.

(6) Plot the weight of the wedges, W_1, W_2, W_3, . . . , and W_n along *AC* to a convenient scale.

(7) Draw lines parallel to line *AD* through points W_1, W_2, W_3, . . . , and W_n. These lines should terminate on the side of the corresponding failure wedge, e.g., W_1 to side AC_1, W_2 to side AC_2, etc.

(8) Fit a smooth curve through the points of intersection on the failure wedges (the Culmann line). Draw a line or lines parallel to line *AC* and tangent to the Culmann line. (Depending on surcharge conditions, it may be possible to draw more than one.)

(9) Construct a line or lines back to AC and parallel to AD through the tangent point or points constructed in step 8. Using the same relative scale utilized to establish the weights W_1, W_2, . . . , and W_n, determine P_a maximum (that is, the largest value if several tangents are present).

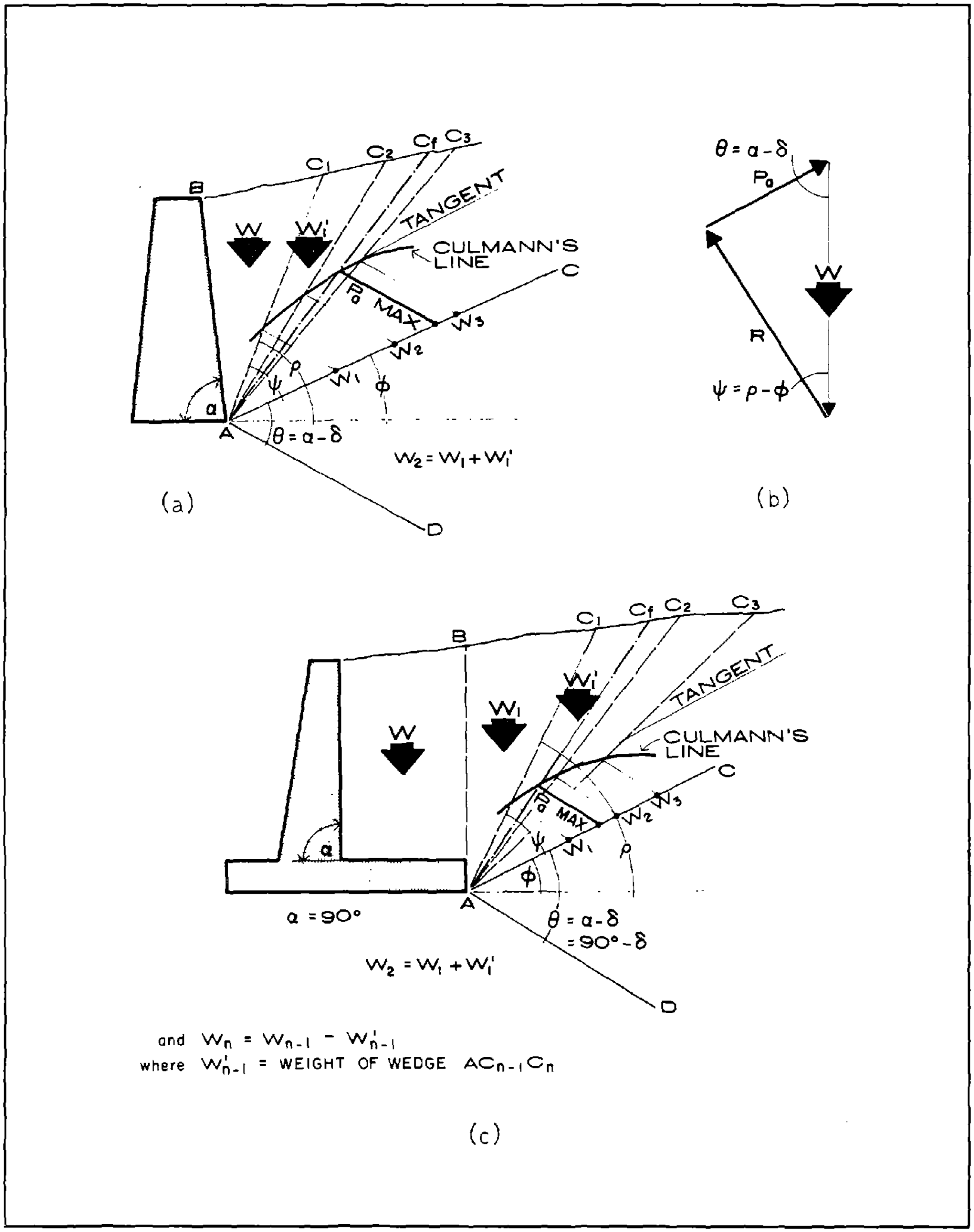


Figure 3-16.—Culmann's solution for active earth pressure.

(10) Construct the failure surface through A and the intersection P_a maximum and the Culmann line. The point of application of P_a is determined by the following procedure:

Condition 1. No concentrated loads, as shown in example (a) of figure 3-17.

(a) Determine the center of gravity of the failure wedge. This can be accomplished by balancing a cardboard construction of the wedge at several orientations on a knife edge (a three-sided scale) and marking the axis of balance. The intersection of two or more axes is the center of gravity.

(b) Through the center of gravity draw a line parallel to the failure plane until it intercepts AB, the back of the wall or a plane through the heel of the wall. P_a acts at an angle δ to a line perpendicular to AB, as demonstrated in example (a) of figure 3-17.

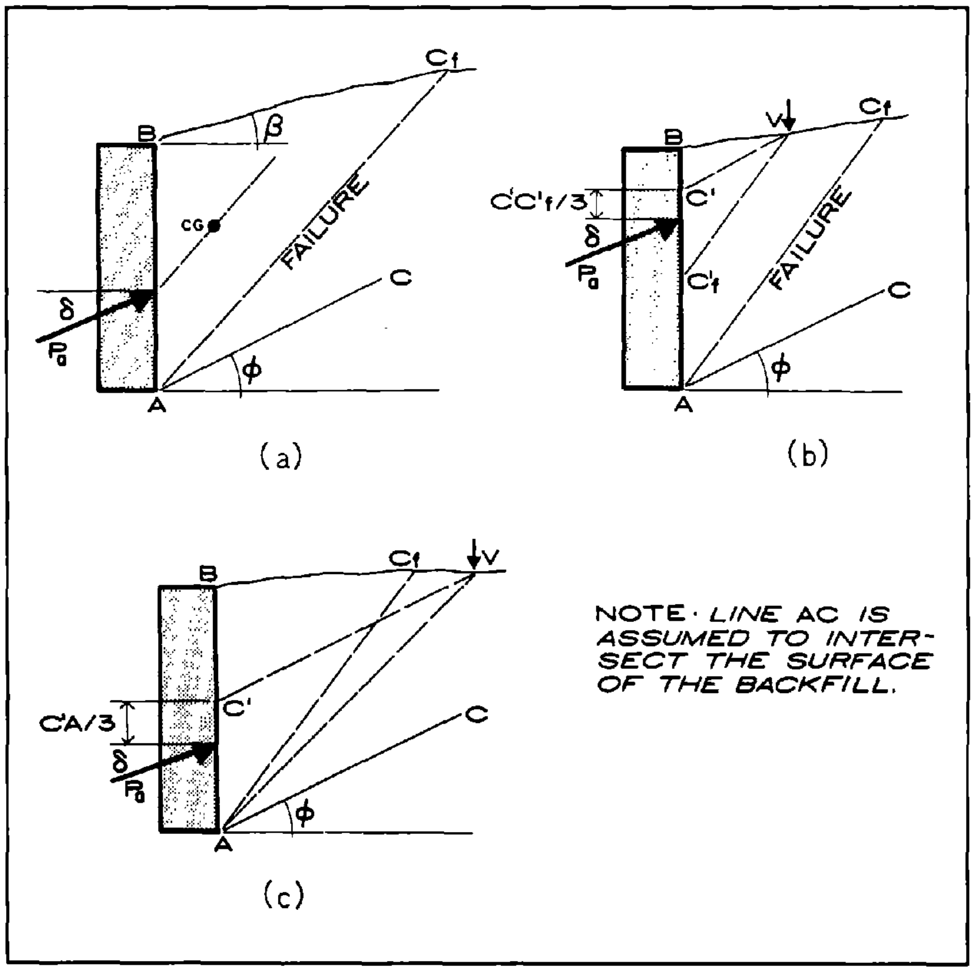


Figure 3-17.—Location of resultant of the Culmann solution.

Condition 2. Concentrated load or line load within the failure wedge, as shown in example (b) of figure 3-17.

(a) Draw line VC' parallel to AC and line VC_f parallel to AC_f.

(b) The point of application of P_a is $CC'_f/3$ below C', as shown in example (b) of figure 3-17.

Condition 3. Concentrated load or line load outside of the failure wedge, as shown in example (c) of figure 3-17.

(a) Draw a line from the concentrated load V to point A.

(b) Draw VC' parallel to AC.

(c) Take $C'A/3$ below point C' as the point of application.

3D.4.2 The Trial Wedge Solution

The trial wedge solution is similar to the Culmann solution; however, the influence of cohesion may be included in the analysis. Example (a) in figure 3-18 shows an idealized wall and backfill with the forces acting on the failure wedge for the active case. The following steps outline a graphical method of solution for determining the maximum value of Pa shown in the force polygon in example (b) of figure 3-18.

(1) Draw a cross-section of the site with a tentative wall section. Using equation 3-15, compute the theoretical depth of the tension crack, h_c, and plot the tension crack profile, as shown in example (a) of figures 3-18 and 3-19.

(2) Draw the trial wedges ABE_1D_1, ABE_2D_2, . . . , and ABE_nD_n and compute the weights of the wedges W_1, W_2, . . . , and W_n.

(3) Compute c_w and c_s. Construct c_w to a convenient scale at the effective slope of the wall, (see examples (a) and (b) in figure 3-19). When computing c_w and c_s, reduce the length of the failure surface by an amount equal to the depth of the tension crack. Frequently, c_w is assumed equal to zero.

(4) Construct weight vectors W_1, W_2, . . . , and W_n along OY.

(5) From the terminus of c_w construct c_{s1}, c_{s2}, . . . , and c_{sn} at the respective slopes of the assumed failure planes. Note that c_w is a constant for each wall geometry.

(6) Through points W_1, W_2, . . . , and W_n; that is, the ends of the weight vectors, construct P_a at angle δ to a perpendicular to the effective wall surface.

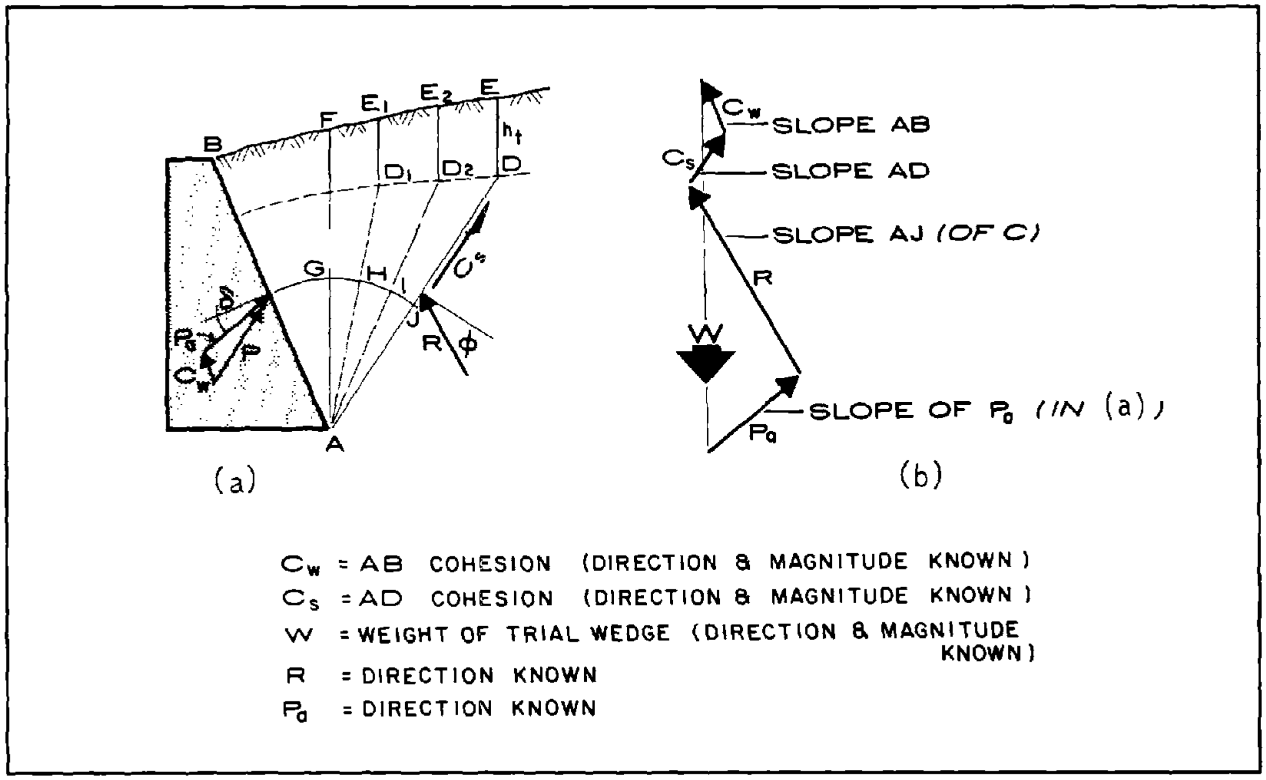


Figure 3-18.—Trial wedge method free body diagram for an active case.

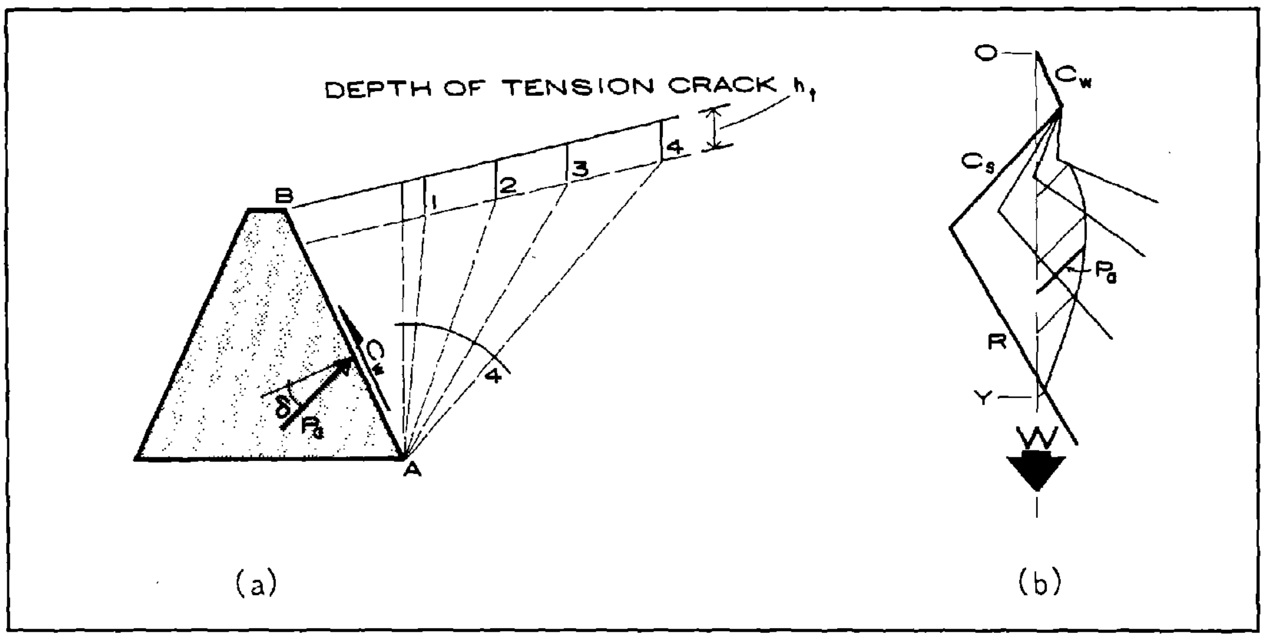


Figure 3-19.—Trial wedge solution for an active case.

(7) Construct the resultant, R, through the terminus of c_s at a slope perpendicular to the assumed failure surfaces AD_1, AD_2, . . . , and AD_n. This method is greatly facilitated by the use of an adjustable protractor.

(8) Fit a smooth curve through the points established by the intersection of respective P_a and R vectors (see example (b) in figure 3-19).

(9) Construct vertical lines tangent to the curve constructed in step 8. The largest value of P_a is the design value.

3D.4.3 Generalized Graphic Solutions

Generalized graphic solutions for the active and passive cases are presented in examples (a) and (b) in figure 3-20, respectively. The importance of understanding the free body and force diagrams presented in the preceding sections on Rankine and Coulomb earth pressures is now apparent. The use of this approach permits the engineer to evaluate all possible variables acting on a retaining structure. Note that this is a trial-and-error method, and the critical values of P_a or P_p must be established for each trial wedge or failure surface. While this method is somewhat detailed, the circular wedge-type, trial-and-error solution is the only method available for determining the location of the critical failure surface of a complex problem. In lieu of a circular segment in the passive case, as shown in example (b) of figure 3-20, a log spiral may also be used. However, this makes the problem even more cumbersome and not much more accurate. As previously mentioned, the log spiral method is presented by Terzaghi and Peck, 1967.

3D.5 Surcharge Loadings by the Theory of Elasticity

Stress distributions resulting from point, line, and strip loadings may be developed by the use of the elastic theory, with modifications based upon experimental results. The charts and graphs presented in this guide have been adjusted to make the theoretical agree with the measured pressure.

3D.5.1 Point Loads

Example (a) in figure 3-21 shows a point load applied to a roadway and the generalized location of a point on the wall facing. Examples (b) and (c) in figure 3-21 show generalized curves of influence factors for the computation of the lateral stress distribution on a vertical face perpendicular to the line of the load application. Also shown are equations required to develop the influence factors.

The equations presented in figure 3-21 have been solved for various geometries and are summarized in examples (a) and (b) of figure 3-22. Example (c) in figure 3-22 shows the influencing factors required to compute the resultant, P_h, of a given point loading condition. Example (d) in figure 3-22 shows influencing factors for the determination of the location of the resultant, P_h, above the wall base.

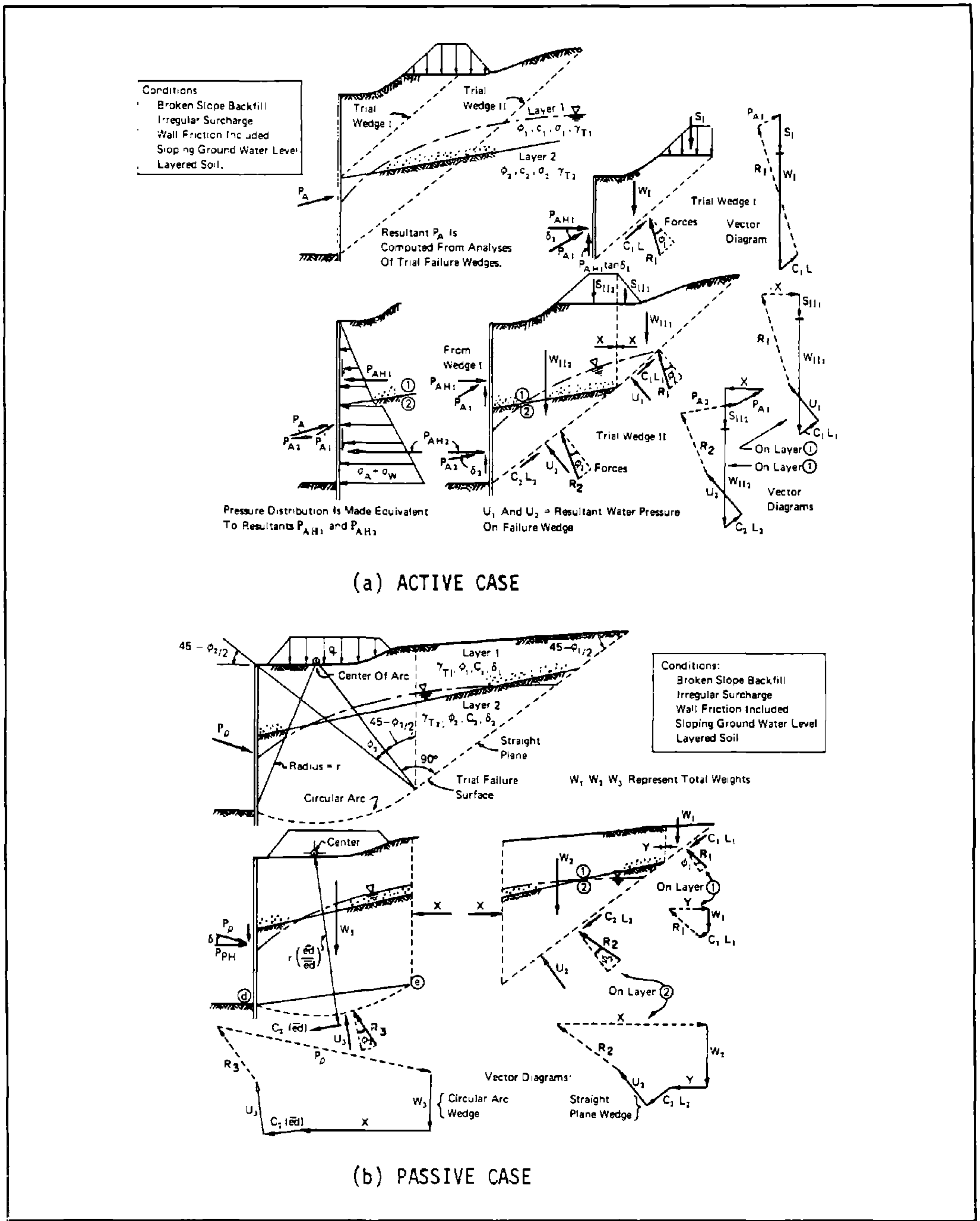


Figure 3-20.—Generalized determination of earth pressure (after NAVFAC Dm-7).

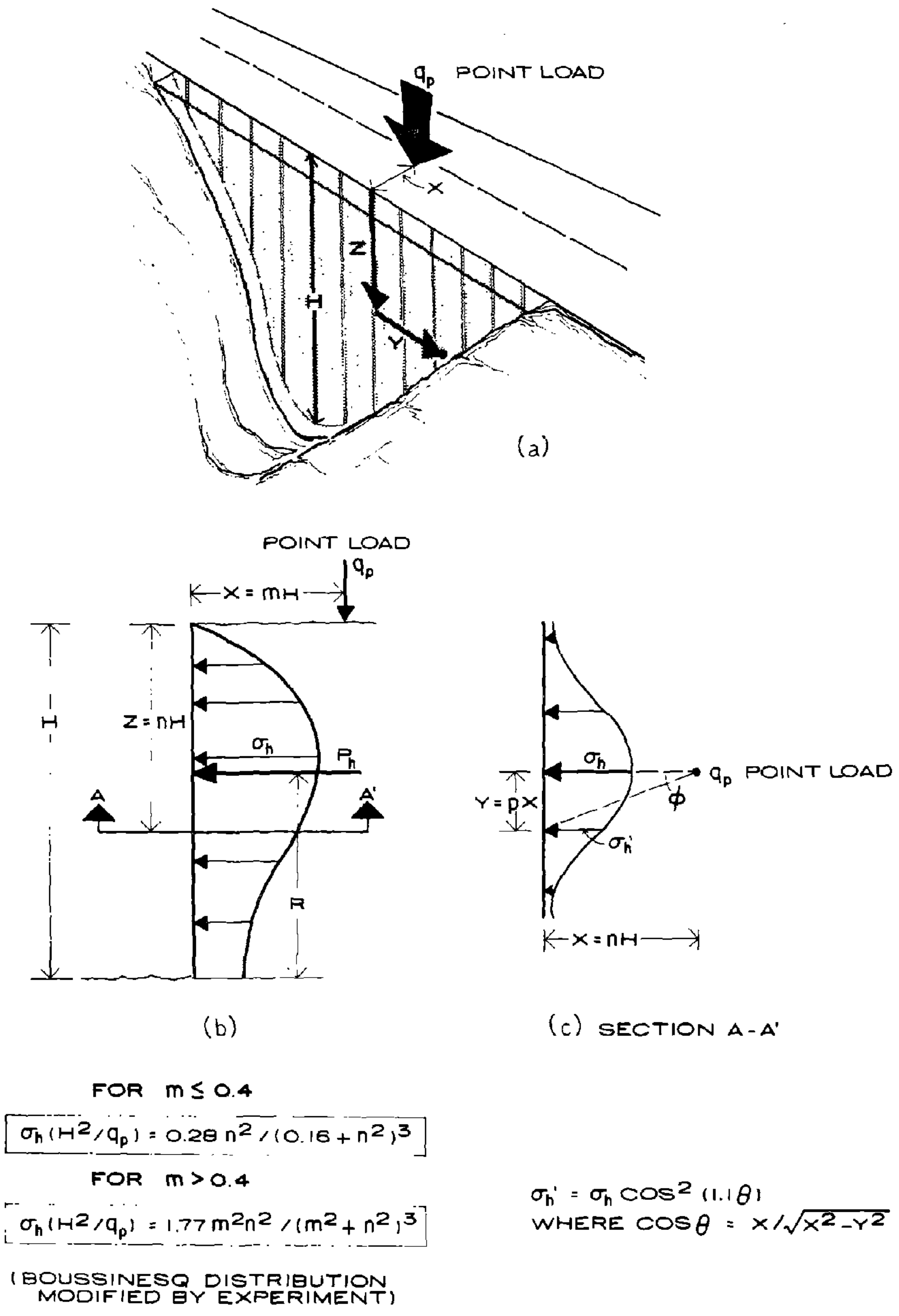


Figure 3-21.—Generalized conditions of point load surcharge.

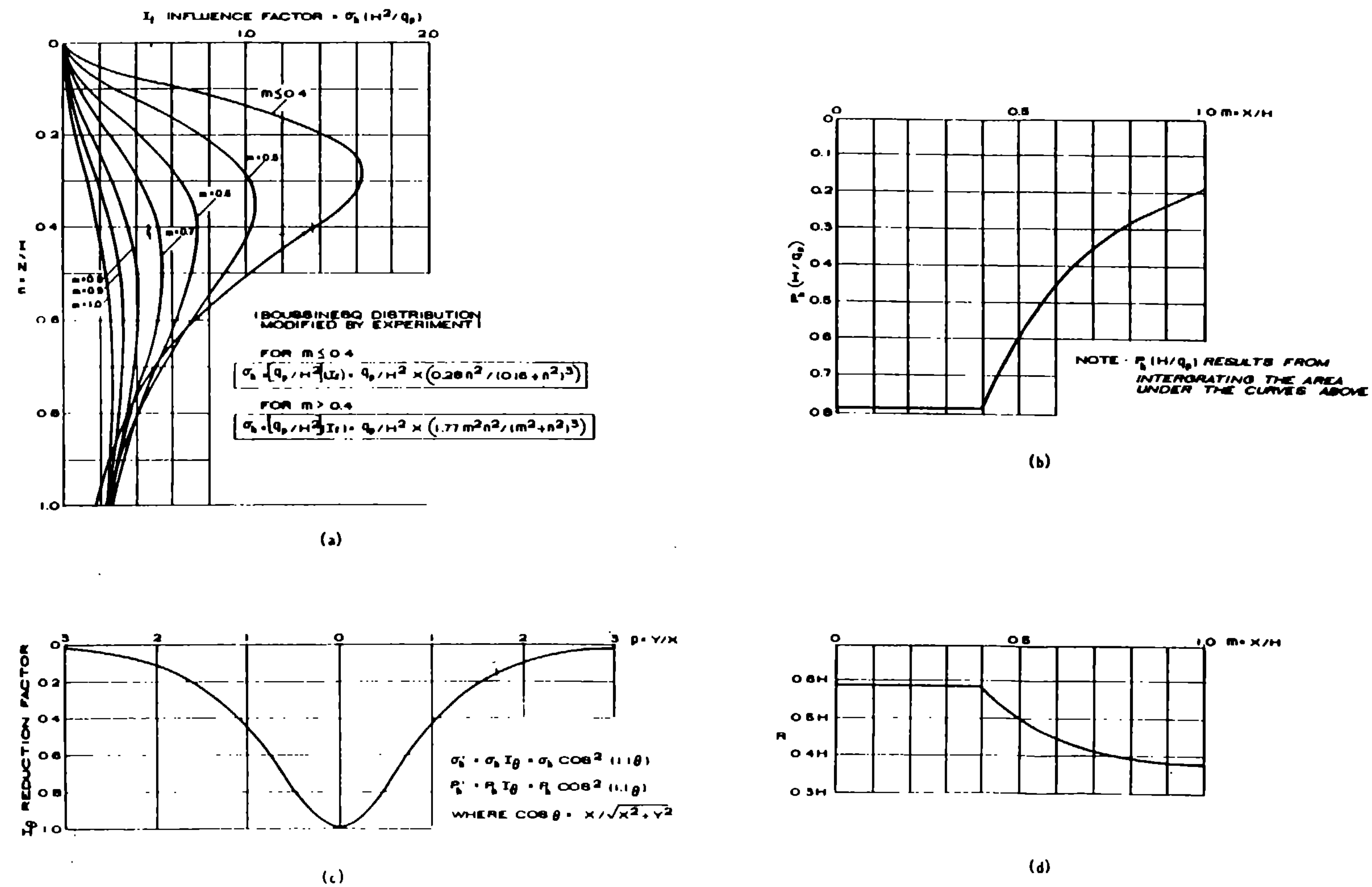


Figure 3-22.—Influencing factors for point loading.

The influence of groups of point loadings, such as wheel loads, may be evaluated with the graphs presented previously and the principle of superposition. In table 3-9, A and B show a summary of horizontal surcharge loadings that would commonly be imposed on retaining walls on low volume Forest Service roads. These values are derived from the values in figures 3-21 and 3-22. The values presented are in kips and represent the total load imposed on the wall from a tandem axle. For each combination of wall height and setback distance, two values of ΔP_h are presented. The value corresponding to "O" represents the increase in force directly opposite of an axle. The "C" value corresponds to the increase in force at the center of the two axles. For purposes of design, it is recommended that the resultant, ΔP_h, be assumed to act at a point 0.58 H above the base of the wall.

3D.5.2 Line Loads

Example (a) in figure 3-23 schematically shows a line load, q_l, applied behind a retaining wall and the resulting stress distribution. The modified Boussinesq equations are also presented. Example (b) in figure 3-23 shows a graphical summary of the solution of m and n. For a given line load, q_l, the resultant, P_h, and the location of the resultant, R, may be determined from examples (c) and (d) in figure 3-23, respectively.

3D.5.3 Strip Loads

The influence of a strip loading on a retaining wall may be evaluated with the geometric relationship and equations shown on figure 3-24. Calculation of the influence factor, I_f, can be accomplished by making a series of graphical constructions, as shown on figure 3-24, and physically measuring the required angles.

The resultant of the stress distribution can be estimated by assuming an area composed of simple geometric shapes or by integration by means of Simpson's rule. The point of application of the resultant is obtained by summing the moments about a fixed point.

3D.6 Seismic Analysis

3D.6.1 External Seismic Stability

The external seismic stability analysis is applicable to reinforced soil structures and other retaining structures. During an earthquake, the retained fill exerts a dynamic horizontal thrust, P_{AE}, on the retaining wall and static thrust. In addition, the retaining structure experiences an internal inertial force, $P_{IR} = Ma_m$, in which M is the mass of the wall section and a_m is the maximum horizontal acceleration.

Force, P_{AE}, can be evaluated by the pseudostatic Mononobe-Okabe analysis as shown in figure 3-25, and is added to the static forces (weight, surcharge, and static thrust) acting on the wall.

Table 3-9.—Typical point load surcharges.

A

Total horizontal load ΔP_h, kips on retaining wall due to 60 kip/axle, dual axle such as log yarders or other heavy equipment.

Height of retaining wall H (ft)	Position of point	Horizontal setback distance to retaining wall, from center of dual wheels X (ft) 2	5	10	15	20	25	30
10	0	3.51	2.84	1.01	-	-	-	-
	C	(2.54)	(3.10)	(1.05)	-	-	-	-
15	0	3.07	3.22	1.74	0.71	-	-	-
	C	(2.46)	(3.46)	(1.78)	(0.72)	-	-	-
20	0	2.85	2.85	2.29	0.90	0.61	-	-
	C	(2.42)	(3.05)	(2.35)	(1.29)	(0.61)	-	-
30	0	2.19	2.51	2.30	1.70	1.16	0.81	0.51
	C	(1.89)	(2.62)	(2.36)	(1.72)	(1.17)	(0.82)	(0.51)

B

Total horizontal load ΔP_h, kips on retaining wall due to 16.5 kips/axle (legal highway truck).

Height of retaining wall H (ft)	Position of point	Horizontal setback distance to retaining wall, from center of dual wheels X (ft) 2	5	10	15	20	25	30
10	0	0.97	0.78	0.28	-	-	-	-
	C	(0.70)	(0.85)	(0.29)	-	-	-	-
15	0	0.84	0.89	0.48	0.20	-	-	-
	C	(0.68)	(0.95)	(0.49)	(0.20)	-	-	-
20	0	0.78	0.78	0.63	0.25	0.17	-	-
	C	(0.67)	(0.84)	(0.65)	(0.35)	(0.17)	-	-
30	0	0.60	0.69	0.63	0.47	0.32	0.22	0.14
	C	(0.52)	(0.72)	(0.65)	(0.47)	(0.32)	(0.22)	(0.14)

Note: The resultants of different axle loads of similar geometries may be computed by simple proportion.

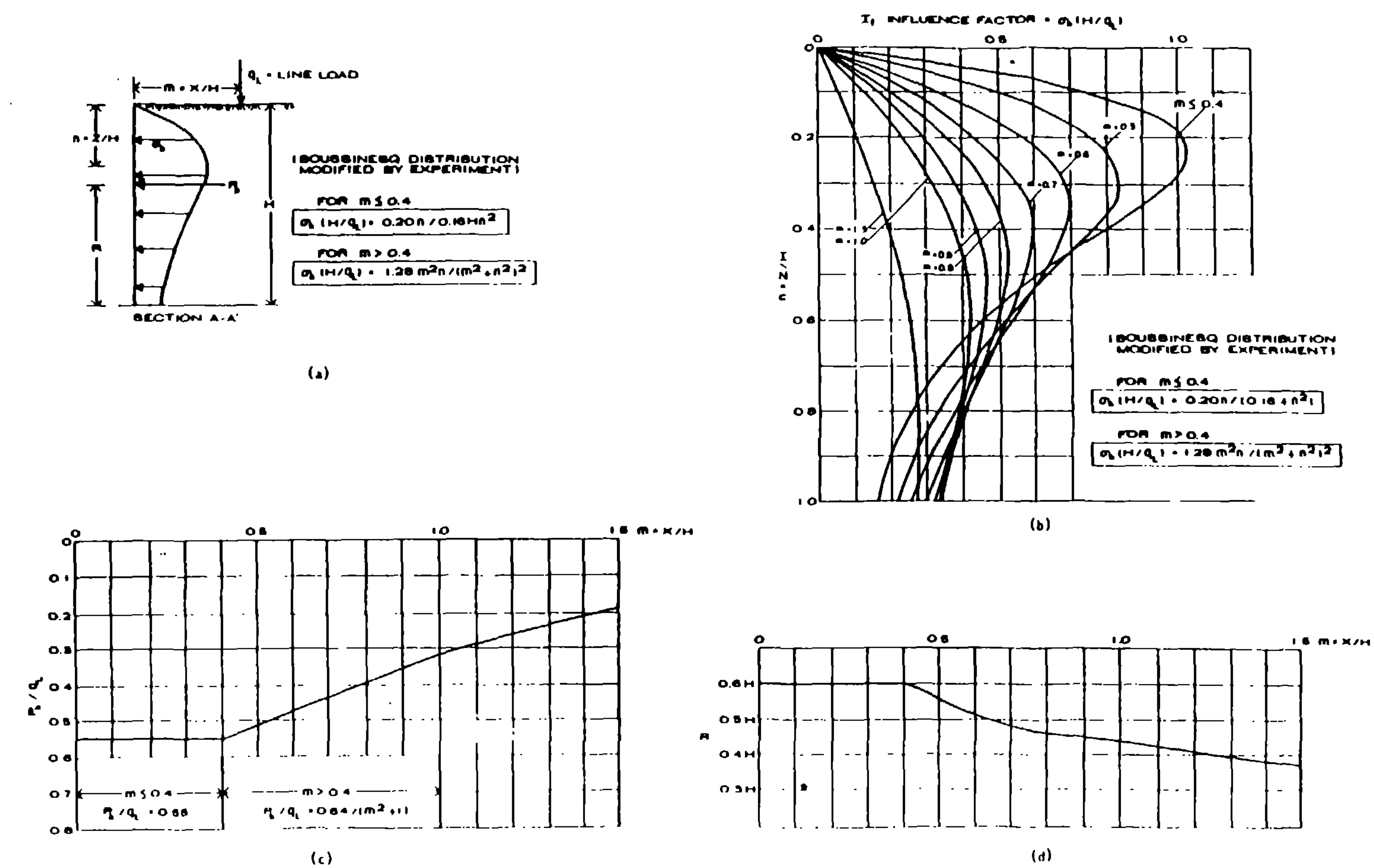


Figure 3-23.—Influence factors for line loading.

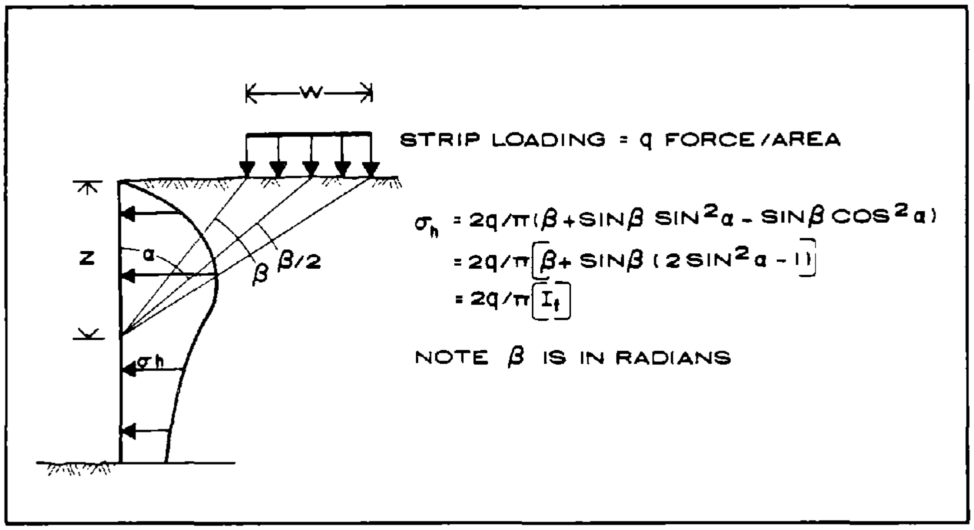


Figure 3-24.—Earth pressure from strip load.

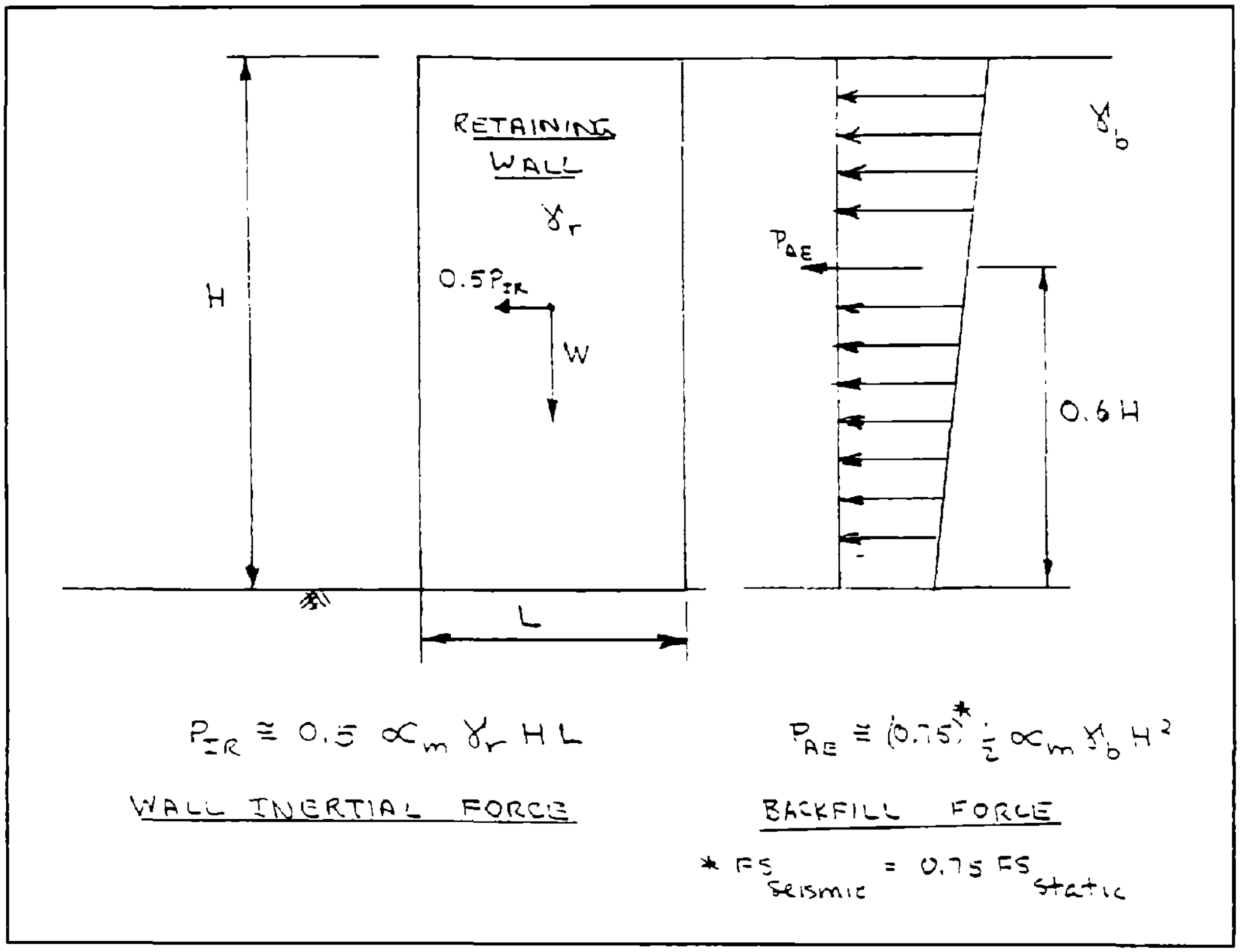


Figure 3-25.—Seismic external stability.

Procedure

(1) Select the peak horizontal ground acceleration, $\alpha_o g$, for the design earthquake from the *Map of Horizontal Acceleration in Rock* with 90 percent probability of nonexceedance contained in AASHTO standard specifications for seismic design of highway bridges (see figure 3-26). Peak acceleration is the contour number divided by 100. (Figure 3-25 illustrates this procedure.)

(2) Calculate the maximum acceleration, $a_m = \alpha_m g$, developed in the wall according to the formula:

$$\alpha_m = (1.45 - \alpha_o)\alpha_o \qquad (3\text{-}31)$$

in which α_o equals the maximum ground acceleration coefficient, and α_m equals the maximum wall acceleration coefficient at the centroid.

(3) Calculate the horizontal inertia force, P_{IR}, and the seismic thrust, P_{AE}, using

$$P_{IR} = \alpha_m \gamma_r HL \qquad (3\text{-}32)$$

and

$$P_{AE} = 0.375\alpha_m \gamma_b H^2 \qquad (3\text{-}33)$$

(4) Add to the static forces P_b and P_q acting on the structure, seismic thrust P_{AE} and 50 percent of P_{IR}, respectively. P_{IR} is reduced since it is unlikely that P_{AE} and P_{IR} will be in phase or peak simultaneously.

(5) Evaluate the sliding and overturning stability using the increased dynamic thrust in place of the static thrust.

(6) Check that the safety factors against sliding and overturning using the dynamic thrust are greater than or equal to 75 percent of the static safety factors.

Figure 3-27 provides an example problem.

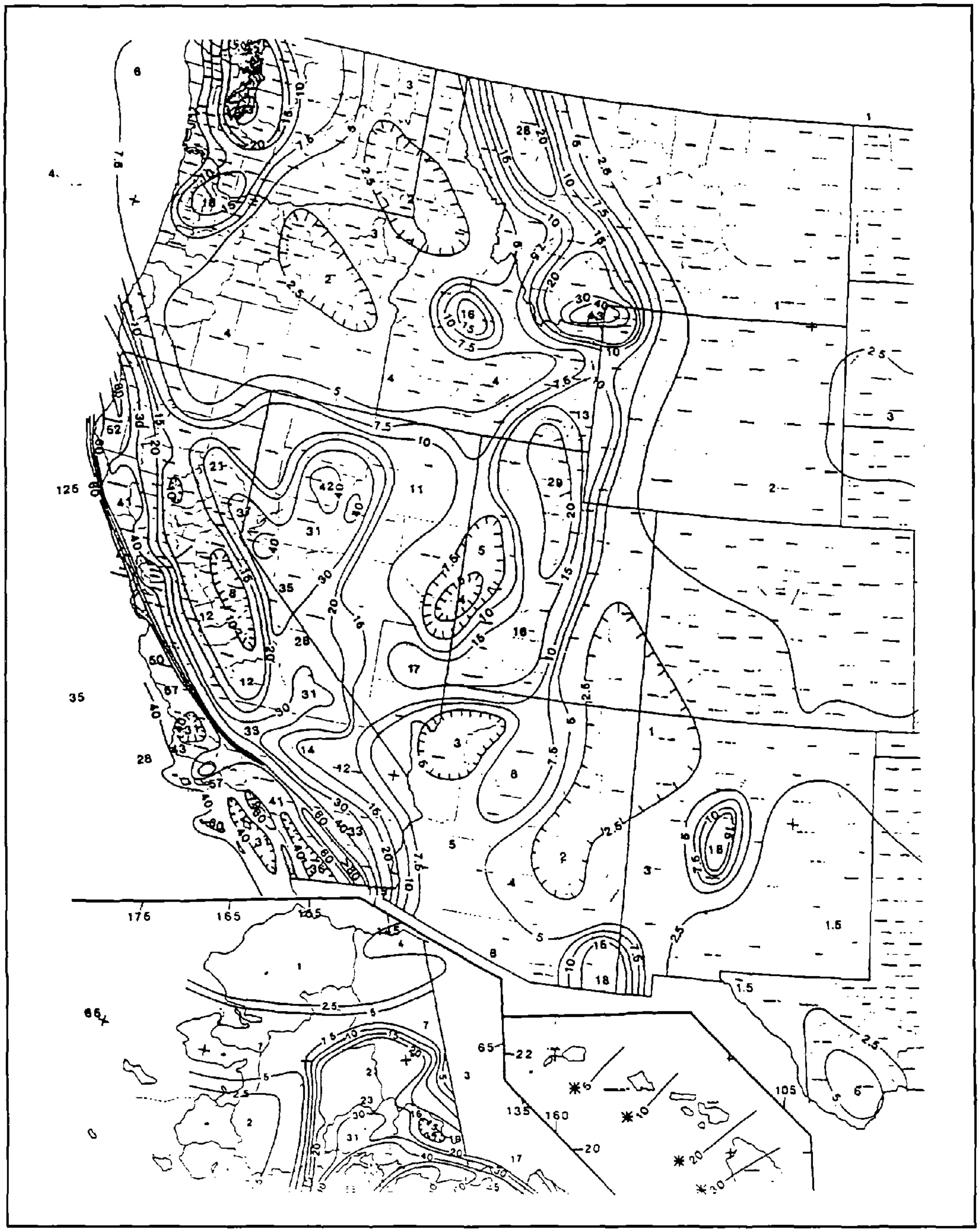


Figure 3-26.—Map of horizontal acceleration.

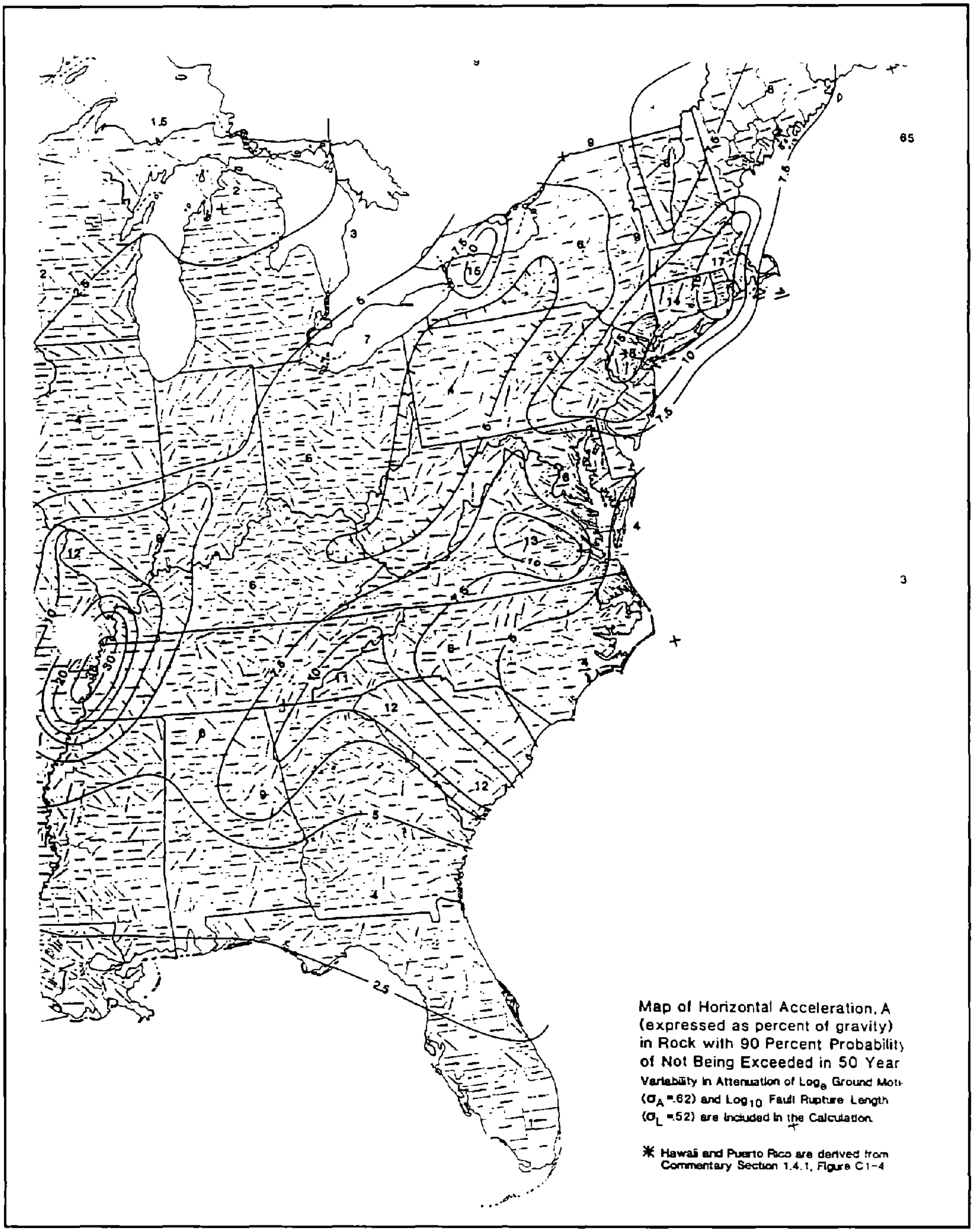


Figure 3-26.—Map of horizontal acceleration (cont'd.).

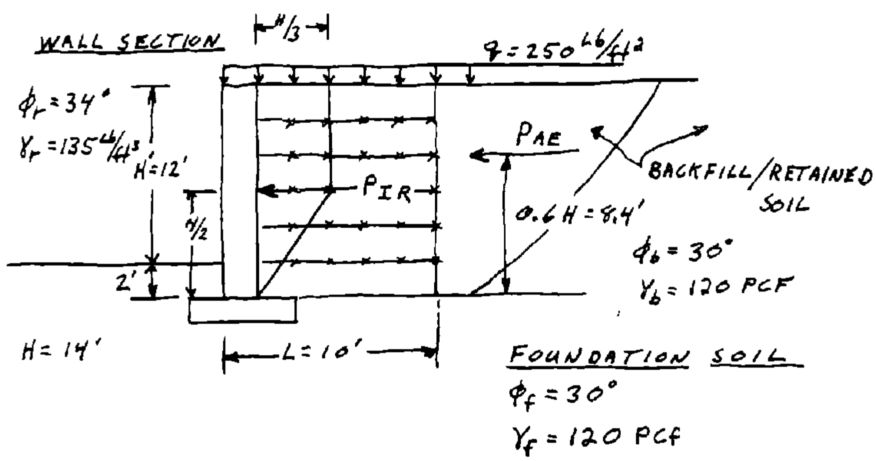
SEISMIC EXAMPLE—Refer to example problem on page 318.

EXTERNAL STABILITY

Applicable to all retaining structures.

Assume α_O= 0.2 from AASHTO standard specifications for seismic design of highway bridges.

1. $\therefore$ Maximum acceleration $\alpha_M = (1.45 - \alpha_O)\,\alpha_O = 0.25g$ at centroid of wall.



2. Calculate horizontal inertia force P_{IR} and seismic thrust P_{AE}.

$P_{IR} = \alpha_m \gamma_r HL$

$= (0.25)(135)^*(14)10$ *Assume facing γ same as reinforced soil

$P_{IR} = 473$ k/ft

$P_{AE} = 1/2 \alpha m \gamma_b H^2$

$= (1/2)(0.25)(120)(14)^2$

$\underline{P_{AE}} = \underline{2.95 \text{ k/ft}}$

Figure 3-27.—Seismic example.

Adding P_{IR} and P_{AE} to the static equations for sliding and overturning:

For sliding from step 4.1.

$$L_{Min} = \frac{(75\%)\ [FS_{SL}\ \overbrace{[(K_{Ab})\ H\ [q + (\gamma_b\ \tfrac{H}{2})]}^{STATIC} + \overbrace{50\%\ P_{IR} + P_{AE}]}^{SEISMIC}}{(\gamma_B\ H + q)\ \tan\ \phi_f}$$

$$L_{Min} = \frac{(0.75)\ (15)\ [(0.33)\ (14)\ [250 + (0.120)\ (14/2)] + (.5)\ (4.73) + 2.95]}{1.24}$$

$$L_{Min} = \frac{1.125\ [5.04\ \text{k/ft} + 2.37\ \text{k/ft} + 2.95\ \text{k/ft}]}{1.24}$$

$$L_{Min_{Seismic}} = 9.4\ \text{ft} > 7.0\ \text{ft static}$$

For overturning step 4.2.

$$L_{Min_{Seismic}} = \sqrt{\frac{(0.75)FS_{OT}[H^2K_{ab}(\gamma_b H+3q)+50\%P_{IR}\frac{H}{2}+P_{AE}0.6H]}{3[\gamma_r H+q]}}$$

$$= \sqrt{\frac{(0.75)(2)[(14)^2(.33)(1.12)(14)+(.75)+(.5)(4.73)(7)+(2.95)(.6)(14)}{3[(1.35)(14)+.25]}}$$

$$L_{Min_{Seismic\ OT}} = 6.8\ \text{ft} < L_{Min_{Seismic\ Sliding}}$$

Figure 3-27.—Seismic example (cont'd.).

3D.6.2 Internal Seismic Stability

Internal seismic stability is primarily applicable to reinforced soil retaining walls. The internal reinforcement must be capable of withstanding the inertial force P_{IR} acting horizontally on the active zone and internal static forces. The inertial force will increase the maximum tensile forces in the reinforcement and the total pullout force on the reinforcement. It is assumed that the location and slope of the maximum tensile force line does not change during the seismic loading. (See figure 3-28 for an illustration of the procedure.)

Procedure

(1) Calculate the maximum acceleration, $\alpha_m g$, in the wall and the force P_{IA} acting on the reinforced soil mass above layer z using

$$P_{IA} = \alpha_m W_A \qquad (3\text{-}34)$$

and

$$\alpha_m = (1.45\text{-}\alpha_o)\alpha_o \qquad (3\text{-}31)$$

in which W is the weight of the active zone above level z.

(2) Calculate the total horizontal static stress in the reinforced soil and the maximum static tensile force as follows:

Calculate the horizontal stress, σ_h, according to step 7.3 of the welded wire example problem in chapter 4. Calculate the total horizontal static load, P_H, for each layer *in each reinforcement.*

(3) Calculate the incremental force in each layer due to seismic loading and distribute the seismic force in proportion to the "resistant areas."

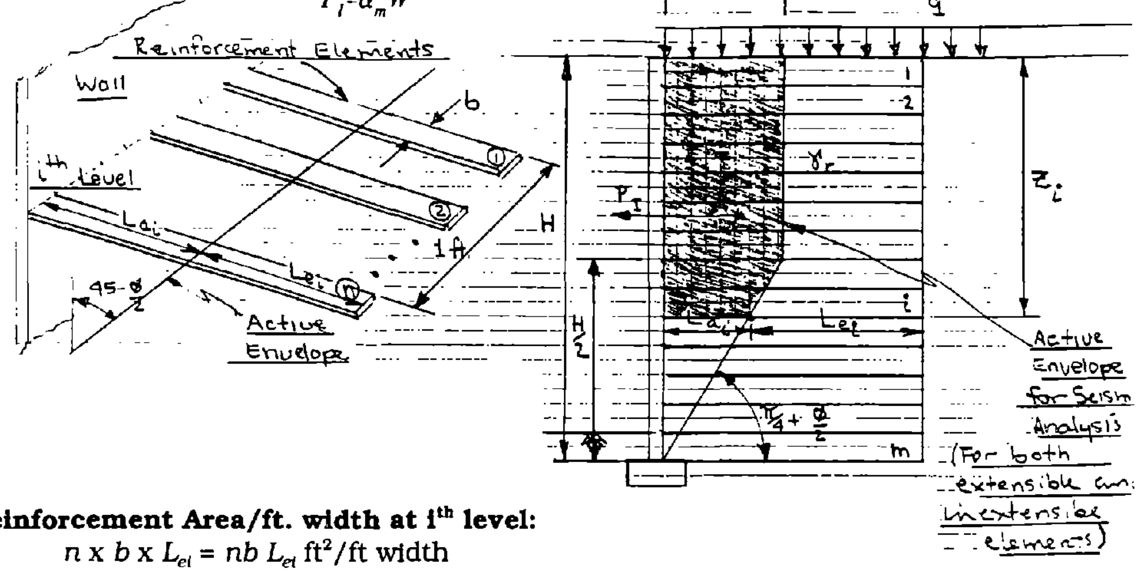
(4) Compute the total maximum tensile force in each element for breakage and pullout, and verify that the seismic factors of safety equals 0.75 times the static factors of safety.

The recommended design method for seismic loading was developed for inextensible reinforcements. The extensibility of the reinforcements affects the overall stiffness of the reinforced soil mass. As extensible reinforcement reduces the overall stiffness, it is expected to have an influence on the design diagram of the lateral earth pressure induced by the siesmic loading. As the overall stiffness decreases, damping and amplification should increase. Thus the resulting inertia force may not be much different for inextensible reinforcement. In addition, since there is a substantial factor of safety in the design tension for potential creep of extensible reinforcement under long-term static loads, an additional factor of safety against a dynamic overload is provided. Therefore, the inextensible reinforcement analysis should be safe for extensible reinforcement.

Dynamic Force Increment, P_I:

Maximum acceleration in wall: $\alpha_m = \dfrac{(1.45 - \alpha)\alpha}{0.3\,H}$

$P_I = \alpha_m W$



Reinforcement Area/ft. width at i^{th} level:

$n \times b \times L_{ei} = nb\,L_{ei}$ ft²/ft width

$\therefore$ Total reinforcement/ft width $= \sum_{j}^{i}$
(down to the i^{th} level)

Procedure:

Static
1. Determine σ_v
2. $\sigma_y = K\sigma_v = K(\gamma_r\,Z+q)$
3. $P_{H_{1_i}} = \dfrac{K(\gamma Z_i + q)\,S_Z}{n}$ (Static Force/Reinforcement Element)

Dynamic
4. Determine Dynamic Increment

$$P_{H_{Z_i}} = P_I \frac{nb\,L_{e_i}}{\sum_{j=1}^{m} nb\,L_{e_j}} = \frac{nb\,P_I\,L_{e_i}}{nb\sum_{j=1}^{m} L_{e_j}} = \frac{P_I\,L_{e_i}}{\sum_{j=1}^{m} L_{e_j}}$$

$$P_{h_{T_i}} = P_{h_{1_i}} + P_{H_{Z_i}} = \frac{K(\gamma_r Z_i + q)\,S_Z}{n} + P_I \frac{L_{e_i}}{\sum_{j=1}^{m} L_{e_j}}$$

Figure 3-28.—Extensible reinforcement.

Internal Stability - Example - Ref: Chapter 4, Example 1, Welded Wire Wall

(Applicable to internally reinforced earth retaining structures)
(e.g., Hilfiker welded wire walls)

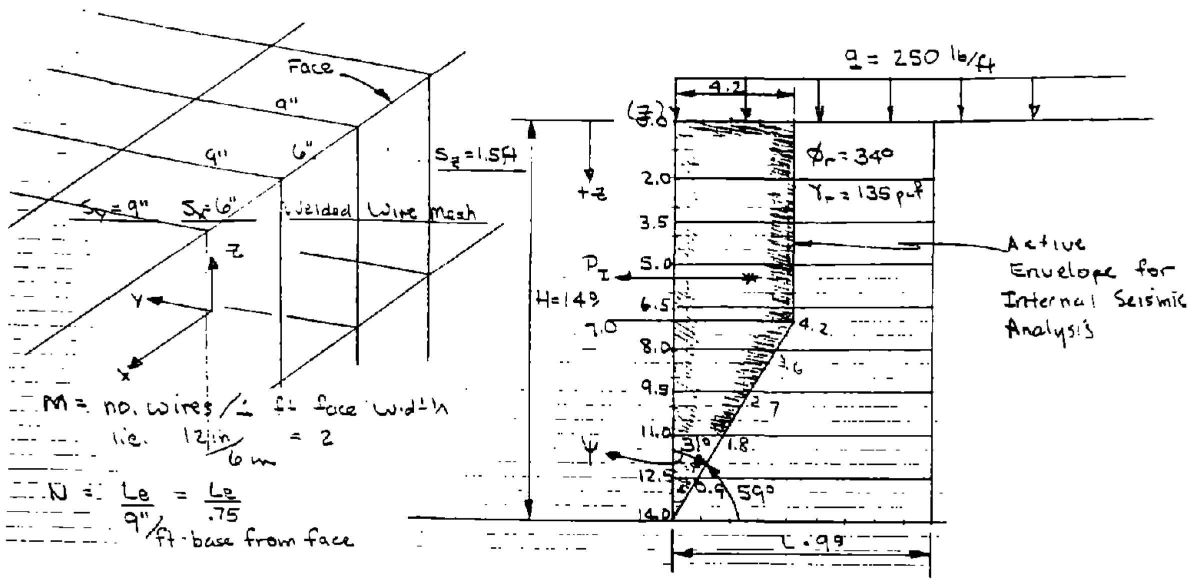


Figure 3-28.—Extensible reinforcement (cont'd.).

$$FS_p = \frac{P_t}{MP_H} \qquad FS_R = \frac{31{,}200}{F_T}$$

Static Condition

Layer No.	Depth Z (ft)	σ_Y (psf)	P_{HI} (lbs/wire)	Wire Size	F_T (psi)	M	Le ≥ 3 (ft)	N $N = \frac{L_e}{0.75}$	P_t (lbs/ft)	FS_p > 1.5	FS_R > 1.0
1	2.0	338.0	253.5	W3.5	7,243	2	4.8	6.4	1,801	3.6	4.3
2	3.5	496.6	352.2	W3.5	10,063	2	4.8	6.4	2,678	3.8	3.1
3	5.0	601.3	450.9	W3.5	12,884	2	4.8	6.4	3,554	3.9	2.4
4	6.5	732.9	549.7	W3.5	15,705	2	4.8	6.4	4,430	4.0	2.0
5	8.0	864.5	648.4	W3.5	18,525	2	5.4	7.2	5,890	4.5	1.7
6	9.5	996.1	747.1	W3.5	21,346	2	6.3	8.4	7,917	5.3	1.5
7	11.0	1128	845.8	W3.5	24,166	2	7.2	9.6	10,272	6.1	1.3
8	12.5	1259	944.5	W3.5	26,987	2	8.1	10.8	12,955	6.9	1.2
9	14.0	1391	1043	W3.5	29,807	2	9.0	12.0	15,967	7.6	1.1

$$\sigma_y = K\,(\gamma_r Z + q) = 0.65\,(135Z + 250) = \underline{87.75z + 162.5}$$

$$P_H = \sigma_y\, S_x\, S_z = \sigma_y \left[\frac{6\text{ in}}{12\text{ in/ft}}\right] 1.5\text{ft} = \underline{0.75\text{ft}^2\sigma_y}$$

$$F_T = \frac{P_H}{A_{WIRE}} = \frac{P_H}{.035\text{ in}^2} \qquad W3.5 \Rightarrow 0.035\text{ in}^2\text{ wire} = \frac{\pi d^2}{4}\ or\ d = 0.211\text{ in}$$

$$= 0.0176$$

For z ≥ 7: $L_e = 9 - (14 - z)\tan\Psi \quad \tan\Psi = 0.60$

$L_e = 9 - (14 - z)\,0.60 = 9 - 8.4 + 0.60\,z = \underline{0.6\,(1 + z)}$

For washed sand:

$P_t = 633\text{ lb/ft} + \gamma_r Zd\,[0.75\pi\, L_e M\,(2/3\tan\phi_r) + 36.8\,N]$

$P_t = 633 + 2.376\,Z\,[2.12\,L_e + 49.1\,L_e]$

$\underline{P_t = 633 + 121.7\,Z\,L_e}$ lb/ft

Figure 3-28.—Extensible reinforcement (cont'd.).

$$FS_p = \frac{P_t}{MP_H} \qquad FS_R = \frac{31{,}200}{F_T}$$

Static Condition

Layer No.	Depth Z (ft)	P_{H1_i} (lbs/ wire)	A $(ft)^2$	W (lb)	P_I (lbs)	L_{ei} (ft)	ΣL_e (ft)	P_{H2_i} (lb)	P_{HT_i} (lb)	FS_{pi} > .75(1.5)	FS_{Ri} > .75(1.0)
1	2.0	253.5				4.8		64.8	318.3	2.8	3.4
2	3.5	352.2				4.8		64.8	417.0	3.2	2.6
3	5.0	450.9				4.8		64.8	515.7	3.4	2.1
4	6.5	549.7				4.8		64.8	614.5	3.6	1.8
5	8.0	648.4				5.4		72.9	721.3	4.1	1.5
6	9.5	747.1				6.3		85.1	832.2	4.8	1.3
7	11.0	845.8				7.2		97.2	943.0	5.4	1.2
8	12.5	944.5				8.1		109.4	1,053.9	6.1	1.0
9	14.0	1043.0	44.1	2,977	744	9.0	55.2	121.5	1,164.5	6.9	0.9

A W: Mass in active envelope:

$$W = S_x \times \overbrace{\text{Area Active Envelope}}^{A} \times \gamma_r$$
$$= (0.5)\,(135)\,A$$
$$\underline{W = 67.5\,A}$$

B Inertial force, P_I:

$$P_I = W\,\alpha_m \qquad \alpha_m = 0.25g$$
$$P_I = (2977)\ .25$$
$$\underline{P_I = 744\ \text{lbs}}$$

C $$P_{H2_i} = P_I \frac{L_{e_i}}{\sum_{j=1}^{m} L_{e_j}} = \frac{744 L_{e_i}}{55.2} \qquad \underline{13.5 L_{e_i}}$$

D $$FS_{p_i} = \frac{P_{t_i}}{M_{P_{HT_i}}} = \frac{P_{t_i}}{2P_{HT_i}}$$

$$FS_{R_i} = \frac{31{,}200}{P_{HT_i}/.035\ \text{in}^2} = \frac{1{,}092}{P_{HT_i}}$$

Figure 3-28.—Extensible reinforcement (cont'd.).

3E. Backfill Considerations

3E.1 Materials

Relatively clean, free-draining, granular soils are generally recommended by engineers and preferred by contractors as backfill for retaining structures. However, under most circumstances, any nonplastic to low plasticity frictional soil or rock type may be utilized as backfill, provided that the wall is designed to resist the imposed forces. It is generally more economical to use local native or fine-grained backfill and design for those appropriate strength parameters than to import free-draining materials. As discussed further at the end of this section, when fine-grained or marginal material is placed as backfill, care should be taken to closely control placement moisture content and density to provide a well-designed drainage system, and to consider the likelihood of accelerated corrosion.

Backfill material should meet the requirements shown in figure 3-65.

3E.2 Placement and Compaction

The earth pressure acting on a retaining structure is directly related to the unit density (degree of compaction) and the method of the backfill placement. Typically, the backfill for an un-reinforced retaining structure should be placed in horizontal lifts and machine-compacted to *92 to 93 percent of the maximum dry density as determined by AASHTO T-99.* Experience has shown that this approach to backfill construction results in an acceptable balance of developed friction angle, unit density, and desired wall performance for conventional retaining walls (Broms, 1971).

Backfill densities in excess of 93 percent of AASHTO T-99 generally result in earth pressures that approach the K_0, or the at-rest state of stress. With rigid, nonyielding structures, excessive compaction can result in pressure approaching the passive state of stress. Such overstressing can result in undesirable deflections of the wall and structural damage. With extensible backfill reinforcement, such as geotextiles and some geogrids, the active earth pressure state, K_a, is appropriate.

Reinforced backfill structures generally require a minimum of 95 percent of the maximum dry density, as determined by AASHTO T-99, in order to mobilize adequate frictional properties along the reinforcement strips (FHWA-RD-89-043). For backfill materials containing more than 30 percent retained on the 3/4-inch sieve, a required method of compaction consisting of at least four passes with a heavy roller should be used.

The placement water content of moisture-sensitive soils should be maintained within 2 percent of the optimum moisture content as determined by the designated compaction specification. If the settlement or stability of the backfill is a concern, the moisture content should be held on the dry side of optimum. Generally clean (less than 3 percent passing the no. 200 sieve), coarse-grained, cohesionless soils may be placed independently of moisture content, and, hence, they are recommended for wintertime construction.

Compaction equipment selection should depend on soil type. Segmented foot, sheepsfoot, or pneumatic-tired rollers prove most effective for cohesive soils (including silt), and smooth drum vibratory rollers are most effective for cohesionless soils and rock aggregations. Likewise, hand-operated, vibratory plate compactors and small, smooth-drum rollers are better suited for cohesionless soils, while Wacker-type rammers are better suited for cohesive soils (including silts). Use of vibratory compactors on fine-grained soils with optimum moisture will frequently cause water to rise to the ground surface, causing a pumping condition.

The allowable lift thickness of the uncompacted fill depends on the material type and the available compaction equipment. In reinforced backfill structures, the practical lift thickness may be additionally controlled by the specified spacing of the reinforcement layers. Also, the lift thickness may be dictated by the height of the facing elements or some multiple of that height. For hand-operated compaction equipment, the allowable uncompacted lift thickness should generally be in the range of 6 to 8 inches. For medium to heavyweight machine-driven compaction equipment, the maximum uncompacted lift thickness is generally about 9 inches for fine-grained cohesive soils (including silt), and 12 inches for cohesionless soils and crushed rock.

Compaction within 3 feet of an earth-reinforced wall back face should be achieved by at least three passes of a lightweight mechanical tamper, roller, or vibratory system. With fine-grained backfill materials, a zone of coarse, granular material such as pea gravel may be placed immediately behind the face to facilitate compaction in this area.

3E.3 Marginal Backfill Materials

Often of marginal quality, onsite or local materials are consistently used by the Forest Service as backfill in retaining structures and reinforced embankments because of the unavailability or expense of imported materials. "Marginal" soils are defined as fine-grained, low plasticity materials which are often difficult to compact, and have poor drainage or strength parameters sensitive to density. Coarse rockfill material, occasionally available, can be suitable for backfill if well-graded, but it often has enough oversize material to make layer placement difficult and rocks can damage the reinforcement material.

Local "marginal" materials already used have varied from silty sands (SM, SC) to silts and clays with over 50 percent fines (passing the no. 200 sieve ML, CL). Marginal materials should be specifically tested for their strength and density relationships, and tested for peak and residual shear strength parameters. Strength parameters should be determined using direct shear or consolidated-drained triaxial tests.

For granular and low plasticity soils where strength is given by granular friction, peak strength parameters should be used. However, if a significant portion of the strength comes from cohesion of a soil, then residual strength parameters should be used. In any case, one should be careful using higher plasticity clays as backfill.

The use of marginal backfill has been acceptable, but it can present problems in construction and long-term performance. Compaction of fine-grained soils is sensitive to moisture, so close construction control is needed to insure that specified densities are achieved. Occasionally the specified density of 95 percent of the AASHTO T-99 maximum density has been difficult to achieve; however, once achieved, results have been overall satisfactory. A reduced density may lead to additional long-term settlement.

Inability to achieve the specified compaction near the wall face, or loss of fines through the face have resulted in some face settlement. Unless the settlement is severe, however, it is more an issue of aesthetics and typically does not present a structural problem for a reinforced structure.

Marginal backfill materials with poor drainage characteristics are difficult to construct under wet conditions, and they will develop hydrostatic pressures in the backfill unless the material is well-drained or encapsulated. Surface drainage should be designed to keep water from infiltrating into the backfill.

Table 3-10 shows examples of local, somewhat marginal soils that have been successfully used in Forest Service structures. Note that the soils, though fine-grained, have good frictional characteristics.

Table 3-10.—Local "marginal" soils used in structures.

Site	Wall Type	USC	Percent minus 200	PI	Phi′ deg	c′ psf	Comments
Goat Hill Plumas, NF	Welded wire	SM SC	21 20	5 8	34 31	200 300	Some face settlement
L. North Fork Plumas, NF	Reinforced fill 1:1	SM ML	38 55	2 3	34 33	100 150	Slight slope ravel
B. Longville Plumas, NF	Welded wire	CL	50+	—	26	200	Poor foundation
Grave Plumas, NF	Geotextile	SM	26	NP	35	850	Irregular face
Butt Valley Plumas, NF	Tire-faced	SC	38	8	26	400	8% face settlement
Klamath, NF	Timber-faced	SM	27	NP	30	0	Minimal settlement
Willamette, NF	Wood chips and geotextile	GP	0	NP	32	0	Some settlement

Note: Phi′ and c′ are from consolidated-drained tests at 95 percent of T-99 density.

3E.4 Poor, Clay-Rich Backfill Material

Generally, poor quality, clay-rich cohesive soils should not be used in retaining structures, except in special circumstances of low risk, substantial cost savings, and under conditions of careful design, construction control, and monitoring. However, cohesive soils have been used in reinforced structures and embankments worldwide with moderate success and significant cost savings.

Zornberg and Mitchell (1992) have reviewed the use of poorly drained backfill materials in reinforced soil structures. The use of cohesive backfill materials presents design and construction difficulties, such as making drainage and compaction difficult to achieve. Under the circumstances, deformation must be expected and acceptable. The material must be compacted under relatively dry conditions and should be on the dry side of optimum. Large surface deformation in plastic embankment materials suggests that the reinforcement should be longer than that used for conventional materials. Some pore pressure build up may oc- cur, reducing the frictional resistance of the backfill. Also, poorly drained soils can cause significantly accelerated corrosion rates in materials.

Walls with "clayey" soils for backfill can be designed, but there are unknown long-term creep, deformation, and strength and stresses on the system. Walls should not be vertical; they should be constructed with a batter or a stepped, flexible face to accommodate the expected deformation. Expansive clays should be avoided, unless considering adding lime to improve the soil properties. Forces on face connections will be relatively large.

The use of thick nonwoven geotextiles for reinforcement may be valuable to act as a "wick" drain throughout the backfill to dissipate pore pressure buildup and increase shear strength and pullout resistance. It may also speed up construction by reducing excess pore pressure induced during compaction.

3F Drainage

Unless a retaining structure is positively and permanently drained, it must be designed to resist a hydrostatic head equivalent to the effective height of the wall. Most retaining structures have drainage provisions included in construction that either removes local ground water or insures that the backfill will remain in a drained condition, as assumed in a conventional design. It is generally far more economical to provide a positive and complete drainage scheme for the backfill than to design for saturated conditions. *Drainage should be an integral part of a retaining wall design solution*, and it should be considered "cheap insurance" for a wall to perform its best.

Retaining structure drainage is typically achieved using gravel chimney drains or geocomposite drains installed behind the wall or backfill. However, layers of drainage material can be incorporated into the backfill material, or the backfill material can free-drain itself. Drainage of retaining structures can be achieved by at least four basic methods (and their variations), as presented in the order of effectiveness, not cost:

(1) Free-draining backfill material

(2) Sloping or near-horizontal drains

(3) Behind-the-face or backfill vertical drains

(4) Weep holes with gravel pockets behind the wall

For these methods it is assumed that the water is collected behind the wall or backfill and passed through the wall face or around the structure, usually in a pipe. To additionally achieve good drainage, the surface of the backfill material should be shaped to drain towards ditches or gutters, off of the backfill. Occasionally, an impervious seal, such as a layer of asphalt concrete, can be used to cap the backfill and prevent or reduce infiltration of surface moisture.

The following sections present analysis methods and criteria or requirements for drainage of various ground water conditions found in retaining wall applications.

3F.1 Perched Water Table

Figure 3-29 shows a wall with a perched water table and the location and orientation of the active and hydrostatic forces acting on the wall.

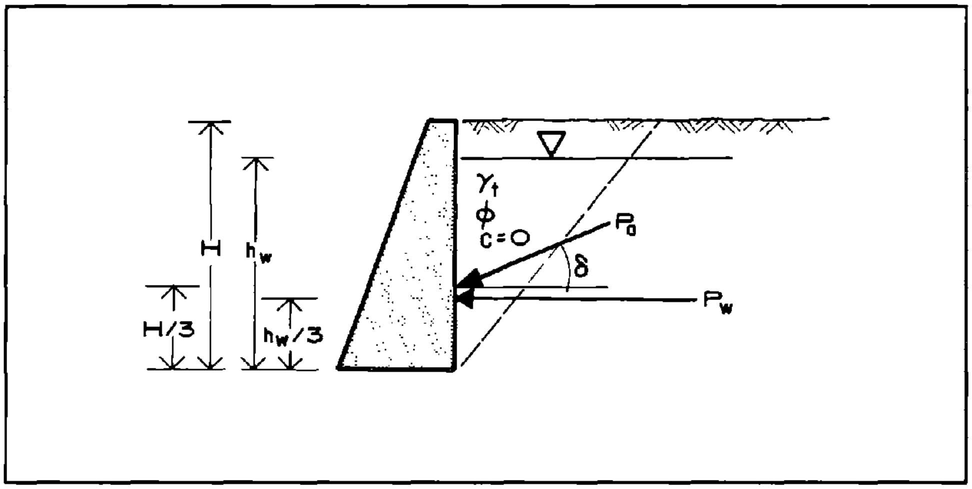


Figure 3-29.—Perched water table.

The active thrust, P_a, may be calculated using

$$P_a = 1/2\ \gamma_t\,(H - h_w)^2 K_a + 1/2\gamma_b h_w^2 K_a \tag{3-35}$$

in which K_a is obtained from equation 3-26. The hydrostatic pressure is computed using

$$P_w = 1/2\gamma_w h_w^2 \tag{3-36}$$

If the ground water level is coincident with the ground surface, $H = h_w$, the total horizontal thrust, P_h, is

$$P_h = P_a \cos\delta + P_w \tag{3-37}$$

and acts at a point $H/3$ above the base of the wall. P_a and P_w are obtained from equations 3-35 and 3-36, respectively.

3F.2 Free-Draining Backfill

There is no hydrostatic pressure in a free-draining backfill; hence, the lateral pressure acting on the retaining structure is simply the lateral earth pressure as computed by any method applicable for dry soil. However, theoretically, the unit weight of the soil should be increased to compensate for the increase in the amount of water (degree of saturation) in the soil.

Backfill permeability, and thus the ability to be free-draining, is very sensitive to the percentage of fine substances in that material. A free-draining material should have no more than 2 to 3 percent material passing the no. 200 sieve, and preferably no fine substances. On significant structures, the gradation or permeability of the free-draining material should be confirmed by laboratory testing.

In practice, the use of "free-draining" material is usually too costly. Instead, either a good quality local backfill or backfill meeting the requirements of FHWA FP-92 specifications is used (figure 3-67). To prevent the buildup of hydrostatic and seepage pressures, a drainage system such as the one shown in figure 3-32 is provided.

3F.3 Sloping Underdrain

Sloping or relatively flat underdrains or blanket drains are occasionally used, and they are an effective method of controlling ground water in the backfill of a retaining structure. Figure 3-30 summarizes the important aspects of a sloping drain.

The flow net for steady rainfall (see example (b) in figure 3-30) shows that, since the pressure head in the drain is zero, the hydrostatic pressure within the backfill is also zero. Hence, as in the free-draining case, the pressure on the wall may be calculated with any method applicable for a dry soil. Likewise, the unit weight of the soil must be increased to compensate for a degree of saturation equal to 100 percent.

3F.4 Vertical Drain

Vertical drains are another commonly used, easily constructed, and reasonably effective method of draining the backfill for a retaining structure. Figure 3-31 shows a retaining wall with a vertical drain and the flow net resulting from steady rainfall on the surface of the backslope.

The influence on the wall of the water seeping through the backfill must be evaluated with the aid of the flow net. Trial-and-error solutions must be completed for various orientations of the failure plane. A sample calculation of the total pore water force, U, is shown on figure 3-32 for $\rho = 45°$ (rho is the angle of the assumed failure plane behind the wall).

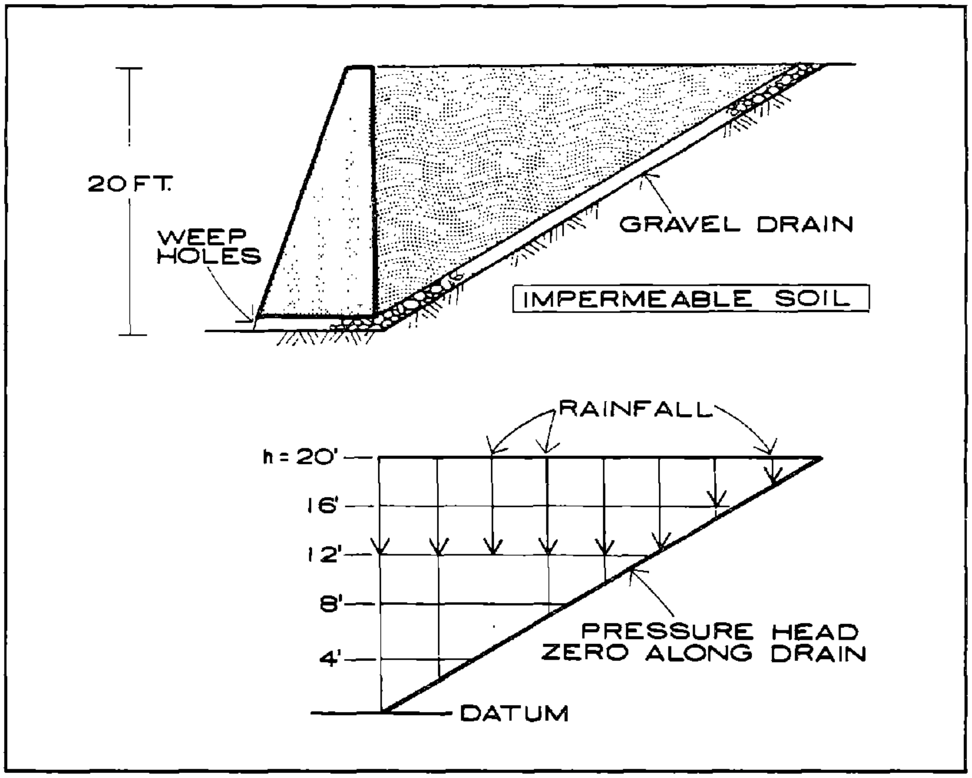


Figure 3-30.—Backfill with sloping underdrain.

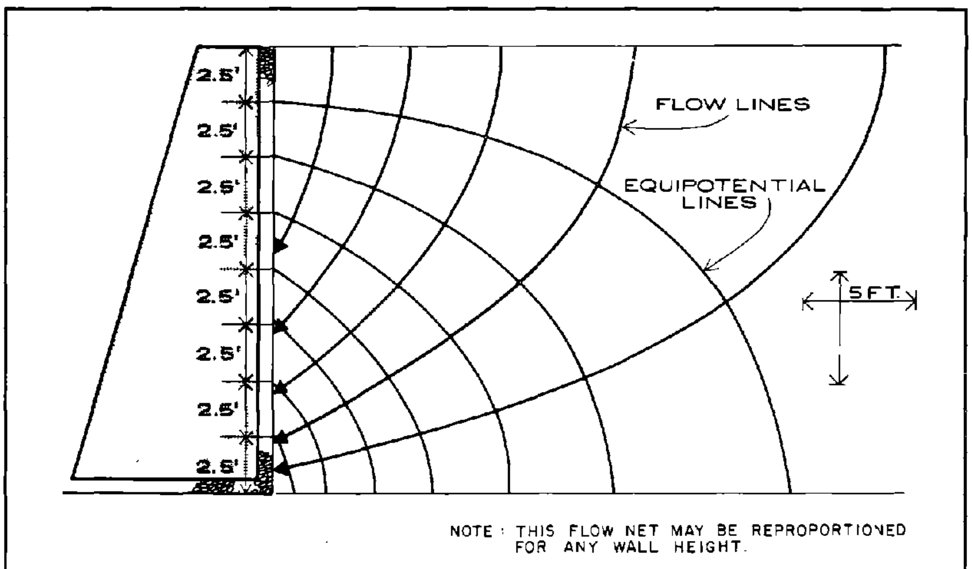
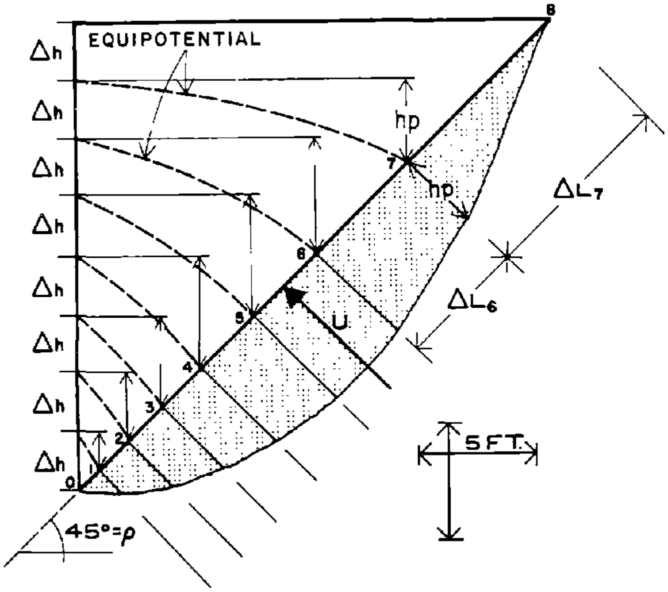


Figure 3-31.—Vertical drain and flow net for steady rainfall.



POINT	hp	INTERNAL ΔL	(hp) ave ΔL
0	0		
		1.2	0.9
1	1.5		
		1.7	3.8
2	3.0		
		2.1	7.2
3	3.9		
		2.3	10.0
4	4.8'		
		3.1'	15.4
5	5.1'		
		3.6'	19.0
6	4.9'		
		5.5'	23.1
7	3.5'		
		8.5'	14.9
8	0		
			94.3 ft²

PORE WATER FORCE ,U,
= 94.3 X 62.4
= 5890 LB./FT OF WALL

Figure 3-32.—Vertical drain—pore water force for ρ = 45°.

Careful note should be made of the method for determination of the pressure head, h_p. Also note the equal spacing of the equipotentials along the rear of the wall. The latter is always true for vertical drains. Having computed U for a given value of ρ, the total thrust, P_h, can be computed using

$$P_h = \frac{(w - U\cos\rho)\ \tan(\rho - \phi)\ +\ U\sin\rho}{\sin\delta\tan(\rho - \phi)\ +\ \cos\delta} \qquad (3\text{-}38)$$

or by graphical analysis of the force diagram. The total unit weight, γ, should be used to compute the weight, W, of the failure wedge in the solution by equation and graphical methods.

The location of the pore water distribution resultant, U, is found by breaking the distribution into simple geometric areas and taking moments about a fixed point, generally at the base of the wall.

Commonly, the vertical drain in an earth reinforced wall is placed behind the backfill, against the backslope excavation. Either near-vertical or sloping gravel chimney or geocomposite drains can be used (see figure 3-33). This configuration drains the structure before ground water can enter the backfill, so the analysis is similar to the free-draining backfill case with no additional water forces acting on the wall. If the backfill is not capped, or it has inadequate surface drainage such that surface water saturates the backfill (despite the backslope drain), then the analysis will be similar to that presented in this section, but with a double-draining case.

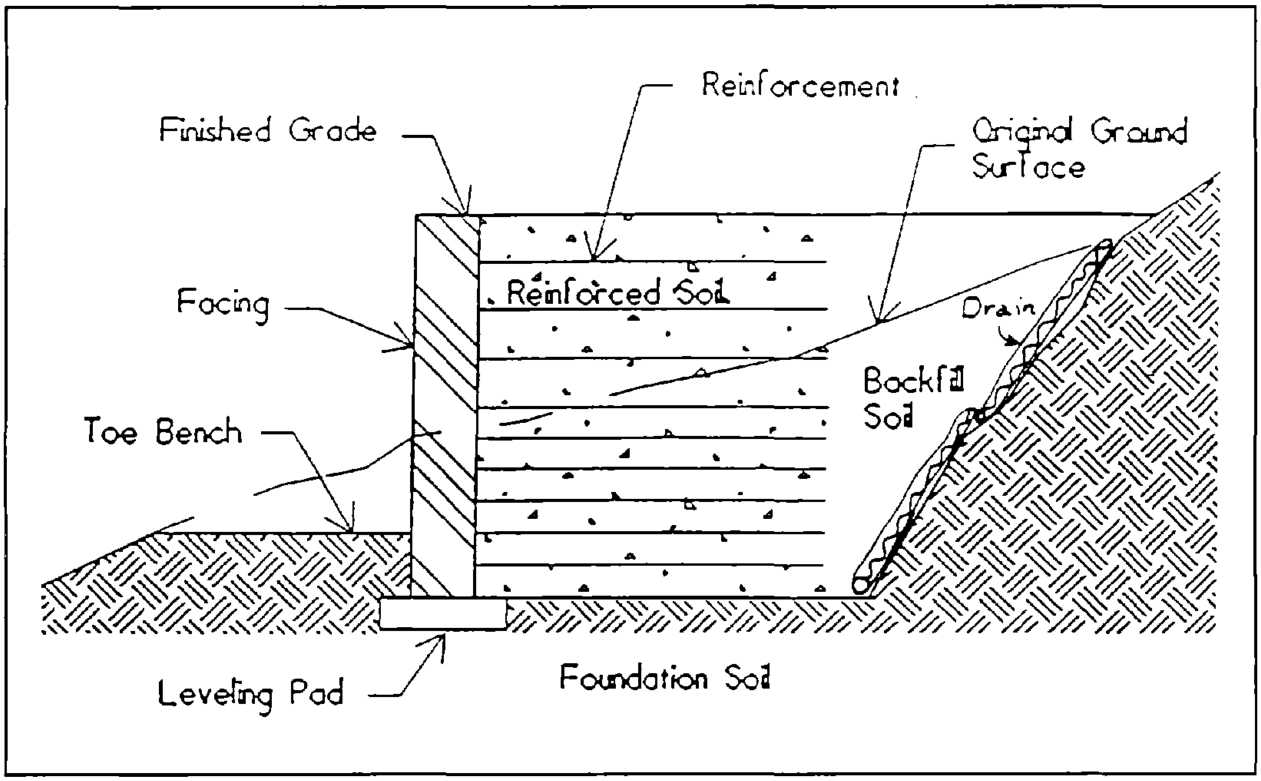


Figure 3-33.—Reinforced soil wall cross-section.

3F.5 Weep Holes

If weep holes are used with a free-draining backfill, the analysis is similar to that used for free-draining backfill and sloping underdrain. If weep holes are utilized as the only method of drainage for a fine-grained backfill, no simple, rational method of analysis exists. The wall should be designed for a full hydrostatic head with the weep holes supplying some unknown increase in the factor of safety. Weep holes should be provided for drainage behind rigid structures such as concrete walls. However, the use of only weep holes to achieve wall drainage is not recommended.

3F.6 Filter Requirements

Filters must be designed to prevent piping or migration of fine soil materials into the filter, and to prevent movement (erosion) of the filter material itself. While preventing the migration of fine substances, the filter must, at the same time, remain unplugged, remove water, and prevent the buildup of pore pressure in the system. Filter material has traditionally been a graded natural material such as sand and gravel, but, currently, geosynthetic "filter fabrics" or geotextiles are most commonly used for filtration. These types of materials must meet certain requirements, as shown in the following section.

3F.6.1 Granular Filter Criteria

To prevent migration of fine substances into the more coarse filter material, the filter needs to retain the coarsest 15 percent (D85) of the soil, or D15f (filter material)/D85s (soil) shall be less or equal to 5 percent.

To prevent the build up of pore pressure in the fine grain material (soil), the filter material must be relatively pervious, so D15f (filter material)/D15s (soil) shall be greater than 5 percent. Criteria referring to ratios of D50 (filter)/D50 (soil) should not be used.

For fine silts and clay-rich soils, the D15f should be less than or equal to 0.3 mm. See figure 3-34, which graphically shows the previously specified filter criteria.

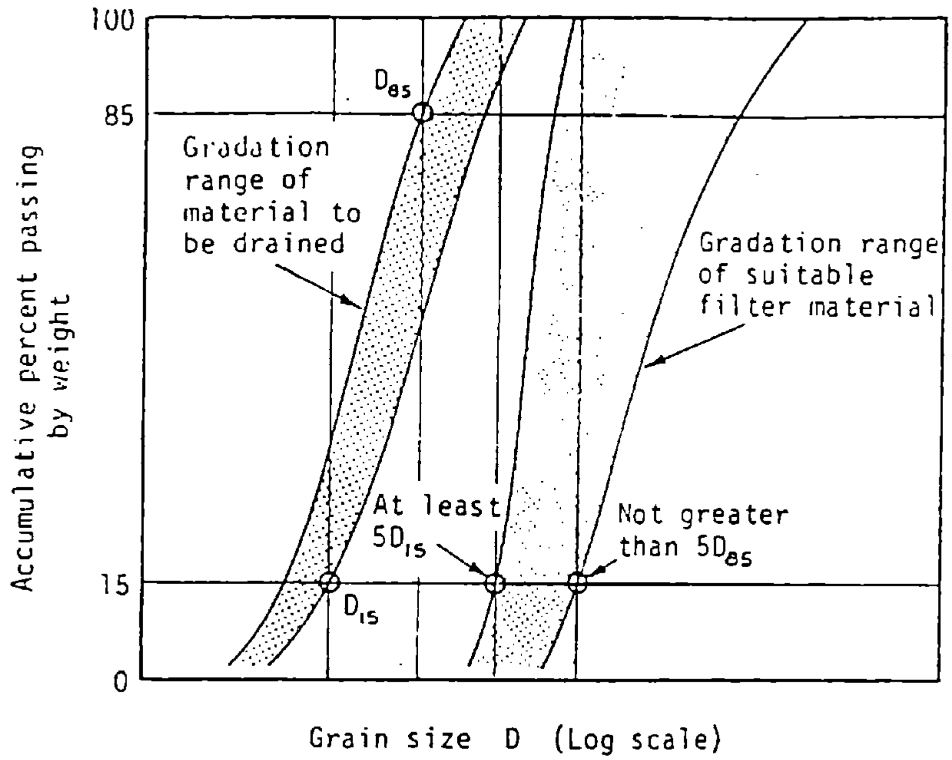


Figure 3-34.—Typical gradation requirements for filters and pervious blankets.

Filter material should contain no material larger than 3 inches, and no more than 2 percent passing the no. 200 sieve. The grain size curves of the filter material and soil should be approximately parallel in the finer range. Between fine or uniform soils and a coarse filter material, more than one layer of filter material (a "graded filter") may be necessary to meet the specified filter criteria.

3F.6.2 Geotextile Filter Criteria

For a geotextile to effectively perform as a filter in a geotextile drainage system, it must be capable of removing all the water that reaches the drain without excessive head buildup and without clogging or piping.

The key objectives of filter criteria with geotextiles are soil retention to control piping, sufficient water moving capability, long-term effectiveness, and survivability and durability.

Accurate geotextile design will depend on the specific site, soil, and hydraulic conditions. The most effective means of quantitatively evaluating a geotextile is to conduct long-term filtration testing with the project soil and the geotextile. However, for most routine applications, more qualitative evaluation and standard criteria are adequate.

A wide range of geotextile filter design criteria exists, considering application and factors such as soil density, gradation, uniformity, permeability, and the geotextile apparent opening size, percent open area, permittivity, strength, and durability properties. The *FHWA Geotextile Engineering Manual-Course Text* (1984) or the text by Koerner (1990) thoroughly addresses these factors and various available criteria. Reasonable minimum guidance is provided by the recommended filter criteria and materials requirements established by the AASHTO-AGC-ARTBA task force 25 (1990). This criteria is shown in table 3-11. Also AASHTO Specification M-288 on geotextiles is a good reference on the subject.

For soil retention to hold soil in place and control piping the following criteria should be noted:

(1) For soil with 50 percent or less of particles passing the no. 200 sieve, the effective fabric opening size number shall be greater than or equal to the no. 30 sieve (i.e., opening smaller than 0.60 mm).

(2) For soil with more than 50 percent of particles passing the no. 200 sieve, the effective fabric opening size number shall be greater than or equal to the no. 50 sieve (i.e., opening smaller than 0.297 mm).

For sufficient water passage capability the following criteria should be noted:

(1) For normal applications, the permeability of the geotextile shall be greater than that of the soil.

(2) For critical or severe applications (coarse angular aggregate, high degree of compaction, and deep (over 10 feet) applications), the permeability of the geotextile shall be at least 10 times greater than that of the soil.

Table 3-11.—AASHTO-AGC-ARTBA Task Force 25.

Recommended minimum[1] engineering fabric selection criteria in drainage and filtration applications

I. Piping resistance (soil retention—all applications)

A. Soils with less than 50 percent of particles by weight passing U.S. no. 200 sieve:
EOS no. (fabric) 30 sieve

B. Soils with more than 50 percent particles by weight passing U.S. no. 200 sieve:
EOS no. (fabric) 50 sieve

Note:
1. Whenever possible, fabrics with the lowest possible EOS no. should be specified.
2. When the protected soil contains particles from 1 inch to those passing the U.S. no. 200 sieve, use only the gradation of soil passing the U.S. no. 4 sieve in selecting the fabric.

II. Permeability

Critical/severe applications*	Normal applications
k (fabric) ≥ 10 k (soil)	k (fabric) ≥ k (soil)

* Woven monofilament fabrics only: percent open area ≥4.0 and EOS no. ≤100 sieve.

III. Chemical composition requirements/considerations

A. Fibers used in the manufacture of civil engineering fabrics shall consist of long-chain, synthetic polymers composed of at least 85 percent by weight of polyolephins, polyesters, or polyamides. These fabrics shall resist deterioration from ultraviolet exposure.

B. The engineering fabric shall be exposed to ultraviolet radiation (sunlight) for no more than 30 days in the time period following manufacture until the fabric is covered with soil, rock, concrete, and so forth.

IV. Physical property requirements (all fabrics)

	Fabric unprotected	Fabric protected
Grab strength (ASTM D-4632) (minimum in either principal direction)	180 lb	80 lb
Puncture strength (ASTM D-4833)[2]	80 lb	25 lb
Burst strength (ASTM D-3786)	290 psi	130 psi
Trapezoid tear (ASTM D-4533) (any direction)	50 lb	25 lb

Footnotes:

[1] All numerical values represent minimum average roll values (i.e., any roll in a lot should meet or exceed the minimum values in the table). Note: These values are normally 20 percent less than what manufacturers typically reported.

[2] Tension testing machine with ring clamp (i.e, steel ball replaced with a 5/16-inch-diameter solid steel cylinder with hemispherical tip centered within the ring clamp).

[3] Diaphragm test method.

[4] Fabric is said to be protected when it is used in drainage trenches or beneath/behind concrete (Portland or asphalt cement) slabs. All other conditions are said to be unprotected.

Examples of each condition are:

Protected: highway edge drains, blanket drains, smooth stable trenches <10 feet deep. In trenches where the aggregate is extra sharp, additional puncture resistance may be necessary.

Unprotected: stabilization trenches, interceptor drains on cut slopes, rocky or caving trenches, or smooth stable trenches >10 feet deep.

For woven monofilament geotextiles, the percent open area should be greater than or equal to 4, and the effective opening size sieve number should be less than or equal to the no. 100 sieve. Physical and chemical properties for the geotextile are shown on table 3-11. The geotextile shall not be exposed to sunlight for more than 30 days between the time it is manufactured and the time it is covered.

3F.7 Use of Geocomposite Drains

Geocomposite drains are commonly used in retaining structures to provide backfill drainage. They consist of some type of geotextile (to provide filtration) which is wrapped around or tacked or glued onto a somewhat rigid core material that can transport water. This "composite" drain then drains into a collector pipe or gravel bed to move water offsite. Geocomposite drains are particularly applicable behind fills and retaining structures, where the excavation backslope is steep or nearly vertical, making conventional gravel drains difficult to construct. Numerous manufacturers produce geocomposite drains and a variety of models and core materials are available. The installed cost of geocomposite drains in 1993 was approximately $2 to $4 per square foot.

The installation of geocomposite drains is relatively simple. For a rigid structure the drain is tacked up against that structure with the geotextile on the side of the drain facing the soil. Geotextile may not be needed on the side facing the structure, but it is still desirable, depending on the type of core material used. In soil, such as at the backslope excavation, a geocomposite drain with geotextile on both sides is required. Prefabricated or locally fabricated panels or sheets of geocomposite drain are placed against the backslope and tacked in place. Multiple panels should slightly overlap to keep the water out of the backfill, which should be lightly compacted against the drain.

Where considerable ground water flow is encountered and where gradients are low, such as in the flatter toe area of a slide, the capacity of some commonly available geocomposite drains may be exceeded. Conventional gravel drains, enveloped in geotextile, or a double layer of geocomposite drain may be best suited for this area. A combination of gravel drains and geocomposite drains have also been used.

Testing by the Forest Service (Stuart, Inouye, and McKean, 1991, and Koerner, 1990) and other institutions show that many geocomposite drains currently available have both good crushing strength properties and high flow capacity; however, some geocomposite drains do not have these features, and products vary considerably. In specifying geocomposite drains for wall or reinforced embankment applications, the items that should be specified include adequate crushing strength and flow capacity, and the use of a geotextile which will satisfy needed filter criteria. The horizontal and vertical flow of water within the sheet drain should interconnect at all times.

Suggested Forest Service specifications require a geocomposite drain that can maintain a flow of at least 1 gallon per minute per linear foot of drain under a hydraulic gradient of 1.0. Crushing strength must exceed 1.5 tons per square foot. Either woven or nonwoven geotextiles are specified, depending on local soils, but needle-punched, nonwoven materials are typically specified for general use. For the best performance, the geotextile should be tight or glued to the core material.

3G Construction Considerations

3G.1 Construction Changes

When dealing with subsurface materials, there may be changes from what was originally envisioned in the design process. Changes may include:

(1) Encountering rock in the excavation site where it was not expected. When this occurs there may not be enough room for the wall (i.e., reinforcing elements may need to be shortened, rock-excavated, or the wall redesigned or relocated). Any of these solutions may require the wall design to be reanalyzed by the design engineer for both internal and external stability. If the construction engineer or inspector is not familiar with the design, then an experienced designer should be consulted.

(2) When soil changes occur (e.g., soft soils, wet soils, and water or past water evidence such as piping cavities or mottling). For soft soils reanalyze external stability (bearing capacity, overturning, sliding, overall stability). If unexpected water is encountered, check the drainage system to make sure it is adequate to handle the water.

(3) If conditions require a higher wall (i.e., if good foundation is not founded at the expected depth) you will need to reanalyze both internal and external stability or overexcavate and backfill with good material.

3G.2 Site Layout

The contract should be specific on whether the engineer or contractor is responsible for survey control. If so, it is preferable to have the contractor do the survey control. Include the establishment of bench-marks and reference points and horizontal and vertical control during the work.

3G.3 End Treatments of Walls

Walls are often built too short to tie into natural ground. Wall ends should be buried, otherwise the slope may ravel or slide, exposing the ends. If during construction it is apparent that more length is needed, steps should be taken to lengthen the wall. This may require some redesign, and it may be necessary to have the designer review any field changes. Fills placed at the end of retaining walls cannot be placed on slopes steeper than 1 1/2:1; walls need to be extended to slopes that are no steeper than this ratio.

Handling surface drainage can be another problem at the end of a wall. Steps should be taken at the transition point to make sure there is no erosion. Once the erosion process starts, it is very difficult to repair.

3G.4 Inspection

To verify that the foundation, soil, and water conditions do not differ from the values assumed for the design, the design engineer should inspect the excavation site prior to erecting the wall. If conditions differ, the engineer should check the design with the new information to make sure that the safety factors are sufficient, or to redesign if necessary.

Specifications should consist of several factors the inspector needs to check during construction:

Construction tolerance. The amount of variation allowed in both the horizontal and vertical directions.

Easements. Make sure the necessary easements have been obtained. Do not overlook factors such as anchors that may extend beyond the right-of-way.

Clearing limits. These should be clearly staked.

Materials. Check lading bills to verify that the correct material was shipped. If required, sample materials and send them to the laboratory for testing. Collect any certifications as directed by the specifications.

Compaction. Check that compaction requirements are met as specified. Refer to 3D.2, "Semiemperical Methods," for information on requirements for rigid and soil reinforced walls, compaction levels, and equipment requirements.

Erection. Check that all components are fabricated and erected correctly, and the specified batter is achieved.

Excavation stability. Check the stability of excavated slopes since these are often made steeper than they should be.

3G.5 Geosynthethic Walls

The use of an external bracing system for temporary support during construction will restrain the wall from moving until the bracing is removed. Upon removal, the wall can move out as much as 6 inches. After this initial movement there should be no further lateral movement. By using the forming system shown in appendix C, all horizontal movement should take place during construction. The final wall face will be within tolerance for horizontal and vertical alignment and batter.

3G.6 Appurtenances

Culverts. Where culverts exit the face of a wall, special requirements may need to be made in fabrication of wall components. For concrete walls this can be done in forming the wall. Where reinforcing elements are involved, as in MSE walls, some of the reinforcing and face elements may be able to be field-cut to accommodate a pipe. For proprietary walls, check with the manufacturer to make sure what their policy is on handling pipes. The manufacturer generally has been through this before and may have details already worked out.

Guardrails. As with culverts, guardrails require special treatment. Guardrails may need to meet codes, such as FHWA or AASHTO codes for safety on public highways. Consult suppliers for details on the wall components, and make sure those details are approved by the necessary codes. Special fabrication may be required in some situations.

Footings. Where footings, such as for bridges, are placed on retaining walls the extra surcharge load needs to be considered in the design. Again, for proprietary walls it is best to consult the manufacturer for his or her experience, design details, or special requirements for footings on his or her own walls. In the absence of any other guidance, foundation loads should be modeled as area or line loads.

3G.7 Site Access

Those involved in design and construction should be aware of access requirements for construction. Consider the following questions: If one lane is closed for construction, is the other lane open to traffic? Will traffic control be required, and if so, for how long will the road be closed? On single-lane roads will traffic be able to pass or will the road need to be closed for a period of time? Will the contractor have access to both ends of the site during construction? What is the stability of the backslopes during construction?

3G.8 Drainage

Permanent drainage should be designed to handle water during the lifetime of the wall. For information in drainage design, refer to Section 3F, "Drainage." In excavating, it is important to prevent ponding of water which would saturate the foundation during the construction period. It may be necessary for the contractor to provide temporary drainage measures to handle water during construction. Care should be taken to prevent erosion and sediment transport during construction with the use of sediment control measures such as silt fences, hay bales or sediment traps if necessary.

3H Factor of Safety

When designing a structure, the engineer should establish the design assumptions and parameters based on the known facts and past experience. The designer must then select an appropriate factor of safety consistent with the relative degree of confidence that can be placed in the design assumptions and parameters. In considering the factor of safety of a design, the designer should recall that working stress methods of design have safety factors included in the allowable stress values.

Table 3-12 shows recommended factors of safety for the analysis of the external stability of permanent structures. The values presented in the table are based on the assumption that the structure will be designed to resist the real loads imposed on the structural system, and that the structure will be constructed in accordance with the intended design and specifications.

Table 3-12.—Recommended factors of safety.

	Bearing capacity	Over-turning	Sliding at base	Global stability
Normal highway loadings	2.0–3.0*	1.5–2.0*	1.5–2.0*	1.2–1.5
Occasional heavy transient loading	1.5–2.0	1.2	1.2	1.2

* The upper factor of safety range refers to silt and clay backfill or foundation soil.

Note that if either the backfill or the foundation soil is clay or silt, a somewhat higher factor of safety is required to provide satisfactory performance.

If, when designing a structure, the factor of safety does not conform with the guidelines presented in table 3-12, it is recommended that the engineer re-evaluate his design assumptions and soil parameters and attempt to better understand the problem. Then the design parameters that introduce the greatest amount of uncertainty into the computation should be adjusted. Adjusting all of the design parameters and assumptions only serves to compound the uncertainty.

Temporary structures with controlled loading conditions may be designed at a lower factor of safety than those presented in table 3-12; however, factor of safety values lower than 1.2 are not recommended. The design engineer must realize that the structure designed to the lower factor of safety value will probably yield more than one that is designed to the recommended value. Consequently, the designer and owner must be willing to accept a shorter design life and an occasional failure. If loss of life or adverse environmental impacts will result from the failure of a structure, no reduction in design factor of safety should be made, unless a comprehensive risk analysis has been conducted.

3I Stability

3I.1 General

When designing a retaining structure, both the internal and external stability of the structure must be evaluated. The external stability of the structure relates to the soil/ structure interaction and the overall stability of the wall system. The internal stability relates to the working stresses within the construction materials. There is some overlap between internal and external stability; for our purposes, they will be discussed as separate subjects here.

3I.2 External Stability

3I.2.1 Allowable Soil Bearing Pressure

The ultimate soil bearing capacities for conditions of general shear failure of a continuous footing placed on dense or stiff soil may be computed using

$$q_d = cN_c + \gamma D_f N_q + 1/2\ \gamma B\ N_\gamma \tag{3-38}$$

The values of N_c, N_q, and N_γ should be obtained from figure 3-35.

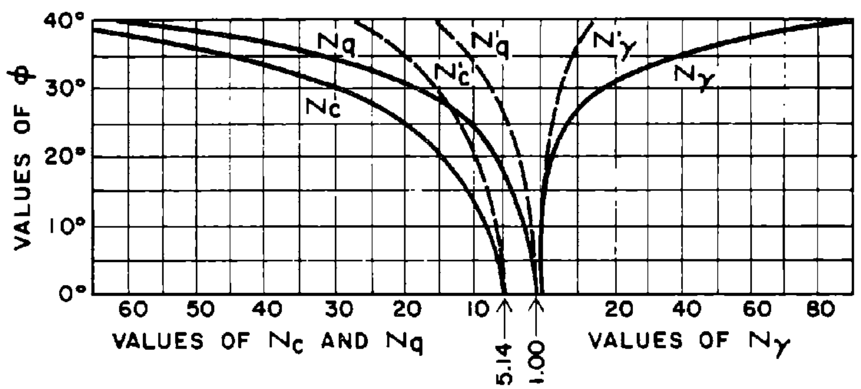


Figure 3-35.—Bearing capacity factors (after Terzaghi and Peck, 1967).

A factor of safety of 2.0 to 3.0 should be applied to determine the allowable soil bearing pressure, q_a, using

$$q_a = \frac{q_d}{FS} \tag{3-39}$$

Continuous footings found on loose or soft soil and designed to minimize foundation movement and wall deflection (which is generally the case for permanent structures) should be based on local shear criterion. The ultimate bearing capacities based on local shear may be calculated using

$$q'_d = 2/3\ c\ N'_c + \gamma D_f N'_q + 1/2 \gamma B N'_\gamma \tag{3-40}$$

Using a suitable factor of safety, generally 3.0, the allowable soil bearing pressures for local shear is obtained using

$$q_a = \frac{q'_d}{FS} \tag{3-41}$$

The foundation soils of the retaining wall must support not only the dead load of the wall, but also all moment-induced stresses, which are generally the controlling factor with respect to the bearing capacity of gravity and cantilever walls.

It is known that the bearing capacity of the footings on slopes should be reduced from the values computed in equation 3-41. However, experience shows that the overall stability of the retaining structure be generally the controlling criterion; therefore, the reduction in bearing capacity is usually of no consequence.

The following equations show relationships for computing q_d and q'_d for square footings. Again, the allowable soil bearing pressure should have a factor of safety of 2.0 to 3.0, depending on soil type.

$$q_d = 1.2\ cN_c + \gamma_t\ D_f N + 0.4\ \gamma_t BN \tag{3-42}$$

$$q'_d = 1.2\ cN_c' + \gamma_t\ D_f N' + 0.4\ \gamma_t BN_\gamma' \tag{3-43}$$

In the bearing capacity equations (3-38, 3-40, 3-42, and 3-43), the ground water table has been assumed to be at a depth equal to or greater than the footing width, B, below the base of the footing. If the ground water level is at the ground surface, the buoyant unit weight should be used in the calculations. For intermediate ground water levels the unit weight should be proportioned accordingly.

If the water table were at the ground surface, the allowable soil bearing pressure would be equal to half of the value shown for the equivalent 1 inch of settlement. Intermediate values of water level should be proportioned accordingly. Other design charts and tables are available that relate probable foundation behavior to various test data and soil conditions. They include those by Peck Hanson and Thornburn (1973) in figure 3-36, Teng (1964), and Bowles (1988).

For undrained conditions, i.e., $\phi = 0$, the ultimate soil bearing capacity for all footings is approximated by using

$$q_d = 6.2c \tag{3-44}$$

in which c is the undrained shear strength as determined from unconfined compression, unconsolidated undrained shear, or Torvane tests ($2c = q_u$ in which q_u is the unconfined compressive strength). Hence, for a factor of safety of 3.0 the allowable soil bearing pressure, q_a, is approximately equal to the unconfined compressive strength, q_u, as follows

$$q_a \approx q_u \tag{3-44}$$

31.2.2 Overturning

The factor of safety of a retaining wall to overturning is determined by summing the moments about the toe of the wall and evaluating

$$FS = \frac{\Sigma\ resisting\ moments}{\Sigma\ overturning\ moment} \tag{3-46}$$

In addition, the resultant of the moments should lie within the middle third of the retaining wall base.

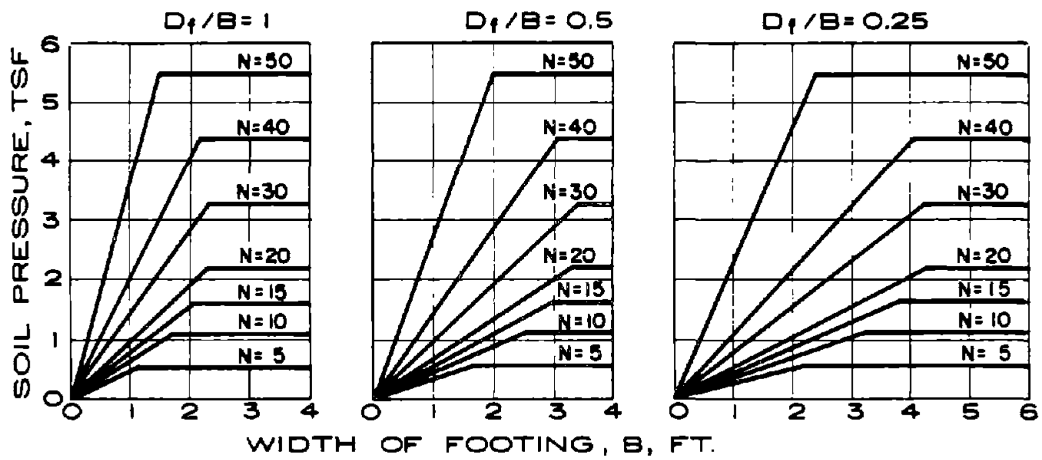


Figure 3-36.—Design chart for proportioning shallow footings on sand (after Peck, Hansen, and Thornburn, 1973).

If the resultant of the forces acting on the structure is not located at the center of the wall base, a moment-induced stress occurs beneath the structure. The magnitude of the moment-induced stress, σ_{mom}, is computed by taking the moment of the resultant, N, about the center line (CL) of the wall base and dividing by the section modulus of the base as in the following equation

$$\sigma_{mom} = \frac{M_{CL}}{S} \tag{3-47}$$

The net stress distribution acting beneath the wall is a combination of the uniform stress induced by the weight of the wall and the moment-induced stress developed by the horizontal thrust of the backfill. The magnitude of this combined stress must be less than the allowable bearing capacity.

3l.2.3 Sliding at Base

The factor of safety of a retaining structure against sliding at the base is the ratio of resisting forces to driving forces. Using figure 3-37, the factor of safety would be given using

$$FS = \frac{\overline{c_b} + \overline{N}\tan\delta_b + P_p}{P_h} = \frac{\Sigma\ resisting\ forces}{\Sigma\ driving\ forces} \tag{3-48}$$

in which δ_b, the friction angle of the wall-foundation material, is generally taken as equal to 2/3 $\overline{\phi}$, and $\overline{c_b}$, the cohesion or adhesion of the wall to the foundation material, is equal to 2/3 c, or to some other value based on experience.

Remolding and smoothing the exposed foundation soils can significantly reduce the frictional resistance of the soil-base contact. Hence, the soil-base friction angle for clays might be less than 2/3 ϕ. For firm-to-hard clays, δ_b may approach the residual friction angle, ϕ_r. Therefore, it is generally recommended that soil-wall cohesion be ignored in computing factor of safety against sliding for similar conditions.

For embedment of 3 feet or less, it is recommended that the passive resistance, P_p, be ignored (see figure 3-37).

If adequate resistance to sliding cannot be developed with a conventional base on the wall, shear keys can be incorporated into the design, as shown in figure 3-38. Example (a) in figure 3-38 illustrates a base key located near the stem so that the stem steel may be run into the key; example (b) shows that a sliding surface may develop where little aid is provided by the key. A heel key that presents two possible modes of failure (passive and slip along the plane) is shown in example (c).

Alternatively, the wall could be tied down with anchor tendons or bolts in order to increase the normal stress along the potential slip surface.

3l.2.4 Bearing Capacity on Slopes

Bearing capacity in a slope is reduced because the failure surface is truncated by the slope, as shown in figure 3-39. The reference (Shields et. al., 1990) provides a method for computing the reduced bearing capacity. The method uses the percentages shown in figures 3-40 and 3-41 to adjust the Gemperline flat surface bearing capacity.

The primary application has been to reduce the bearing capacity of bridge footings on fill slopes. The method should be applicable to the bearing capacity of retaining walls on slopes provided the subsurface materials beneath the retaining wall to a depth equal to the wall width are relatively homogeneous and cohesionless. Due to the relative width to burial of most retaining walls, the useful portions in figures 3-40 and 3-41 will be near the surface.

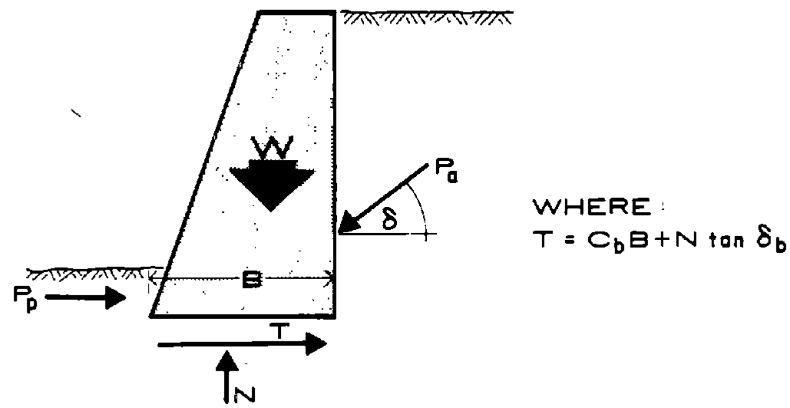


Figure 3-37.—External forces.

The reference provides curves for 1½:1 and 2:1 fill slopes, and extrapolation to flatter slopes is probably reasonable. Any slope steeper than 1½:1 probably has underlying subsurface materials which violate the assumptions of homogeneity and cohesionlessness. The Gemperline equation is invalid for $B > 45$ degrees. For steeper slopes, global stability should be considered rather than reduced bearing capacity. As noted in figure 3-39, the critical failure surface for bearing capacity is assumed to pass through the heel of the wall. If it does not, then global stability controls.

Due to the width-to-height ratios of many retaining walls and the increase of bearing capacity with increased footing or retaining wall width, *B*, the bearing capacity for cohesionless soils are often not a problem for retaining walls.

Figure 3-42 shows an example of the use of this method.

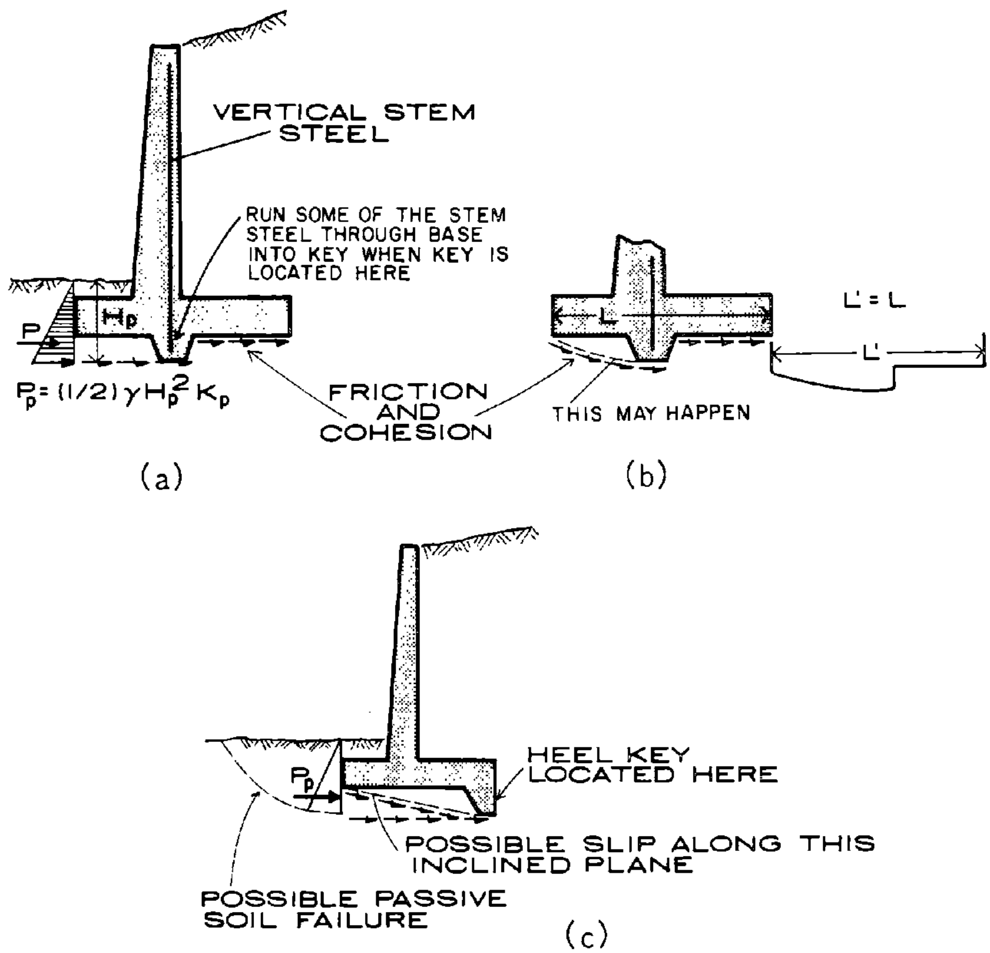
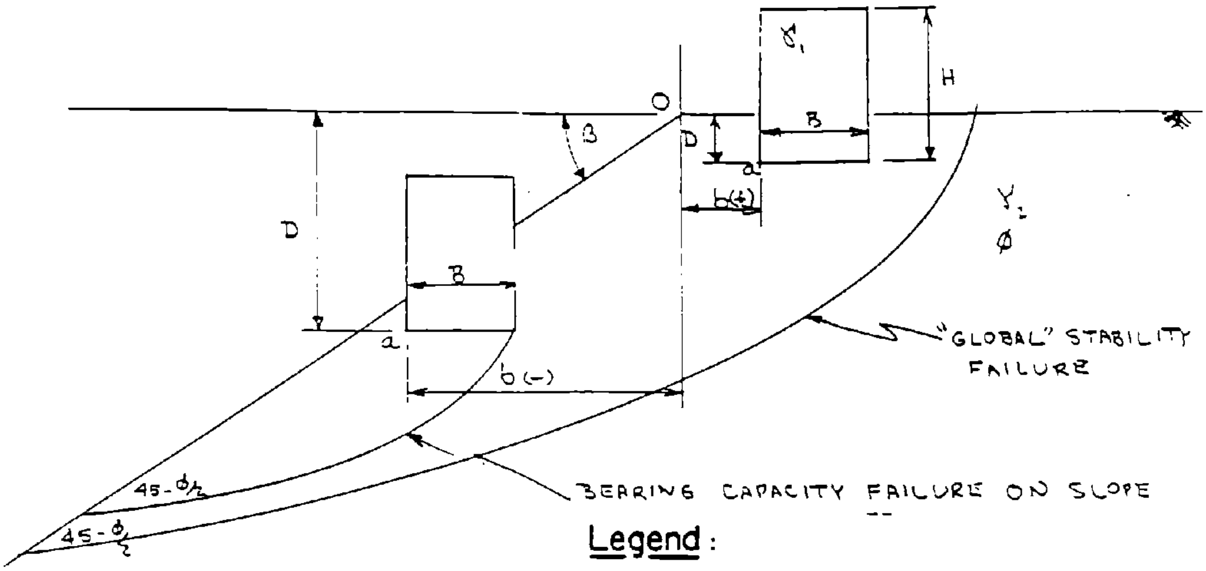


Figure 3-38.—Shear keys.



Note: $\lambda = b/B$
$\eta = D/B$
$\psi = L/B$

Legend:

β = angle of slope with respect to horizontal

D = depth of footing with respect to the level of the horizontal ground

B = footing width

L = footing length

b = horizontal distance leading edge of footing is away from crest of slope

GEMPERLINE FLAT GROUND REFERENCE BEARING CAPACITY EQUATION

$$N\gamma q_R = [10^{(0.1159\phi - 2.386)}][10^{(0.34 - 0.2\log_{10}B)}] \;.. \quad (3\text{-}49)$$

with ϕ in degrees and B in inches.

Figure 3-39.—Definition of terms.

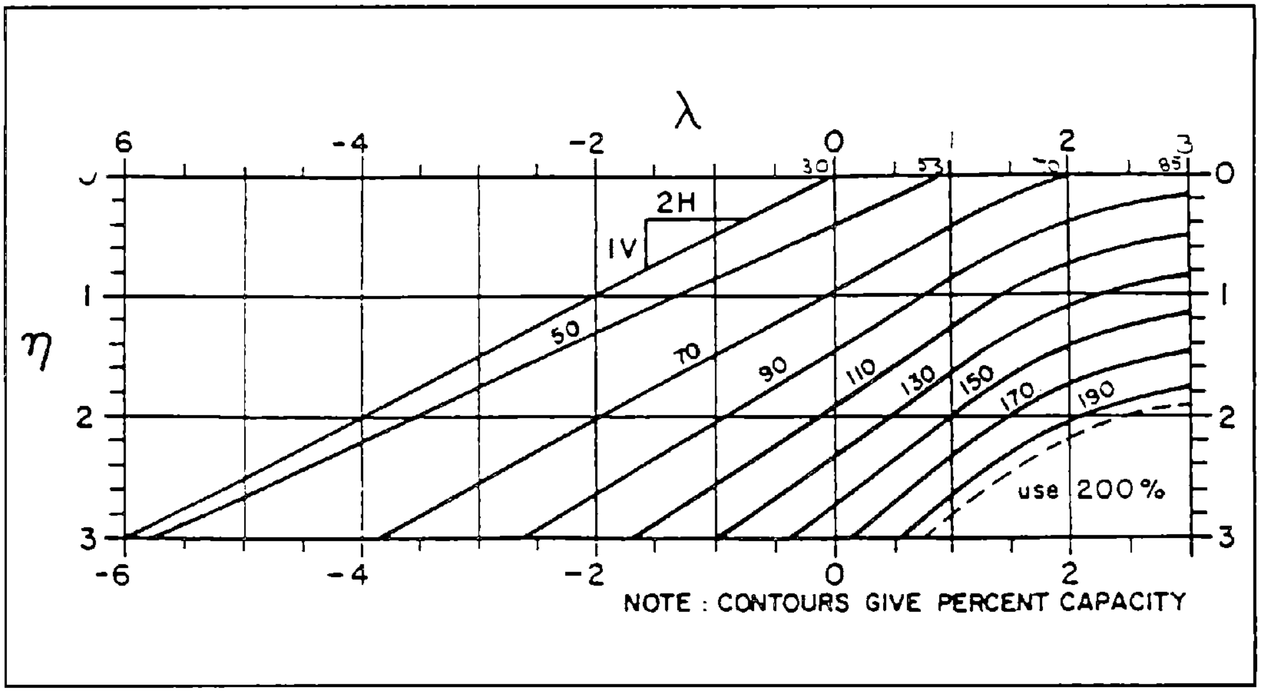


Figure 3-40.—Suggested design for a 26.6° slope.

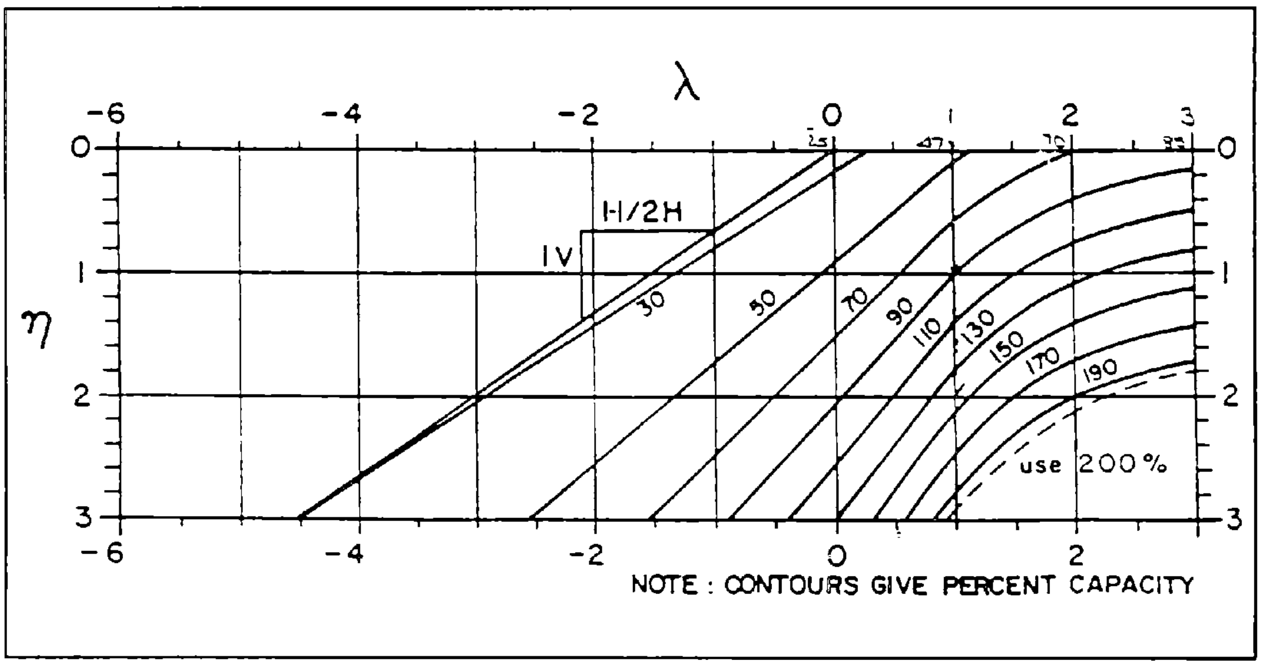


Figure 3-41.—Suggested design for a 33.7° slope.

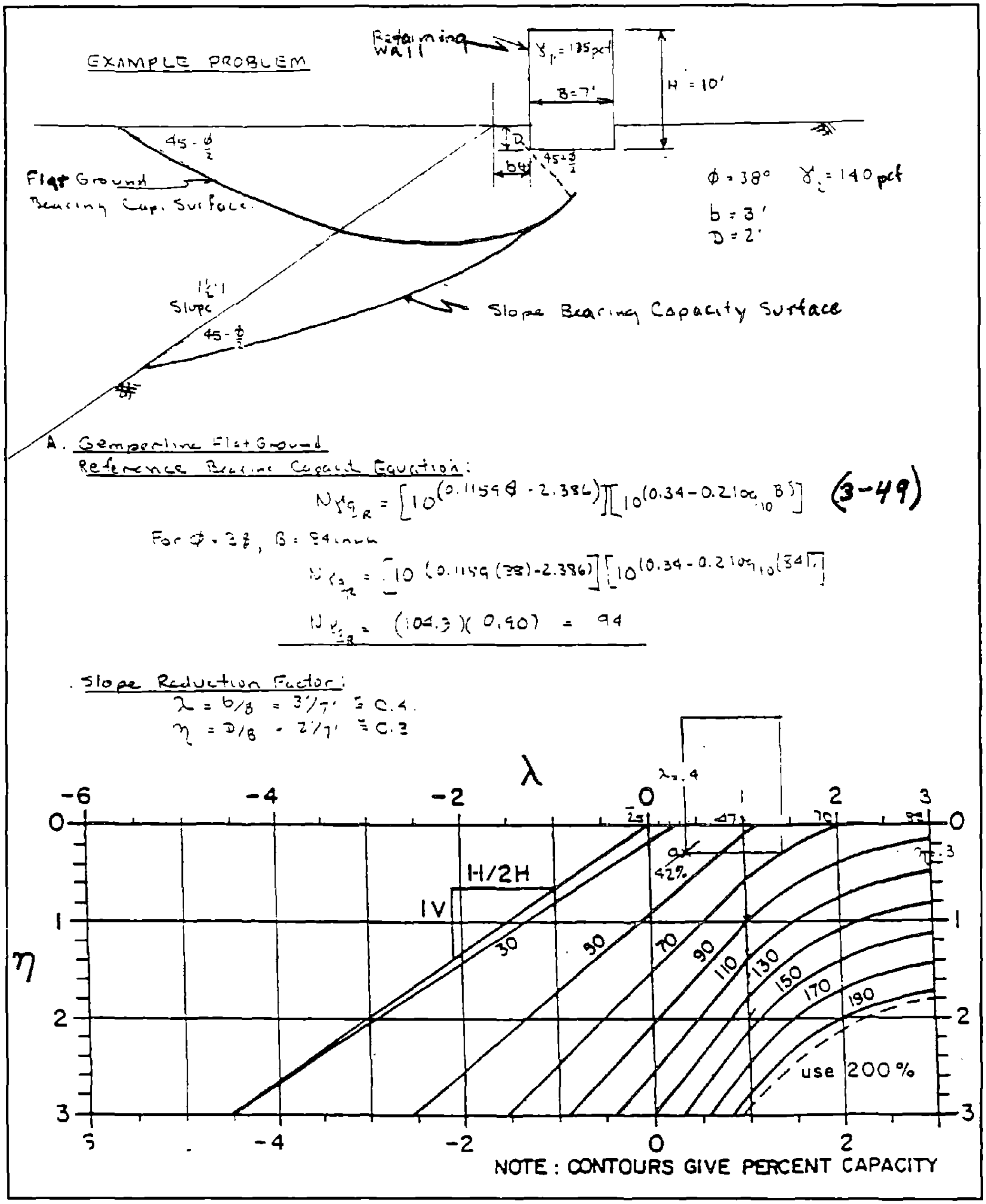


Figure 3-42.—Example: suggested design for a 33.7° slope.

Interpolating between $S_R = 30\%$ and 50%

$S_R \cong 0.6\,(50-30) + 30 = 42\%$

C. Bearing Capacity Factor for Slope:

$$N_{\gamma q_{RS}} = S_R \times N_{\gamma q_R} \qquad (3\text{-}50)$$

$$= (0.42)(94)$$

$$N_{\gamma q_{RS}} \cong 39$$

D. Bearing Capacity for Slope:

$$q_u = 0.5\,\gamma B N_{\gamma q_{RS}} \qquad (3\text{-}51)$$

$$= (0.5)(.140)(7)(39)$$

Ultimate $q_u = 19$ ksf

Allowable: For $FS = 3$

$$q_a = \frac{q_u}{FS} = \frac{19}{3} \qquad (3\text{-}52)$$

$$q_a = 6.3 \text{ ksf}$$

NOTE. Imposed loading, $q_i = \gamma_1 H = (.135)(10') = 1.35$ ksf $\ll q_a$
Reference discussion on general need to consider bearing capacity reduction for slope.

Figure 3-42.—Example: suggested design for a 33.7° slope (cont'd.).

31.2.5 Slope Stability

Upon completion of the wall design, the overall drained and undrained stability of the wall slope system must be calculated. Many methods of slope stability analysis are available. Any of the commonly used methods are sufficiently accurate for most retaining wall problems. Many stability methods, such as the modified Bishop or Janbu methods, are available in computer form for easy use. One such program that is widely used by highway agencies is XSTABL (Interactive Software Designs, Inc.). A slope stability guide has been developed by the Forest Service as a reference on slope stability analysis.

Construction of retaining walls often requires excavation that produces a steep cut slope as a temporary situation during construction. This slope may also need to be analyzed for safety during construction. Appropriate soil, water, and slope parameters reflecting the short-term construction conditions should be used in the analysis.

31.3 Internal Stability

The internal stability of a retaining structure refers to the integrity of the structural elements of the wall. Failure of a tieback rod in a restrained wall or of the stem of a concrete cantilever wall are examples of internal stability failures. Knowledge of the structural action of the various elements of a wall is essential in considering the internal stability of the structure.

This section will discuss methods of structural analysis for various wall types and present the development of the required earth pressure diagram for each wall type. The walls will be discussed by groups in accordance with the classification presented in table 2-1 (repeated here for convenience).

31.3.1 Gravity Walls

The internal stability of a gravity wall must be sufficient to resist the forces and stresses shown on figure 3-43. Also shown on figure 3-43 are the equations required to calculate the shear stress on any horizontal plane, V, the maximum compressive stress, f_c, and the maximum tensile stress, f_t.

31.3.1.1 Concrete Gravity Walls

From figure 3-43, the following equations are recommended for design of a concrete gravity wall.

(1) The shear stress on any horizontal plane, *bb'*, must be less than the available strength of the concrete in shear or

$$v = \frac{P_h}{12B'} \leq 1.1\sqrt{f'_c} \qquad \text{(3-53)}$$

(2) The maximum allowable compressive stress at points b, c, and e is

$$f_c = \frac{V}{12\,B'}\left(1 - \frac{6e}{B'}\right) \leq 0.45\,f'_c \qquad \text{(3-54)}$$

Table 2-1.—Wall classification.

Mechanically Stabilized	Anchored & Tieback
Reinforced Earth	H-pile, timber lagged
Geosynthetic	Vertical sheet pile
Stack Sack	Stack Sack
Modular block	All gravity structures
Welded wire	
Gravity	**Cantilever Piles**
Bin walls	Vertical sheet piles
Rectangular	H-pile, timber lagged
Circular	
Cross-tied	
Concrete crib	
Timber cribs	
Gabions	
Concrete gravity	
Concrete cantilever*	

* A cantilever wall is not a true gravity structure.

(3) The maximum allowable tensile stress at points b', c', and d' is

$$f_c' = \frac{V}{12\,B}\left(1 - \frac{6e}{B'}\right) \leq 1.1\,\sqrt{f}_c \tag{3-55}$$

Typical gravity wall designs are presented in appendix E.

3l.3.1.2 Bin, Crib, and Gabion Walls

Bin, crib, and gabion wall systems are generally considered proprietary wall systems. Their designs are typically accomplished with preexisting charts and tables. The design charts have been developed so that if the external stability of a wall is adequate, the internal stability is also generally satisfactory. For extreme loading conditions, the latter may not be true. Examples of proprietary wall design charts and tables are also presented in appendix E.

Bins, cribs, and gabions are designed to withstand some differential settlement. However, differential settlement resulting in angular distortions greater than about 1:100 should be avoided in order to prevent localized overstressing of structural components around connections, pillow blocks, and basket wires. Differential settlement may also induce localized failures in the form of tearing, buckling, and crushing of structural members at points of high contact stress (see Schuster, Jones, and Smart and Sack, 1978).

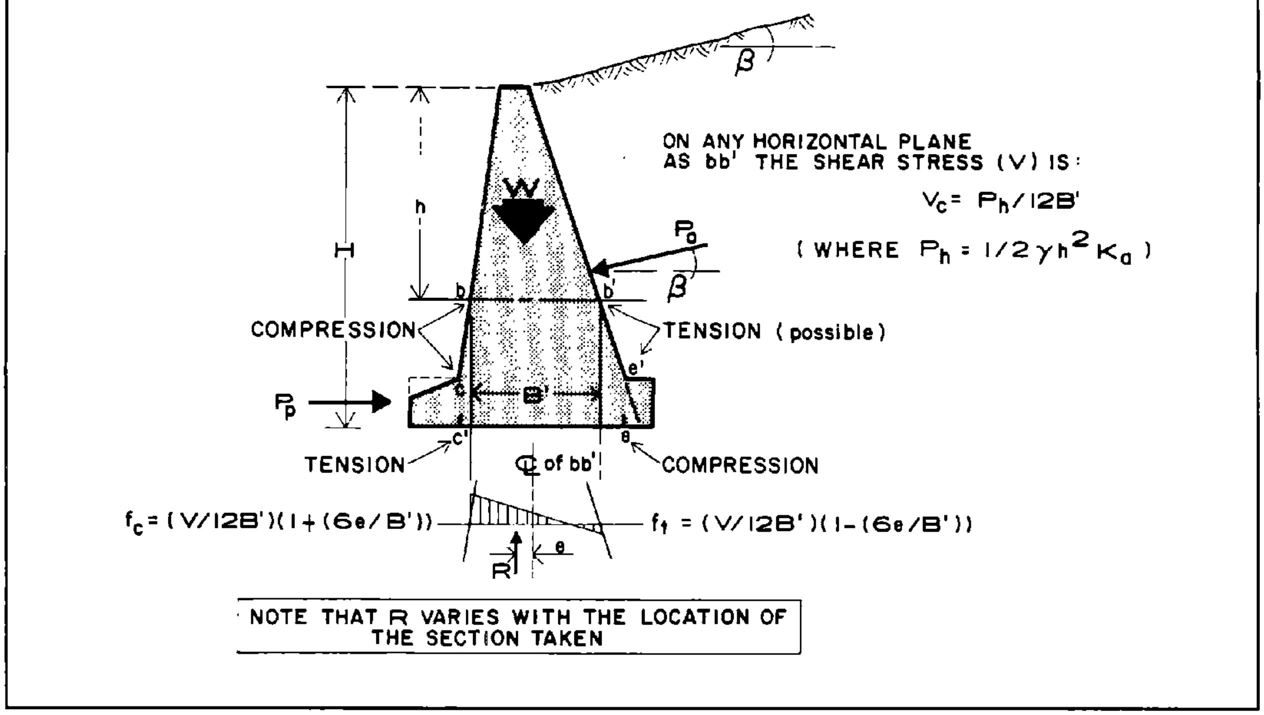


Figure 3-43.—Gravity retaining wall forces and stresses.

31.3.1.3 Concrete Cantilever Walls

Figure 3-44 shows three typical concrete cantilever walls. Of the three wall types, (c) is the most common, although types (a) and (b) have useful applications if encroachment is a problem, as on a stream or property line.

The earth pressure distribution used in the analysis of a concrete cantilever wall is similar to other gravity or semi-gravity structures. Many organizations and agencies have standard wall designs for various height and backfill considerations. Examples of designs by the Portland Cement Association are included in appendix E. Depending on the size and complexity of the job, the standard designs may provide the complete solution or a good preliminary design for cost estimating. If a complete analysis is necessary, a trial and error solution is required in order to optimize the design. Table 3-13 and the following paragraphs present guidelines for the design and analysis of the internal stability of a concrete cantilever wall. Assuming that the external stability of the wall has been satisfied in accordance with Section 31.2, "External Stability," the internal stability should be evaluated.

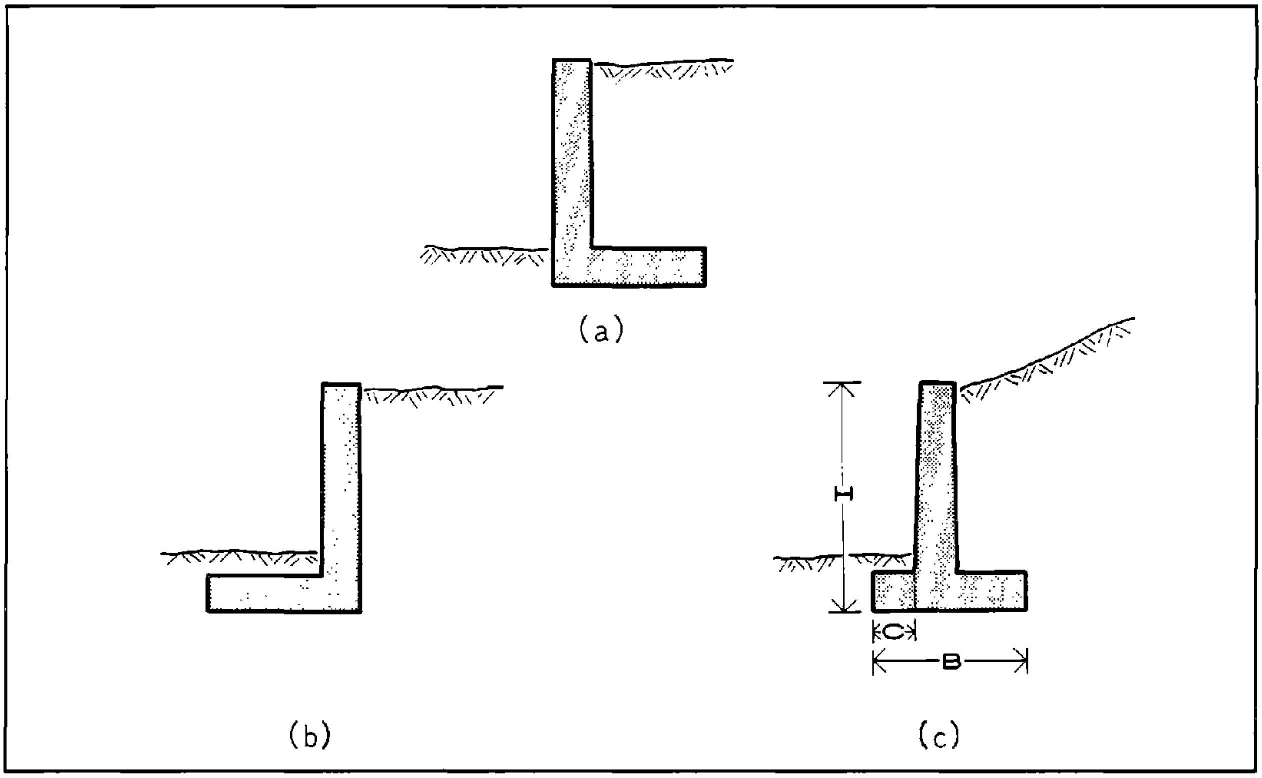


Figure 3-44.—Typical concrete cantilever wall types.

AASHTO requirements should be used in designing transportation structures. The "service load design method" (formerly known as the "allowable stress design method") is recommended for retaining wall design for its simplicity. The "strength design method" (formerly known as the "load factor design method") is also recognized by AASHTO, but it is not presented here. The "strength design method" may result in a smaller section and less reinforcing. General serviceability requirements of either code, such as requirements for deflection and crack control, must be met regardless of which method or code is used.

The design procedure outlined below is based on a 1-foot length of wall. Figure 3-45 shows the forces acting on a cantilever wall. These forces must be computed prior to initiating the internal stability analysis.

(1) Compute the bending moment of the wall at point A (see example (a) in figure 3-43, with equation 3-56) using

$$M_A = (H/3 - T_f)\ P_a \cos\beta \tag{3-56}$$

Table 3-13.—Design guidelines for cantilever walls.

(1) Footing width, B, is 0.4H to 0.55H for level backfills and 0.55H to 0.7H for steeper backslopes or surcharged walls.
(2) Dimension c (see example (c) in figure 3-46) is generally in the range of 0.2B to 0.23B.
(3) Minimum wall thickness at top is about 8 ± inches.
(4) Wall thickness at the stem base is about 0.12B.
(5) Thickness of footing is about 0.12B.

(2) Compute the stem thickness at the footing top (refer to table 3-13).

(3) Check a need for compressional steel in the wall stem.

In figure 3-46, the distance, d, is equal to the stem thickness, t, minus the specified thickness of reinforcing cover (generally 2 inches) plus half the diameter of the reinforcing bar, d_r, as shown in

$$d = t - (2 + d_r/2) \tag{3-57}$$

Using d from equation 3-57, compute F using

$$F = \frac{b\ d^2}{12{,}000} \tag{3-58}$$

in which b is the unit width under evaluation (b is generally taken as 12 inches). For material strengths selected, f'_c and f_y, refer to table 3-14 and select the proper value of K. Compute KF.

For $KF \geq M_A$, f_c is $\leq 0.45\, f'_c$ and no compressional steel is required on the face side of the wall. If $KF \leq M_A$ compressional steel is required (refer to standard reinforced concrete design reference texts).

(4) Compute amount of vertical reinforcing steel, A_s, required per foot at the back of the wall to resist tension using

$$A_s = \frac{M}{ad} \tag{3-59}$$

in which M (ft/kips) is the moment at any point, d (inches) is obtained from equation 3-57, and a is obtained from table 3-14. The amount of

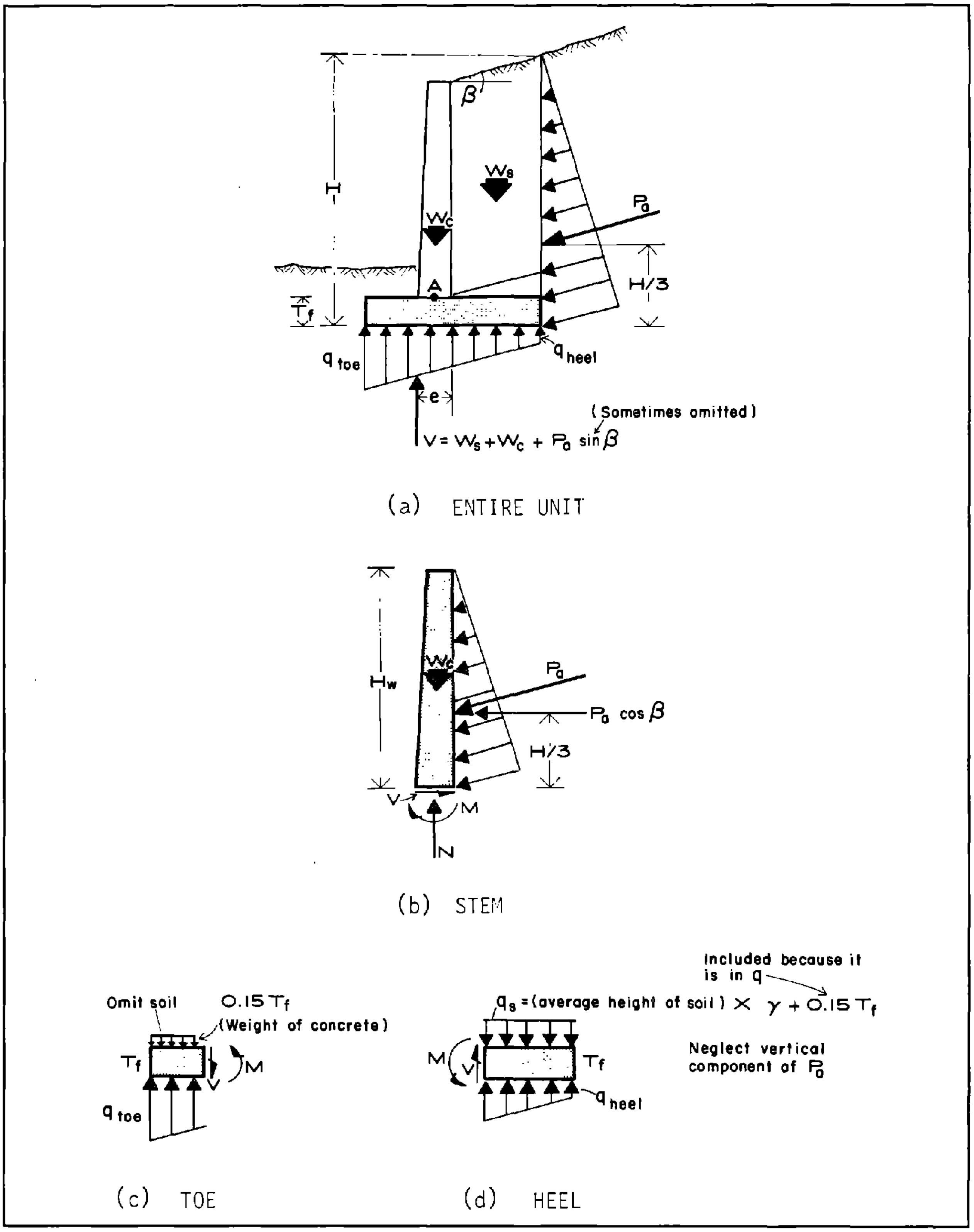


Figure 3-45.—Forces on cantilever wall.

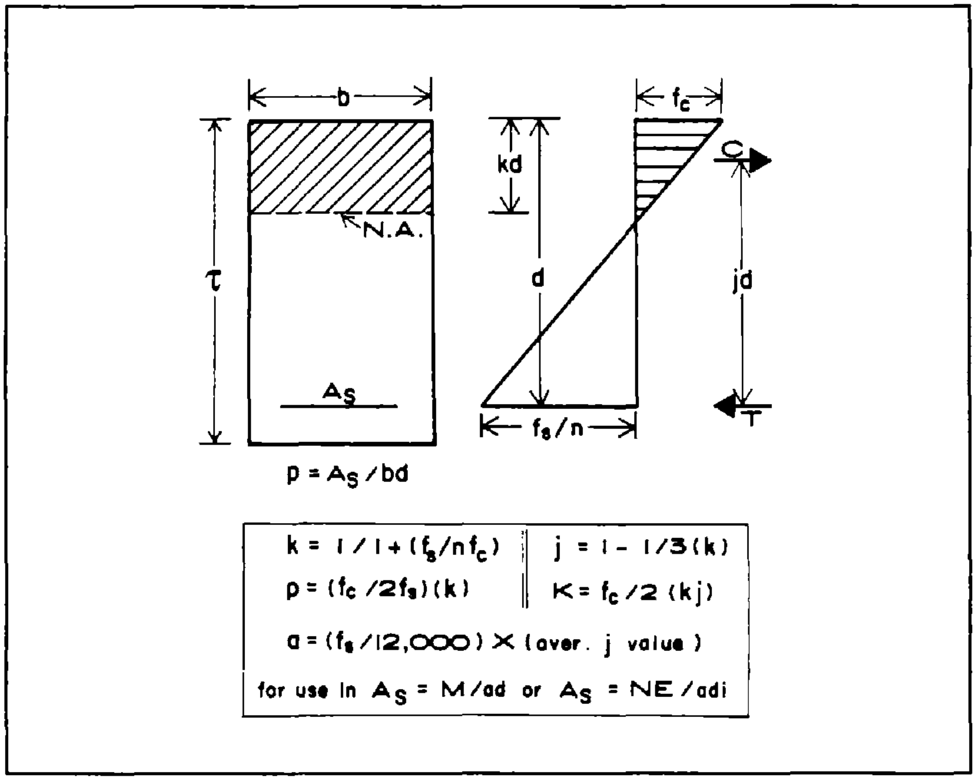


Figure 3-46.—Area equivalents.

Table 3-14.—Allowable design strengths.

f'_c psi	f_c psi	f_y ksi	a	K
2500	1125	40	1.44	179
		60	1.76	161
3000	1350	40	1.44	266
		60	1.76	204
3500	1575	40	1.44	274
		60	1.76	248
4000	1800	40	1.44	324
		60	1.76	295

steel required in the wall is variable and should be calculated for several different locations above point A (see figure 3-45). Likewise, the stem thickness can taper toward the top of the wall, and this variable should be evaluated in order to develop the most economical and eye-appealing design. Walls are generally tapered to keep the tilt from being noticed. A word of caution regarding tapered walls: increased forming costs may outweigh the savings in materials accrued by tapering the wall. When evaluating the internal stability of the stem, the relationship of M to KF and the required area of steel, A_s, should be computed for each point analyzed.

Tables 3-15 and 3-16 show properties of steel reinforcing bars to aid the designer in the selection of size and spacing. When sizing vertical reinforcement, it is customary to select members so that every second or third bar will extend to the top of the wall. The intermediate bars stop at lower points, as determined in the analysis previously described.

(5) Minimum requirements for other wall reinforcement depend on the gross planar cross-sectional area of the wall. Required coefficients of minimum reinforcement are presented in table 3-17. These coefficients are used to compute reinforcement for wall components in which negligible steel areas are computed by equation 3-59. The use of these coefficients in equation 3-60 gives the required minimum area of reinforcement $A_{s\text{ - min}}$

$$A_{s\text{-min.}} \leq (A_{concrete})(C_s) \tag{3-60}$$

Regardless of the relationship of KF to M, all walls greater than 10 inches thick must have steel on the front and back of the stem. In addition, neither the front nor the back of the wall may have *less* than one-third of the required reinforcement at a given section. The minimum allowable bar size is a no. 3, and the maximum allowable spacing is 18 inches.

The required area of tension steel calculated in step 4 will generally satisfy the minimum requirements of vertical reinforcement. Therefore, for walls greater than 10 inches thick the required vertical reinforcement on the face of the wall is simply one-third of the tension steel. Horizontal reinforcement is usually evenly divided between front and back, although it may be apportioned on a one-third to two- thirds basis. Typical reinforcement for cantilever walls is shown on page E-44.

(6) An examination of figure 3-43 shows that the concrete will be in tension at the top of the footing behind the wall and at the base of the footing in front of the wall. The greater tensile stresses are generally always behind the wall. The requirements for compressive steel and the minimum area of reinforcement are evaluated with equations 3-59 and 3-60 and the aid of table 3-17.

Table 3-15.—Areas and perimeter of bars in sections 1-foot-wide.

Areas A_s (or A'_s) (top) sq. in.; perimeters o, (bottom) in

Enter table with values of A_s (or A'_s) and $o = \frac{V}{7/8\ du}$ (V:lb; d:in.; u:psi)

Coefficients $a = \frac{f_s}{12{,}000} x\ j$ inserted in the table are for use in $A_s = \frac{M}{ad}$ or $A_s = \frac{NE}{adi}$

Spacing	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11	Spacing
2	0.30	0.66	1.20	1.86	2.64	----	----	----	----	----	2
	4.7	7.1	9.4	11.8	14.2	----	----	----	----	----	
2-1/2	0.24	0.53	0.96	1.49	2.11	2.88	3.79	----	----	----	2-1/2
	3.8	5.7	7.5	9.4	11.3	13.2	15.1				
3	0.20	0.44	0.80	1.24	1.76	2.40	3.16	4.00	----	----	3
	3.1	4.7	6.3	7.8	9.4	11.0	12.6	14.2			
3-1/2	0.17	0.38	0.69	1.06	1.51	2.06	2.71	3.43	4.36	----	3-1/2
	2.7	4.0	5.4	6.7	8.1	9.4	10.8	12.2	13.7		
4	0.15	0.33	0.60	0.93	1.32	1.80	2.37	3.00	3.81	4.68	4
	2.4	3.5	4.7	5.9	7.1	8.3	9.4	10.6	12.0	13.3	
4-1/2	0.13	0.29	0.53	0.83	1.17	1.60	2.11	2.67	3.39	4.16	4-1/2
	2.1	3.1	4.2	5.2	6.3	7.3	8.4	9.5	10.6	11.8	
5	0.12	0.26	0.48	0.74	1.06	1.44	1.90	2.40	3.05	3.74	5
	1.9	2.8	3.8	4.7	5.7	6.6	7.5	8.5	9.6	10.6	
5-1/2	0.1	1	0.24	0.44	0.68	0.96	1.31	1.72	2.18	2.77	3.405-1/2
	1.7	2.6	3.4	4.3	5.1	6.0	6.9	7.7	8.7	9.7	
6	0.10	0.22	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12	6
	1.6	2.4	3.1	3.9	4.7	5.5	6.3	7.1	8.0	8.9	
6-1/2	0.09	0.20	0.37	0.57	0.81	1.11	1.46	1.85	2.35	2.88	6-1/2
	1.4	2.2	2.9	3.6	4.4	5.1	5.8	6.5	7.4	8.2	
7	0.09	0.19	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67	7
	1.3	2.0	2.7	3.4	4.0	4.7	5.4	6.1	6.8	7.6	
7-1/2	0.08	0.18	0.32	0.50	0.70	0.96	1.26	1.60	2.03	2.50	7-1/2
	1.3	1.9	2.5	3.1	3.8	4.4	5.0	5.7	6.4	7.1	
8	0.08	0.17	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34	8
	1.2	1.8	2.4	2.9	3.5	4.1	4.7	5.3	6.0	6.6	
8-1/2	0.07	0.16	0.28	0.44	0.62	0.85	1.12	1.41	1.79	2.20	8-1/2
	1.1	1.7	2.2	2.8	3.3	3.9	4.4	5.0	5.6	6.2	
9	0.07	0.15	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08	9
	1.0	1.6	2.1	2.6	3.1	3.7	4.2	4.7	5.3	5.9	
9-1/2	0.06	0.14	0.25	0.39	0.56	0.76	1.00	1.26	1.60	1.97	9-1/2
	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.6	
10	0.06	0.13	0.24	0.37	0.53	0.72	0.95	1.20	1.52	1.87	10
	0.9	1.4	1.9	2.4	2.8	3.3	3.8	4.3	4.8	5.3	
10-1/2	0.06	0.13	0.23	0.35	0.50	0.69	0.90	1.14	1.45	1.78	10-1/2
	0.9	1.3	1.8	2.2	2.7	3.1	3.6	4.0	4.6	5.1	

Table 3-15.—Areas and perimeter of bars in sections 1-foot-wide (cont'd.).

Spacing	#2	#3	#4	#5	#6	#7	#8	#9	#10	#11	Spacing
11	0.05	0.12	0.22	0.34	0.48	0.65	0.86	1.09	1.39	1.70	11
	0.9	1.3	1.7	2.2	2.6	3.0	3.4	3.9	4.4	4.8	
11 ½	0.05	0.11	0.21	0.32	0.46	0.63	0.82	1.04	1.33	1.63	11 ½
	0.8	1.2	1.6	2.0	2.5	2.9	3.3	3.7	4.2	4.6	
12	0.05	0.11	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56	12
	0.8	1.2	1.6	2.0	2.4	2.8	3.1	3.5	4.0	4.4	
13	----	----	0.18	0.29	0.41	0.55	0.73	0.92	1.17	1.44	13
	----	----	1.4	1.8	2.2	2.5	2.9	3.3	3.7	4.1	
14	----	----	0.17	0.27	0.38	0.51	0.68	0.86	1.09	1.34	14
	----	----	1.3	1.7	2.0	2.4	2.7	3.0	3.4	3.8	
15	----	----	0.16	0.25	0.35	0.48	0.63	0.80	1.02	1.25	15
	----	----	1.3	1.6	1.9	2.2	2.5	2.8	3.2	3.5	
16	----	----	0.15	0.23	0.33	0.45	0.59	0.75	0.95	1.17	16
	----	----	1.2	1.5	1.8	2.1	2.4	2.7	3.0	3.3	
17	----	----	0.14	0.22	0.31	0.42	0.56	0.71	0.90	1.10	17
	----	----	1.1	1.4	1.7	1.9	2.2	2.5	2.8	3.1	
18	----	----	0.13	0.21	0.29	0.40	0.53	0.67	0.85	1.04	18
	----	----	1.0	1.3	1.6	1.8	2.1	2.4	2.7	3.0	

Table 3-16.—Properties of steel reinforcing bars.

Nominal Dimensions—Round Sections Bar Designation Number						
	2	3	4	5	6	7
Unit weight per ft lb	0.167	0.376	0.668	1.043	1.502	2.044
Diameter (in)	0.250	0.375	0.500	0.625	0.750	0.875
Cross-sectional area (sq in)	0.05	0.11	0.20	0.31	0.44	0.60
Perimeter (in)	0.786	1.178	1.571	1.963	2.356	2.749
	8	9	10	11	14S	18S
Unit weight per ft lb	2.670	3.400	4.303	5.313	7.65	13.60
Diameter (in)	1.000	1.128	1.270	1.410	1.693	2.257
Cross-sectional area (sq in)	0.79	1.00	1.27	1.56	2.25	4.00
Perimeter (in)	3.142	3.544	3.990	4.430	5.32	7.09

Table 3-17.—Coefficients of minimum reinforcement (C_s).

Steel grade	Vertical	Horizontal
40	0.0015	0.0025
60	0.0012	0.0020

(7) Continuous longitudinal reinforcement in the footing is not specified by code, but the use of a minimum of no. 4 bars at 18-inch centers (top and bottom) are recommended to tie all transverse reinforcing bars together.

(8) Shear in a retaining wall design is usually not critical; however, it should be checked. The allowable shear stress, V_c, is provided using

$$V_c = \frac{\text{horizontal forces}}{\text{stem width}} \leq 1.1\sqrt{f'_c}. \tag{3-61}$$

If the allowable shear stress is exceeded, a text on reinforced concrete design should be consulted.

3I.3.2 Cantilever Pile Walls

3I.3.2.1 Cantilever Sheet Piles

The recommended maximum height for cantilever sheet pile walls is 15 feet. The method of analysis presented below is intended for cohesionless soils. The method also applies to the long-term conditions for silts and clays where the cohesion, c, approaches zero and the friction angle, $\bar{\phi}$, has been reduced to an effective value in the range of 20 to 30 degrees. The reduction in $\bar{c}$ and $\bar{\phi}$ occurs as a result of a creep-like motion.

Figure 3-47 shows the earth pressure diagram for a driven sheet pile wall. In the figure, p_a and p_p are equivalent fluid pressures in which

$$p_a = \gamma K_a \tag{3-62}$$

and

$$p_p = \gamma K_p \tag{3-63}$$

Values K_a and K_p are obtained from the appropriate equation or chart (see Chapter 3, "Theoretical Methods").

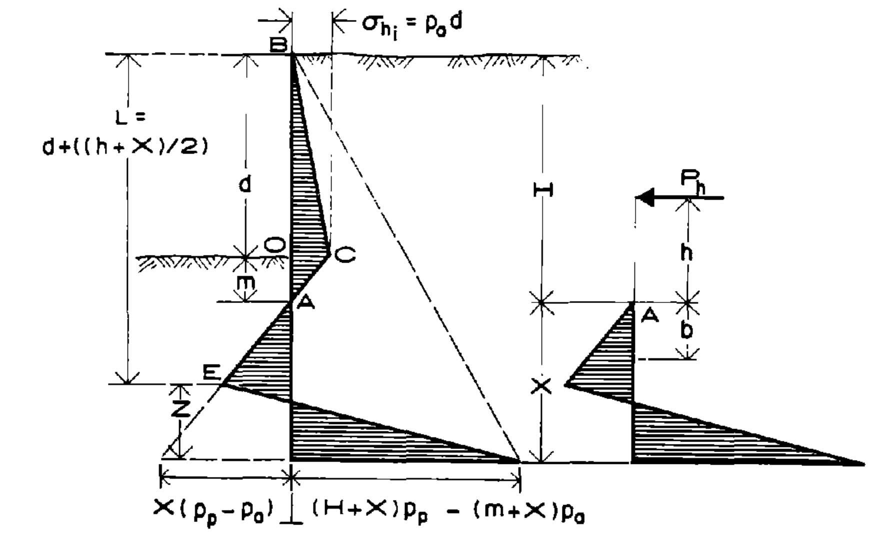


Figure 3-47.—Earth pressure diagram of a cantilever sheet pile wall.

The following steps should be followed to develop the diagram shown in figure 3-47.

(1) Compute the horizontal pressure at point O using equation 3-64 and construct line BC.

$$\sigma_h = p_a d = \gamma \ K_a \ d \tag{3-64}$$

(2) Compute the depth, *m*, to the point where the passive stress equals the active stress using

$$m = \frac{p_a d}{p_p - p_a} \tag{3-65}$$

Construct *CA* and extend the line beyond point *A* for future use.

(3) Compute P_h, the resultant of the forces acting above point *A*, using

$$P_h = \frac{p_a d}{2} (d + m) \tag{3-66}$$

(4) Compute the location of P_h by taking moments about point A. Divide triangle ABC into two smaller triangles, and use the 1/3-point rule.

(5) Compute the required depth of penetration, x, below point A using

$$6P_h(h + x) - x^3(p_p - p_a) + \frac{[(p_p - p_a)x^2 - 2P_h]^2}{2(p_p - p_a)x + Hp_p - mp_a} = 0 \qquad (3\text{-}67)$$

The solution of equation 3-67 is most readily treated as a trial-and-error solution. The use of an electronic spreadsheet or programmable calculator will minimize the effort required for convergence. Typical values of total depth of penetration (see figure 3-47), $m + x$, are presented in table 3-18. These depths are based on horizontal soils above and below the wall. Slopes dropping away in front of the wall or rising above the top of the wall will increase these penetration depths.

Table 3-18.—Approximate penetration depths of cantilever sheet pile walls.

Standard penetration resistance (N) blows/foot	Relative density of soil (D_r)	Depth of penetration (m+x)
0–4	very loose	2.0 d*
5–10	loose	1.5 d
11–30	medium density	1.25 d
31–50	dense	1.0 d
+50	very dense	0.75 d

*d = *The height of piling above the dredge line.*

(6) The loading diagram can be completed by computing z using

$$z = \frac{(p_p - p_a)x^2 - 2P_h}{2(p_p - p_a)x + Hp_p - mp_a} \qquad (3\text{-}68)$$

and constructing the required lines in accordance with figure 3-47. (The problem can be solved without step 6.)

(7) Compute the depth to the point of zero shear, b, using

$$b = \sqrt{\frac{2\, P_h}{p_p - p_a}} \tag{3-69}$$

The point of zero shear is also the point of maximum bending moment.

(8) Compute the maximum bending moment at point b using

$$M_{\max} = P_h(h + b) - \frac{b^3}{6}(p_p - p_a) = P_h\left(h + \frac{2b}{3}\right) \tag{3-70}$$

(9) Compute the minimum required section modulus, S_{min}, using

$$S_{\min} = \frac{M_{\max}}{F_b} \tag{3-71}$$

in which F_b is the allowable design stress of the steel and M_{max} is the maximum bending moment. If the calculated section modulus of the sheet pile appears uneconomical, the designer should investigate the benefits of reducing the calculated bending moments by using Rowe's moment reduction theory. However, little reduction in bending moment is generally realized for low walls. An excellent illustration of Rowe's theory is found in Bowles, 4th edition, 1988.

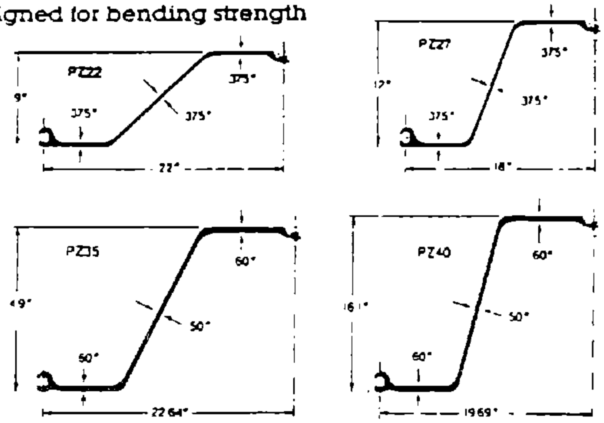
Table 3-19 presents a partial summary of engineering constants for various hot-rolled Bethlehem sheet pile sections. Included in the table are section moduli in terms of individual sheets and per foot of wall. The equations presented in the table are on a per foot basis.

In addition to the hot-rolled sections shown, there is a wide variety of other piling available. This includes hot-rolled sections produced outside the United States, and cold-formed sections produced both domestically and outside the United States. Last counted, at least 150 steel sheet pile sections were being produced. One source of data on available sections is Pile Buck, Inc., steel sheet piling specifications chart. This publication is available from Pile Buck, Inc., P.O. Box 1056, Jupiter, FL 33468-1056, 407-744-8780.

Table 3-19.—Steel sheet piling sections.

BETHLEHEM STANDARD SHEET PILING

Sections designed for bending strength



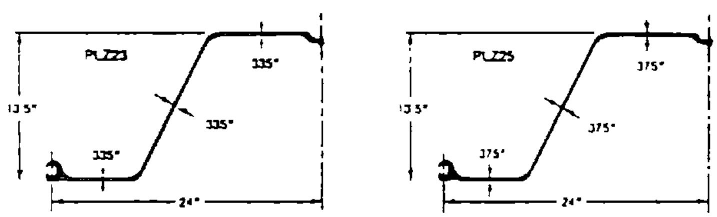
Sections designed for interlock strength

Properties and Weights

Section Designation	Area sq in.	Nominal Width, in.	Weight in Pounds: Per lin ft of bar	Weight in Pounds: Per sq ft of wall	Moment of Inertia, in.⁴	Section Modulus, in.³: Single Section	Section Modulus, in.³: Per lin ft of wall	Surface Area, sq ft per lin ft of bar: Total Area	Surface Area, sq ft per lin ft of bar: Nominal Coating Area*
PZ22	11.86	22	40.3	22.0	154.7	33.1	18.1	4.94	4.48
PZ27	11.91	18	40.5	27.0	276.3	45.3	30.2	4.94	4.48
PZ35	19.41	22.64	66.0	35.0	681.5	91.4	48.5	5.83	5.37
PZ40	19.30	19.69	65.6	40.0	805.4	99.6	60.7	5.83	5.37
PSA23	8.99	16	30.7	23.0	5.5	3.2	2.4	3.78	3.08
PS27.5	13.27	19.69	45.1	27.5	5.3	3.3	2.0	4.48	3.65
PS31	14.96	19.69	50.9	31.0	5.3	3.3	2.0	4.48	3.65

*Excludes socket interior and ball of interlock

BETHLEHEM INTERMEDIATE SHEET PILING



Properties and Weights

Section Designation	Area sq in.	Nominal Width, in.	Weight in Pounds: Per lin ft of bar	Weight in Pounds: Per sq ft of wall	Moment of Inertia, in.⁴	Section Modulus, in.³: Single Section	Section Modulus, in.³: Per lin ft of wall	Surface Area, sq ft per lin ft of bar: Total Area	Surface Area, sq ft per lin ft of bar: Nominal Coating Area*
PLZ23	13.28	24	45.2	22.6	407.5	60.4	30.2	5.98	5.52
PLZ25	14.60	24	49.6	24.8	446.5	65.7	32.8	5.98	5.52

*Excludes socket interior and ball of interlock

Bethlehem
Piling Products
Bethlehem Steel Corporation
Bethlehem, PA 18016

31.3.2.2 Cantilever Sheet Piles with Uniform Surcharge

If the sheet pile wall is influenced by a uniform vertical surcharge, Δq, the wall may be analyzed in a manner similar to that previously presented. The applicable stress distribution is given in figure 3-48 and the required equations are presented below in order of use (see equations 3-72 through 3-75).

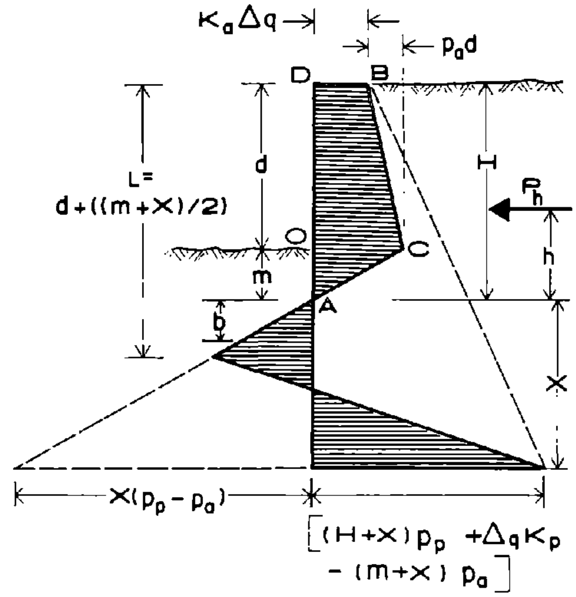


Figure 3-48.—Earth pressure distribution cantilever sheet pile wall—uniform surcharge.

$$\sigma_h = \Delta q K_a + p_a d = \Delta q K_a + \gamma K_a d \tag{3-72}$$

$$m = \frac{p_a d + \Delta q K_a}{p_p - p_a} \tag{3-73}$$

$$P_h = \Delta q d K_a + \frac{p_a d^2}{2} + \frac{(\Delta q Ka + p_a d)m}{2} \tag{3-74}$$

$$6P_h(h + x) - x^3(p_p - p_a) - \frac{[(p_p - p_a)x^2 - 2P_h]^2}{2(p_p - p_a)x + Hp_p - mp_a + \Delta q K_p} = 0 \tag{3-75}$$

The equations for locating the depth below point A to the point of zero shear, maximum bending moment, and allowable section modulus remain unchanged (see equations 3-69, 3-70, and 3-71).

3I.3.2.3 Cantilever Soldier Pile Wall with Lagging

The analyses of lagged cantilever soldier pile walls and cantilever sheet pile walls are similar. The major difference is that the soldier pile wall develops passive resistance only at the pile locations. The two analyses are related by the coefficient C, the ratio of the effective width, and W_e, the approximate effective width of the pile to the center-to-center spacing of the piles. The value of W_e of the soldier pile may be determined from table 3-20. For the ease of computation, the following analysis is based on 1 linear foot of wall. Hence, when the H-pile sections are computed, the bending moment must be multiplied by the pile spacing.

Table 3-20.—Effective pile width.

Soil type	Range of effective pile width (W_e)
All fine-grained soil	B–3B*
Very loose to medium dense granular soil	B–3B
Medium dense to very dense granular soil	3B–5B

* *B is the actual pile width.*

Figure 3-49 shows the earth pressure diagram for a cantilever soldier pile wall. The earth pressure distributions shown in figure 3-49 may be developed using the following steps:

(1) Compute C, the ratio of effective pile width to the center-to-center spacing of the soldier piles.

(2) Compute the horizontal pressure at point 0 using equation 3-64, repeated here. Construct line BC.

$$\sigma_h = P_a d = \gamma\ K_a d \tag{3-64}$$

(3) Compute the depth, m, to the point where the passive stress equals the active stress using

$$m = \frac{C p_a d}{p_p - p_a} \tag{3-76}$$

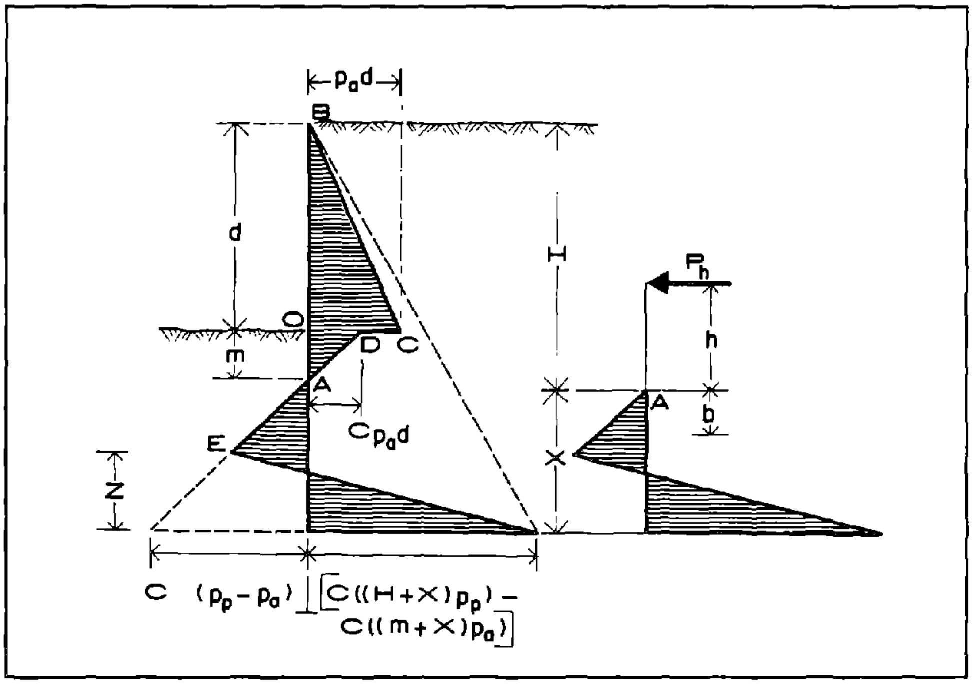


Figure 3-49.—Earth pressure diagram for cantilever soldier pile wall.

(4) Compute P_h, the resultant of the forces acting above point A, using

$$P_h = \frac{p_a d}{2}(d + Cm) \tag{3-77}$$

(5) Compute the location of P_h by taking the moments of triangles *BOC* and *AOD* about *A* (recall the 1/3-point rule for triangles).

(6) Compute the required penetration depth, x, below point A using

$$6P_h(h + x) - x^3(p_p - p_a)C + \frac{[Cx^2(p_p - p_a) - 2P_h]^2}{[2x(p_p - p_a) + Hp_p - mp_a]C} = 0 \tag{3-78}$$

Solve for x using the trial-and-error method. For soils medium in density or denser, the total penetration depth (see figure 3-49), $m + x$, , should be on the order of 1.5 d to 2.5 d, in which d is the cantilever wall height.

(7) The loading diagram can be completed by computing Z using

$$Z = \frac{(p_p - p_a)x^2C - 2P_h}{[2(p_p - p_a)x + Hp_p - mp_a]C} \tag{3-79}$$

and constructing the required lines in accordance with figure 3-49. (The problem can be solved without step 7).

(8) Compute the depth below point A to the point of zero shear, b, using

$$b = \sqrt{\frac{2P_h}{(p_p - p_a)C}} \tag{3-80}$$

The point of zero shear is also the location of the maximum bending moment.

(9) Compute the maximum bending moment per foot of wall with

$$M = P_h(h + b) - \frac{b^3}{6}(p_p - p_a)C \tag{3-81}$$

(10) Compute the minimum required section modulus, S_{min}, per pile using

$$S_{\min} = \frac{M_{\max}}{F_b} s \tag{3-82}$$

in which s equals pile spacing (ft).

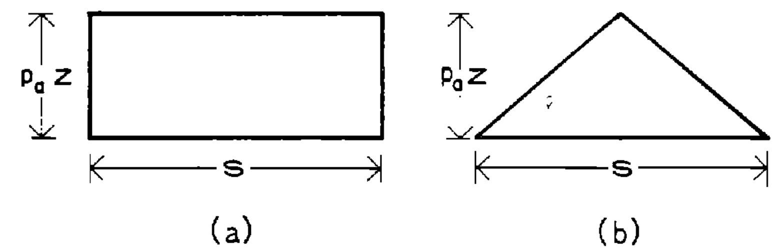
If the soldier piles are placed in predrilled holes and set with concrete, it is recommended that the composite action of the concrete and steel H-pile be ignored in the computation of the section modulus.

(11) If timber soldier piles are used, the shear stresses should be checked. One possible location of maximum shear is at point A (see figure 3-49). Shear stress at point A can be computed using

$$f_v = \frac{1.33\ P_h}{\pi\ r^2} \tag{3-83}$$

in which r is the radius of the pile at point A. The other point of possible maximum shear should also be checked.

(12) Compute the maximum bending moment in the lagging using the appropriate loading diagram (see figure 3-50).



s = Soldier pile spacing Z = Depth below top of wall

Figure 3-50.—Stress distributions on lagging.

The stress distribution shown in example (a) of figure 3-50 is recommended for fine-grained, cohesive soils. The stress distribution shown in example (b) of figure 3-50 is recommended for relatively clean granular soils that have been properly placed, and that have the ability to bridge and arch between soldier piles. For the uniform distribution shown in example (a) of figure 3-50, the maximum bending moment in the lagging should be computed using

$$M = 1/8\ p_a Z s^2 \qquad (3\text{-}84)$$

For the distribution shown in example (b) of figure 3-50, the maximum bending moment should be calculated using

$$M = 1/6\ p_a Z s \qquad (3\text{-}85)$$

in which Z is any distance down from the top of the pile, and s is the clear span distance between soldier piles. For intermediate soil conditions, combinations of the two loading distributions may be used and the resulting moments combined.

(13) Compute the required minimum section modulus, S_{min}, of the lagging using

$$S_{min} = \frac{M_{max}}{F_b} \qquad (3\text{-}71)$$

in which F_b is the allowable timber bending stress determined from the appropriate references.

(14) Dimension the timber lagging based on the value of S_{min} previously computed using

$$S_{\min} = \frac{bt^2}{6} \tag{3-86}$$

in which b is the unit length of the wall under analysis, and t is the lagging thickness. Generally, the unit length of wall under analysis, b, is equal to 12 inches.

31.3.2.4 Cantilever Soldier Pile Wall with Uniform Surcharge

Uniform surcharge conditions may be treated in a manner similar to that previously presented for the nonsurcharged soldier pile walls. Figure 3-51 shows the required earth pressure diagram for a cantilever soldier pile wall with a uniform surcharge. The stresses shown are per foot of wall, not per pile.

$$\sigma h = \Delta q K_a + p_a d \tag{3-72}$$

$$m = \frac{C(\Delta q K_a + p_a d)}{p_p - p_a} \tag{3-87}$$

$$P_h = \Delta q d\ K_a + \frac{p_a d^2}{2} + \frac{C(\Delta q K_a + p_a d)m}{2} \tag{3-88}$$

$$6P_h(h + x) - Cx^3(p_p - p_a) + \frac{[Cx^2(p_p - p_a) - 2P_h]^2}{C[2x(p_p - p_a) + Hp_p + K_p\Delta q - mp_a]} = 0 \tag{3-89}$$

in which C is the ratio of effective pile width to the center-to-center spacing of the soldier piles.

The equations for the depth below A to the point of zero shear (see equations 3-80, 3-81, and 3-82), the maximum bending moment, and section modulus remain unchanged. To complete the analysis, see steps 11 through 14 of the preceding section on cantilever soldier pile walls with lagging.

31.3.3 Anchored Walls

Generally, the purpose of an anchor system is to reduce the bending moment in a flexible structure or the overturning moment in a gravity or quasi-gravity wall.

Anchoring systems may have many different forms. This section will deal specifically with the stress distribution on the wall and the required analysis to size the major structural components. The design of the anchorage and the details of the various connections required will be discussed elsewhere.

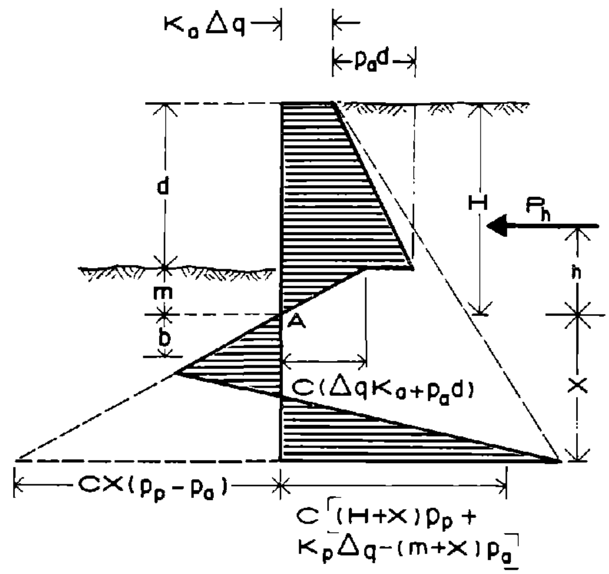


Figure 3-51.—Earth pressure diagram for cantilever soldier pile wall with uniform surcharge.

31.3.3.1 Anchored Sheet Pile Walls

When analyzing an anchored sheet pile wall, consider two cases: case 1, when the sheet pile has the minimum required depth of embedment (free-earth support method); and case 2, when the sheet pile has a depth of embedment greater than the minimum required (fixed earth support method). The free-earth support method assumes that the pile remains rigid and potential failure would involve rotation about the anchor rod. The fixed earth support method assumes that the bottom of the pile cannot rotate, and consequently, that there are two possible points of maximum moment in the pile. Depending on subsurface conditions and piling length, it may not be necessary to analyze both cases in all situations. For deep, soft soils with minimum pile lengths, designing for case 1 would be appropriate. On the other hand, case 2 would be appropriate for situations with more than minimum pile lengths, dense soils, or highly weathered rock conditions at the lower end of the pile. Many situations will fall somewhere between both cases and require analysis for both.

Figure 3-52 shows the earth pressure diagram for case 1, free-earth support method, with the anchor near the top of the pile.

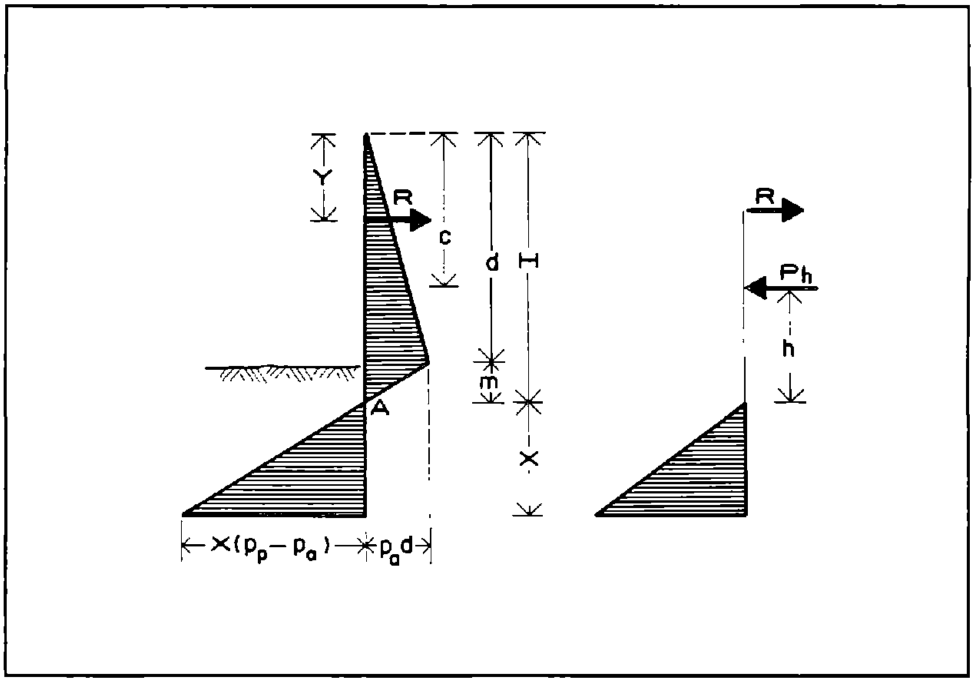


Figure 3-52.—Anchored sheet pile wall—case 1 (earth pressure distribution minimum depth of embedment).

In figure 3-52, the load, P_h, and the distances, m and h, are computed as though the wall were a cantilever wall. The following steps complete the analysis:

(1) Compute the minimum required depth of penetration, x, using

$$6\ P_h\ (H\text{-}h\text{-}Y) - (p_p - p_a)(3x^2H + 2x^3\text{-}3x^2Y) = 0 \qquad (3\text{-}90)$$

The equation is developed by summing the moments about the anchor, R.

(2) By knowing x and m, the remainder of the shear diagram may be completed.

(3) Compute the tension, R, in the anchor strand by summing the horizontal forces using

$$R = P_h - 1/2\ (p_p - p_a)\ x^2 \qquad (3\text{-}91)$$

(4) The maximum bending moments occur at the point of zero shear in the wall, below the anchor. The bending moment should also be evaluated at the anchor. With the consideration of the anchor force, R, the condition of zero shear exists at depth c. The depth c may be computed with equation 3-92 for

c between the top of the wall and depth d. For c between depths d and H, equation 3-93 would apply.

$$c = \sqrt{\frac{2R}{p_a}} \quad for\ 0 < c \leq d \tag{3-92}$$

$$c = H - \sqrt{\frac{2(P_h - R)}{p_p - p_a}} \quad for\ d < c \leq H \tag{3-93}$$

(5) The maximum bending moment at depth c may be computed using

$$M_c = R(c - Y) - \frac{c^3 P_a}{6} \tag{3-94}$$

and

$$M_c = R(c - Y) - \frac{p_a d^3}{6} - a\frac{p_a d}{2} - a^2\left[\frac{p_a d - a\,(p_p - p_a)}{6} + \frac{p_a d}{3}\right] \tag{3-95}$$

in which $a = \sqrt{\dfrac{2(P_h - R)}{p_p - p_a}}$

(6) Compute the required section modulus with equation 3-71 for the largest computed bending moment; select section from table 3-19 or any other listing of available sheet piling.

(7) Design required connections and anchor.

Figure 3-53 shows the earth pressure diagram for case 2, the fixed earth support method.

The analysis of case 2 is similar to case 1; however, equation 3-96 must be evaluated for either x or R; that is, assume one and solve for the other.

$$6P_h(h + x) = 6R(H + x - Y) - x^3(p_p - p_a) + \frac{[(p_p - p_a)x^2 - 2P_h + 2R]^2}{2(p_p - p_a)x + Hp_p - mp_a} \tag{3-96}$$

Equation 3-96 is the summation of moments about the base of the pile.

The maximum bending moment occurs at the point of zero shear. With the consideration of the anchor force, R, the condition of zero shear exists at depth c below R in addition to the depth b below point a (see figure 3-53). The depths b and c may be computed with equation 3-97 and 3-92, respectively.

$$b = \sqrt{\frac{2(P_h - R)}{p_p - p_a}} \tag{3-97}$$

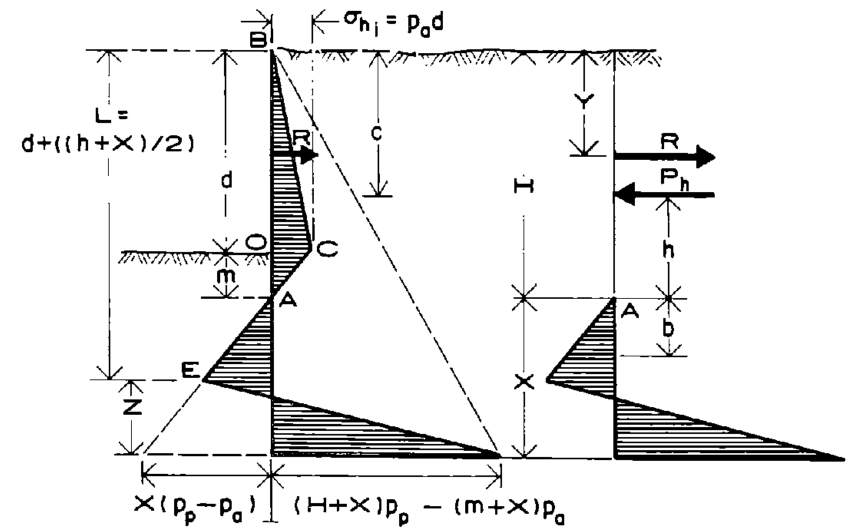


Figure 3-53.—Anchored sheet pile wall—case 2 (earth pressure distribution).

The maximum bending moments may be calculated with equations 3-94 and 3-98.

$$M_b = P_h(h+b) - R(b+m+d-Y) - \frac{b^3}{6}(p_p-p_a) \qquad (3\text{-}98)$$

The remainder of the solution is identical to case 1 (see steps 6 and 7).

31.3.3.2 Anchored Soldier Pile Walls

The analysis of anchored soldier pile walls is similar to that of anchored sheet pile walls, except for the inclusion of the area reduction factor, *C*, below the ground line. Value *C* is the ratio of effective pile width to the center-to-center spacing of the soldier piles (see table 3-20). As with the anchored sheet pile analysis, cases 1 and 2 should be evaluated.

Figure 3-54 summarizes the resulting earth pressure diagram for an anchored soldier pile wall (case 1) with a minimum depth of embedment (case 2, the free-earth support method). Development of the earth pressure diagram shown in figure 3-54 is initiated by repeating steps 1 through 5 of the cantilever soldier pile analysis previously presented.

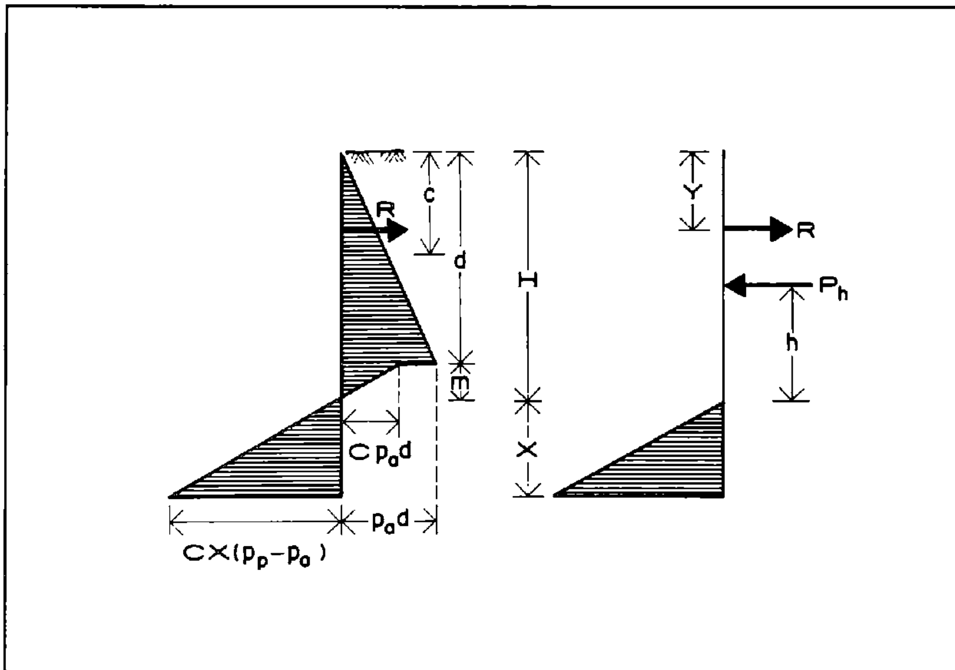


Figure 3-54.—Earth pressure diagram—case 1 (anchored soldier pile wall minimum depth of embedment).

Design stresses for piling and lumber depend on the size, shape, and species of the piece. Shape descriptions are shown in table 3-25. Also, the commonly used grades for various uses are presented. As there are several grades of timber for each use, and care must be taken when selecting structural members to assure that the proper grade is chosen. Higher grades than those shown in table 3-25 are available; however, they are expensive and frequently unavailable.

The following steps should be followed to complete the analysis:

(1) Compute the minimum embedment depth by solving equation 3-99 for x using the trial-and-error method. Equation 3-99 is developed by taking moments about R.

$$6P_h(H - h - Y) - C(p_p - p_a)(3x^2H + 2x^3 - 3x^2Y) = 0 \qquad (3\text{-}99)$$

(2) Complete the earth pressure diagram using the information developed in equation 3-99.

(3) Compute the tension in the anchor by summing the horizontal forces using

$$R = P_h - 1/C(p_p - p_a)x^2 \qquad (3\text{-}100)$$

Note that C is the ratio of the effective pile width to spacing.

(4) Compute the point of zero shear at depth *c* using

$$c = \sqrt{\frac{2R}{p_a}} \; for\ 0 < c \leq d. \tag{3-101}$$

(5) Compute the maximum bending moment at depth *c* and at the anchor location using

$$M_c = R(c - y) - \frac{c^3 p_a}{6} \tag{3-102}$$

(6) Compute the minimum section modulus with equation 3-82 (recall that this is a per foot analysis of the wall) and select a section from the steel handbook.

$$S_{\min} = \frac{M_{\max}}{F_b} \, s \tag{3-82}$$

(7) Design required connections and anchorage.

Figure 3-55 shows the earth pressure diagram for anchored soldier pile walls (case 2) that is greater than minimum depth of embedment. The analysis of case 2 is similar to case 1, except that equation 3-103 must be evaluated for either *x* or *R*; that is, assume one value and solve for the other.

$$6P_h (h + x) - 6R(H + x - Y) - x^3(P_p - P_a)\,C + \frac{[C(p_p - p_a)x^2 - 2P_h + 2R]^2}{C[2(p_p - p_a)x + Hp_p - mp_a]} = 0 \tag{3-103}$$

Equation 3-103 is the summation of moments about the pile tip.

The maximum bending moment occurs at the point of zero shear. With the consideration of the anchor force, R, the condition of zero shear exists at depth *c* below *R* in addition to the depth *b* below point *A* (see figure 3-55). The depths *b* and *c* may be computed with equations 3-104 and 3-101, respectively.

$$eb = \sqrt{\frac{2(P_h - R)}{(p_p - p_a)C}} \tag{3-104}$$

The maximum bending moments may be calculated with equations 3-102 and 3-105.

$$M_b = P_h(h + b) - R(b + m + d - Y) - \frac{b^3}{6}(p_p - p_a)C \tag{3-105}$$

The remainder of the solution is identical to case 1.

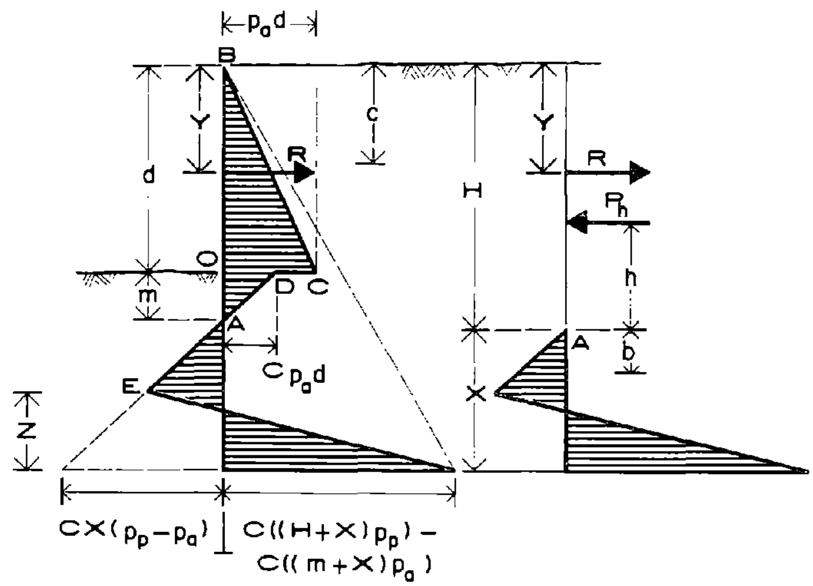


Figure 3-55.—Anchored soldier pile wall—case 2 (earth pressure distribution).

3I.3.3.3 Gravity Structures

The use of tieback anchors with gravity walls generally involves existing structures that require upgrading in order to meet new loading requirements or to improve the stability of a marginal structure. If a new wall were being designed and tieback anchors were required, some wall type other than a gravity wall would generally be considered.

If an existing structure is being upgraded, for example the wall presented in figure 3-37, a complete re-analysis of the stability of the wall is required. In addition to the forces shown on figure 3-37, the anchor forces should be included. If the anchor is not horizontal, the vertical component of the anchor force must also be included in all analyses.

As shown in figure 3-37, the external stability would require the closest study. If the example wall had been a concrete cantilever structure, the internal stability most likely would be more critical.

3J Rockeries

A rockery or rockery wall is built to provide stability, erosion control, and a decorative look for slopes. While stability is provided to the slope, the amount is difficult to calculate. If a slope stability problem exists, a geotechnical engineer should be contacted for further recommendations; otherwise rockeries should be used only where minor support is needed at the slope face. The amount of support obtained will depend to a large extent on the quality of the workmanship, size, and shape of the rocks used. The term rockery used here refers to rock that is placed, not dumped. Rockeries up to 20 feet high are seen, however most failures occur in those exceeding 15 feet in height. It is recommended that rockery heights be limited to less than 15 feet. Examples of rockeries are shown in figures 3-56 and 3-57.

Information in this section pertains to the design and construction of rockeries. For further information the reader should refer to *Standard Rock Wall Construction Guidelines* in the bibliography.

3J.1 Design

Many rockeries are not designed; they are just built. However, there is some design information available in the references. A rockery is not an engineered system in the sense that a retaining wall is. If a slope stability problem exists, a geotechnical engineer should be consulted and possibly another method such as a retaining wall or buttress should be used.

Figure 3-58 shows a design chart developed for Shannon and Wilson geotechnical consultants and described by Gifford and Kirkland (2). This chart gives critical height-to-width ratios for rockeries on soils with friction angles of 30 and 35 degrees. There are curves for well-constructed rockeries (WCR) and poorly constructed rockeries (PCR) for each soil parameter. In practice it is recommended that the curves for PCR be used for design.

By knowing the friction angle of the soil and the angle of inclination of the slope face, the critical height/width (Hc/B) ratio can be determined. In practice the maximum recommended heights of rockeries are 12 to 15 feet at slope angles of 1H:4V to 1H:6V. If a higher wall is needed, it is recommended that two walls are built one on top of the other. The total wall in this case needs to be checked for overall stability.

A check on sliding and global stability should be done especially in cases where there are unusual loadings, such as heavy traffic close to the top of the rockery.

Rockeries are used for both cut-and-fill slopes. Figures 3-59 and 3-60 show typical details for both situations. Figure 3-61 shows typical details for a rock wall in front of a reinforced fill.

3J.2 Materials

A critical factor in rockery construction is the quality of the rock material used. All rock should be sound angular with the longest dimension not exceeding three times the width. All rock should meet the following minimum specifications:

• Absorption ASTM C127 AASHTO T-85	No more than 2.0 percent for igneous and metamorphic rock types, and 3.0 percent for sedimentary rock types.

- Accelerated expansion (15 days) CRD-C-148 *1,*2 — No more than 15 percent breakdown.

- Soundness (MsSO4 at 5 cycles) ASTM C88 CRD-C-137 — No greater than 5 percent loss.

- Unconfined compressive strength ASTM D 2938 — Intact strength of 6,000 psi or greater.

- Bulk specific gravity ASTM C127 AASHTO T-85 — Greater than 2.48 (155 pcf).

*1. The test sample will be prepared and tested in accordance with the Corps of Engineers testing procedure CRD-C-148, "method of testing stone for expansive breakdown on soaking in ethylene glycol."

*2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock.

Rocks used in rockery construction are frequently sized as "man rocks." For example, a two-man rock is a rock that can be placed by two men using steel pry bars. Sizes of rocks commonly used are shown in table 3-21.

Table 3-21.—Sizes of rocks commonly used.

Rock size	Rock weight	Average dimension
One-man	50–200 pounds	12–18 inches
Two-man	200–700 pounds	18–28 inches
Three-man	700–2000 pounds	28–36 inches
Four-man	2000–4000 pounds	36–48 inches
Five-man	4000–6000 pounds	48–54 inches
Six-man	6000–8000 pounds	54–60 inches

3J.3 Construction

The competency of a rockery depends on the skill and experience of the builder. Rockeries are not retaining walls, although some degree of retention is provided.

Rockeries can be used with cuts up to 12 to 15 feet high. Figure 3-59 is a typical detail for rockeries built in a cut section. For cuts greater than 8 feet in height a geotechnical engineer should be consulted.

There are two methods of constructing rockeries against fills. The first is generally used with fills 8 feet or less in height. For this method the fill is overbuilt to get good compaction and then excavated back to the desired face location (see figure 3-60).

For fills greater than 8 feet in height, the fill is first constructed using geotextile reinforcement as shown in figure 3-61, and then the rockery is constructed against the face. For design of the reinforced fill see the corresponding section of this guide.

Drainage and compaction requirements are shown on the figures. As with any drainage system, design should consider filter criteria. Appropriate geotextile or granular filter should be used between backfill and drain rock and between rockery and drain rock if needed. See the section in this guide on drainage.

Rock for use in rockeries should be cubical, rectangular, or tabular in shape. The economics of rockeries depends on having a close source of rock with the right shape, size, and quality available. If not readily available, then alternatives should be considered. Rounded rocks should not be used, except for filling voids. Rocks should be placed in a stable condition. This may require the contractor to shift rocks around to get the best fit. Voids should be chinked with smaller rock. Important construction details are shown on figures 3-59 to 3-61.

A third type of rockery was used on a trail in the San Juan National Forest, as shown on figures 3-62 and 3-63 (3). Because the work was done by hand and wall heights were low, smaller rocks were used. Here, the bottom course of rock is placed and backfilled. The geotextile is then placed on the backfill and extended over the first course of rock. The next row of rock is placed on top of the geotextile, and the next soil layer is placed and compacted. The process is repeated until the full wall height is reached. It is important that the rock is stable when stacked so that a stable face is achieved. As for the other rockeries, the toe should be keyed into the subgrade, and adequate drainage should be provided. The reinforcement, as for any other wall, must be designed.

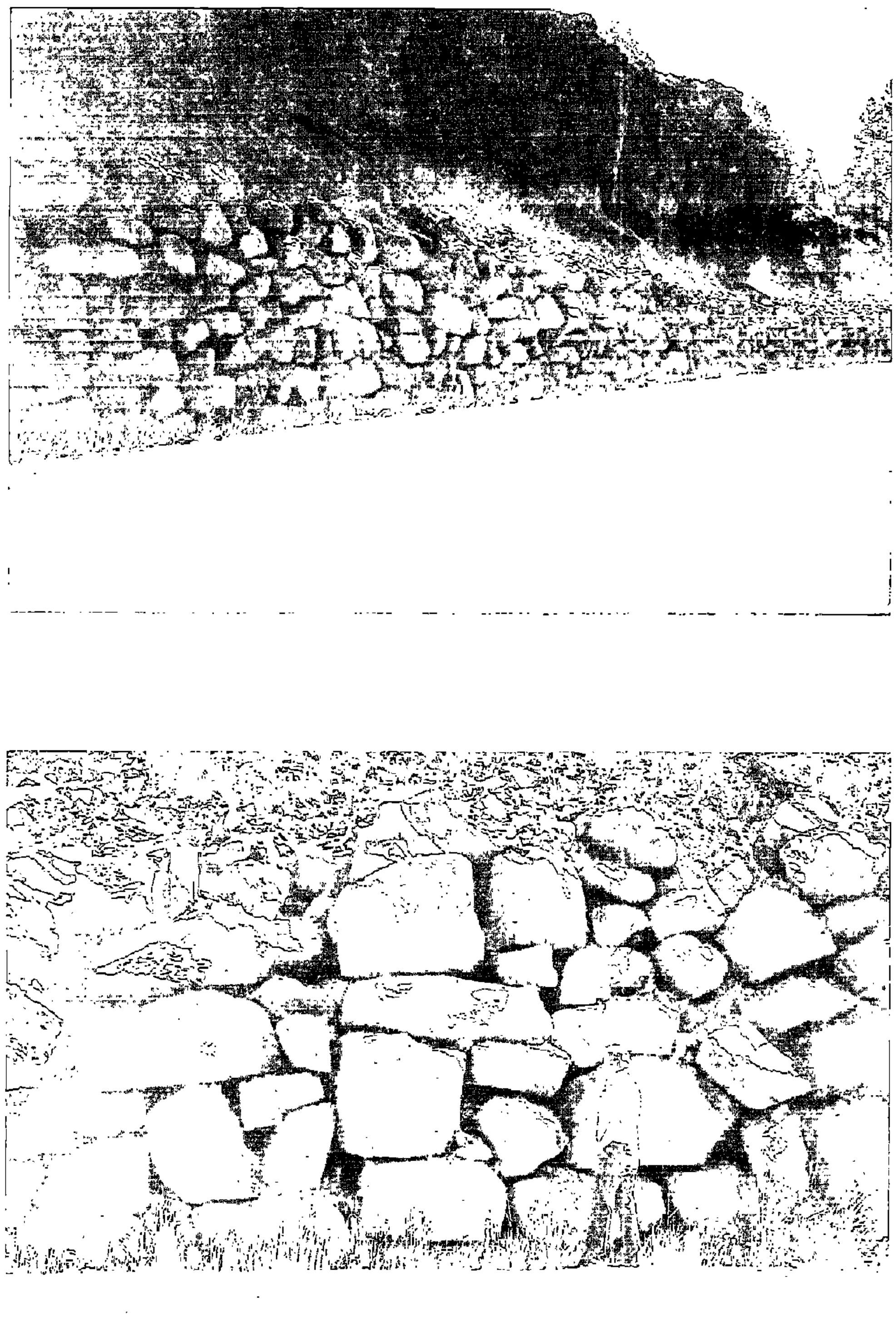
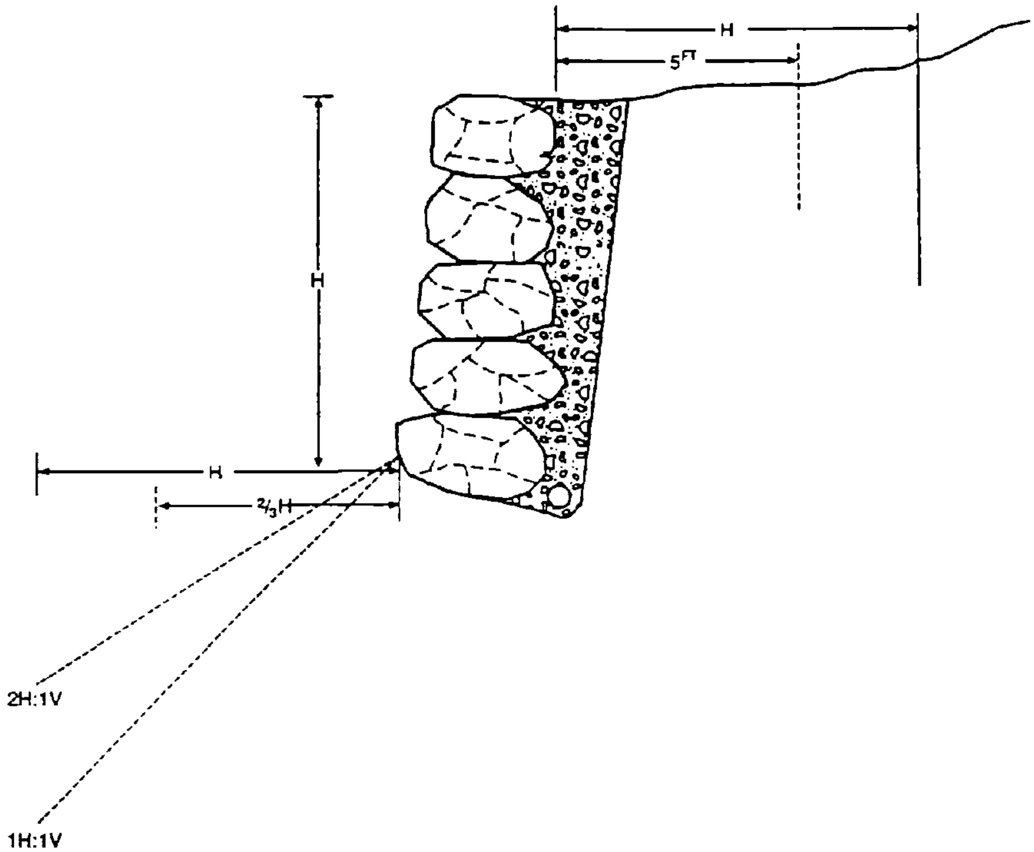


Figure 3-56.—Rockery.



Note: $H=H^1=H^2$

H	Height of rock wall
5 ft	No temporary excavation or trenching
2/3 H	No temporary excavation or trenching
H^1	No permanent removal of subgrade allowed
H^2	No excavation of material or any surcharge allowed
1H:1V	No excavation below allowed
2H:1V	Maximum finished grade or permanent excavation

Figure 3-57.—Postconstruction guideline (after Arc Manual).

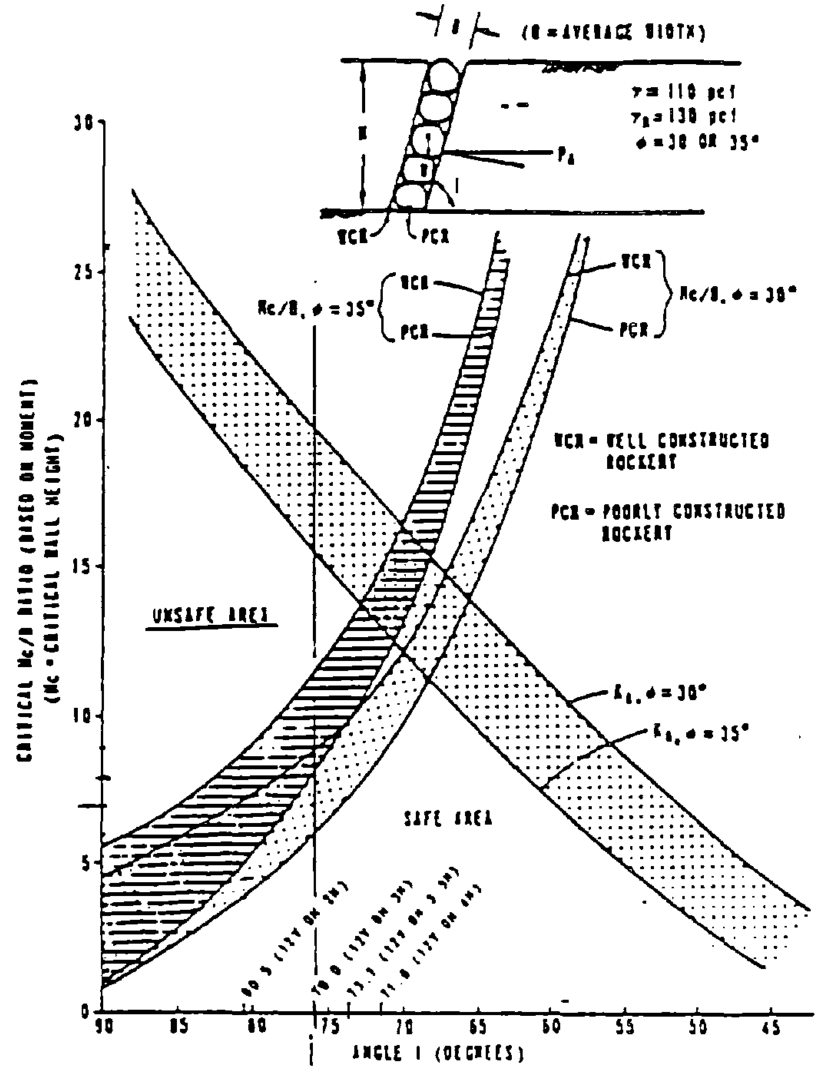
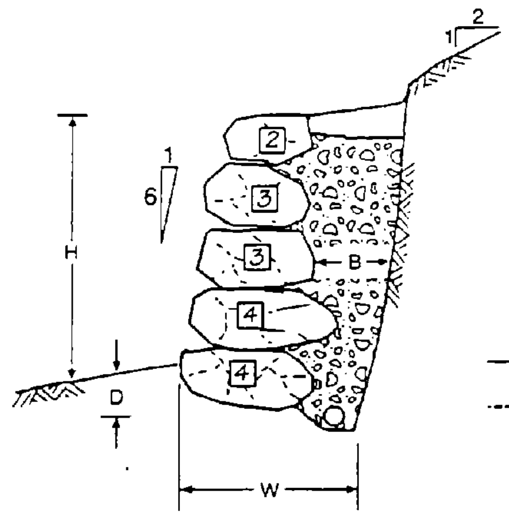
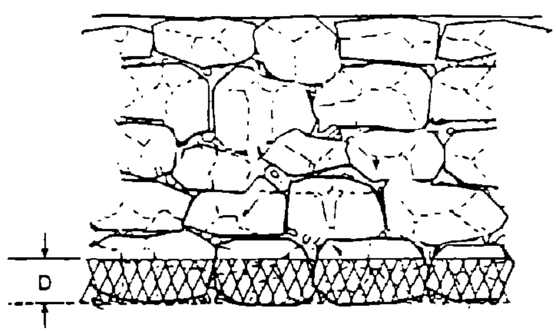


Figure 3-58.—Critical Hc/B ratios.

Schematic Only - Not to Scale



Rock Wall Section



Rock Wall Elevation

NOTES:

- Rock wall construction is a craft and depends largely on the skill and experience of the builder.
- A rock wall is a protective system which helps retard the weathering and erosion process on an exposed soil face.
- While by its nature (mass, size and shape of the rocks) it will provide some degree of retention, it is not a designed or engineered system in the sense a reinforced concrete retaining wall would be considered designed or engineered.
- The degree of retention achieved is dependent on the size of the rock used; that is, the mass or weight, and the height of the wall being constructed. The larger the rock, the more competent the rock wall should be.
- Rock walls should be considered maintenance items that will require periodic inspection and repair. They should be located so that they can be reached by a contractor if repairs become necessary.
- Maximum inclination of the slopes above and behind rock walls should be 2:1 (Horizontal:Vertical)
- Minimum thickness of rock filter layer B = 12 inches. Minimum embedment D = 12 inches undisturbed native soil or compacted fill placed in accordance with report recommendations.
- Maximum rock wall height H = ______ feet.
- Rock walls greater than 8 feet in height to be installed under periodic or full time observation of the geotechnical engineer.
- Rock should be placed to gradually decrease in size with increasing wall height in accordance with geotechnical engineers recommendations.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

- The long dimension of the rocks should extend back towards the cut or fill face to provide maximum stability. Rocks should not be stacked like shoe boxes. They should be placed to avoid continuous joint planes in vertical or lateral directions. Whenever possible each rock should bear on two or more rocks below it, with good flat-to-flat contact.
- All rock walls over 4 feet in height should be constructed on basis of wall mass, not square footage of face.

Size	Approximate Weight - lbs.	Approximate Diameter
1 Man	50 - 200	12 - 18"
2 Man	200 - 700	18 - 28"
3 Man	700 - 2000	28 - 36"
4 Man	2000 - 4000	36 - 48"
5 Man	4000 - 6000	48 - 54"
6 Man	6000 - 8000	54 - 60"

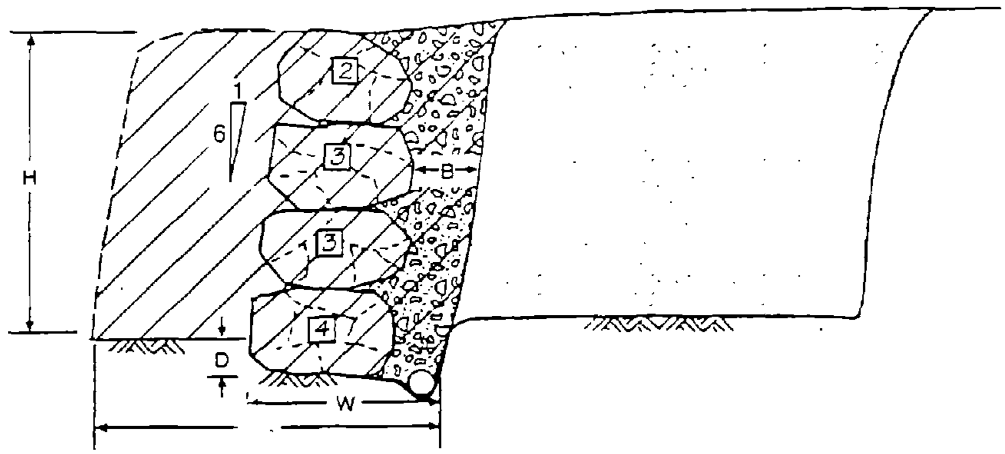
Reference: Local quarry weight study using average weights of no less than six rocks of each man size conducted in January 1, 1988

LEGEND

(drainage symbol)	Drainage materials to consist of clean angular 4 to 2 inch spalls, or other material approved by the geotechnical engineer
(blank box)	Surface seal: may consist of impervious soil or a fine free draining granular material
(hatched box)	Undisturbed firm Native soil
○	Drainpipe: 4-inch minimum diameter, perforated or slotted rigid plastic ADS pipe laid with a positive gradient to discharge under control well away from the wall
$\boxed{4}$	Designates size of rock required, i.e. __4__ man

Figure 3-59.—Typical detail native cut of any height over 4 feet (after Arc Manual).

Schematic Only - Not to Scale



LEGEND

Crushed rock or other approved material ranging between 4 to 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sieve).

Structural fill overbuild, compacted to at least 95% of maximum dry density as determined by ASTM Test Method D-1558-78 (Modified Proctor).

Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to at least 95 percent of ASTM D-1557-78 maximum dry density.

Undisturbed firm Native soil

Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.

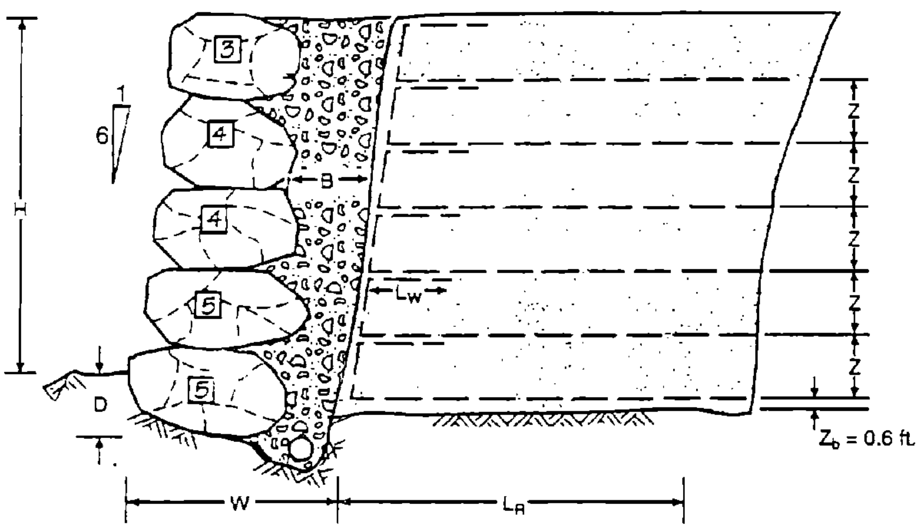
4 Designates size of rock required, i.e. 4 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Thickness of crushed filter rock layer, B, should be no less than 12 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches.
- Height of rock wall, H, should not exceed ______ feet.
- Lateral extent of fill overbuild, L_0 should be no less than H feet.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

Figure 3-60.—Typical detail overbuild fill construction rock wall less than 8 feet in height (after Arc Manual).

Schematic Only - Not to Scale



LEGEND

Crushed rock or crushed concrete drain rock material ranging between 4 and 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sieve).

Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to a least 95 percent of ASTM D-1557-78 maximum dry density.

Undisturbed firm Native soil

Geogrid reinforcement approved by geotechnical engineer.

Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.

5 Designates size of rock required, i.e. 5 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Minimum length of geogrid wrap over top of fill, L_W, should be no less than 3 feet.
- Length of reinforcing geogrid, L_R shall be ______ feet
- Geogrid reinforcement layer spacing Z shall be ______ feet as determined by the geotechnical engineer's design
- Height of rock wall, H, should not exceed ______ feet
- Thickness of crushed drain rock layer, B should be no less than 18 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B

Figure 3-61.—Typical detail geogrid reinforced fill construction rock wall 8 feet or more in height (after Arc Manual).

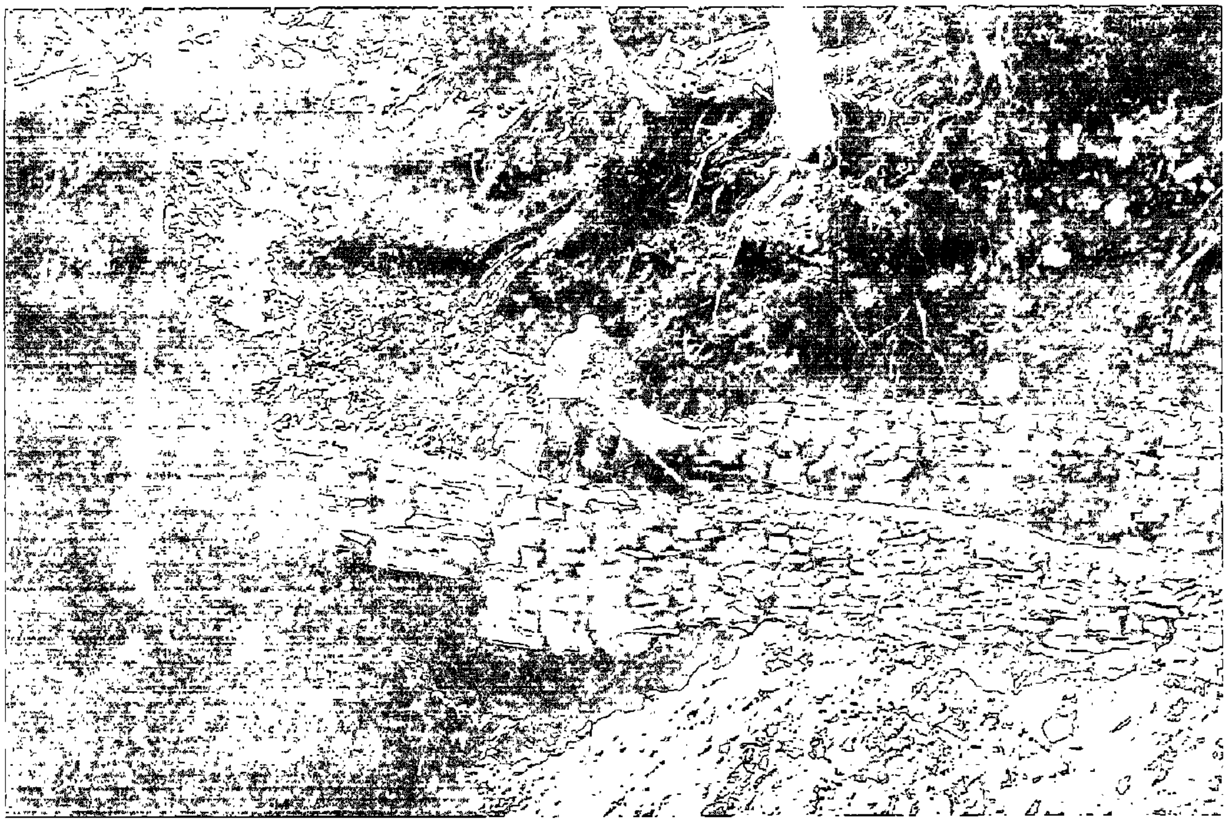


Figure 3-62.—Rockery retaining walls on a trail.

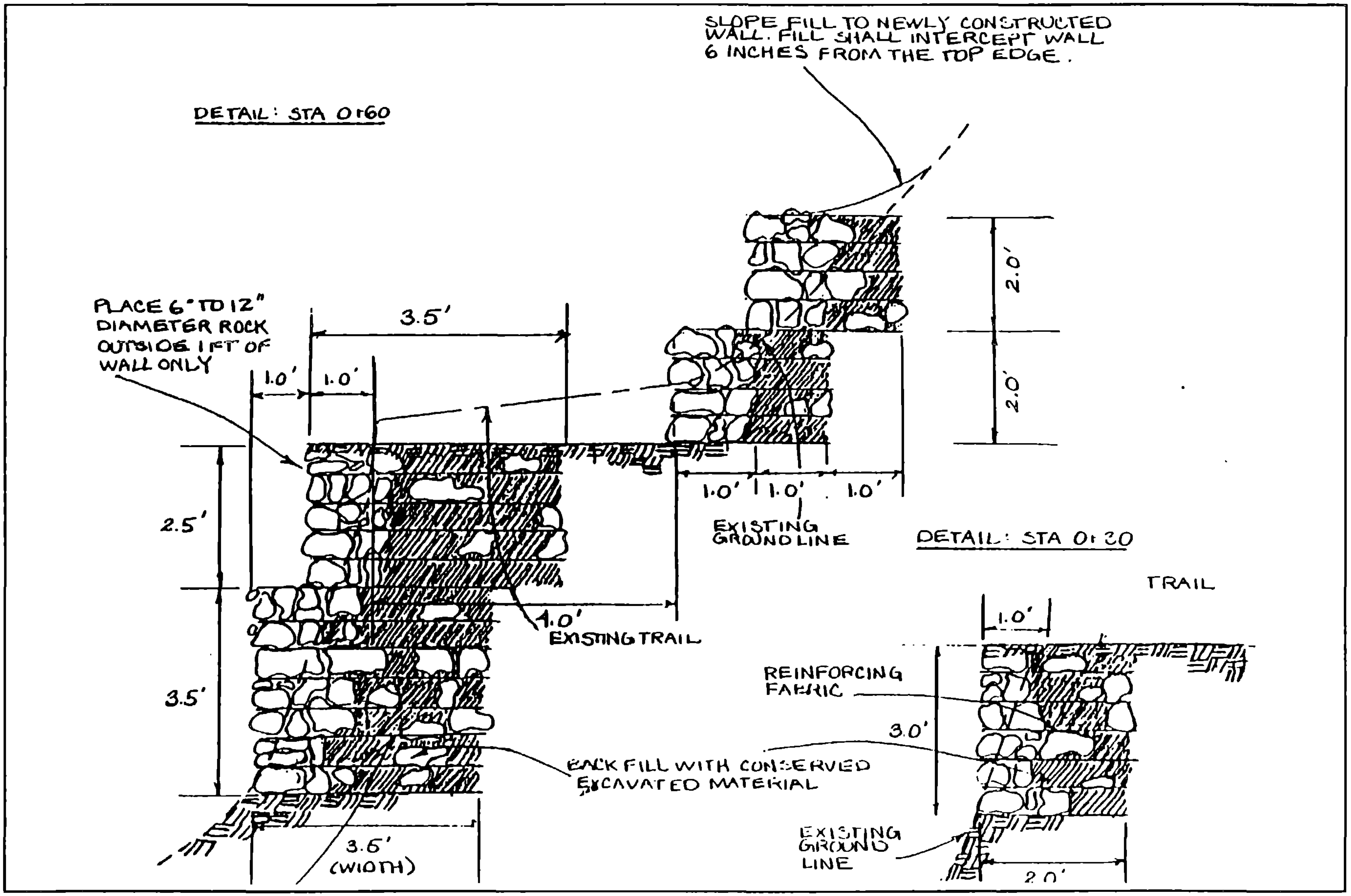


Figure 3-63.—Rockery retaining walls on a trail (section view).

3K Reinforced Soil Retaining Walls

3K.1 General

This section provides guidelines for the design of reinforced soil retaining walls (also known as mechanically stabilized earth (MSE) walls) using layers of polymeric (such as geogrids) and metallic (such as welded wire mesh) reinforcement. This design method is applicable to a variety of soil reinforcement and wall facing types. To name a few, some of the more familiar wall types include proprietary systems by Tensar Corporation, Hilfiker, Keystone, and the Reinforced Earth Company.

Because this guide is intended to be used by a general civil engineer, the size and complexity of the walls addressed by these guidelines are limited. The assumptions and limitations are presented in section 3K.4. Designers of critical walls or complex systems should consult with a qualified geotechnical engineer. For additional study, references have been presented throughout the text.

Since the design methods presented here are applicable for a wide variety of wall types, the results could be conservative for a particular product or type of material. Also, since manufacturers often have design assistance available, users can frequently request that a design be reviewed by a registered engineer representing a particular manufacturer. On some projects, the manufacturers can also provide plans prepared by a registered engineer together with certifications of material properties.

Sections 3K.2, 3K.3, and 3K.4 show a general overview of reinforced soil retaining wall design, a discussion of the current state of the practice in design, and a listing of the assumptions and limitations used for this guideline, respectively. Design methodology and design calculations are presented thereafter.

3K.2 Elements of a Reinforced Soil Retaining Wall

Reinforced soil retaining walls act as gravity wall structures that depend on the mass of the reinforced soil zone behind the wall facing to resist destabilizing forces imposed by the retained backfill and other loads such as surcharges. Reinforcing elements are placed in horizontal layers within the backfill soil zone to create a composite reinforced soil mass. The reinforcement provides a tensile strength within the soil mass that allows the composite structure to maintain its integrity over the specified design life.

A general cross-section of a reinforced retaining wall is shown in figure 3-64. The primary components of a reinforced retaining wall include reinforcement, wall facing, reinforced soil backfill zone, retained soil zone, foundation soil zone, and drainage features.

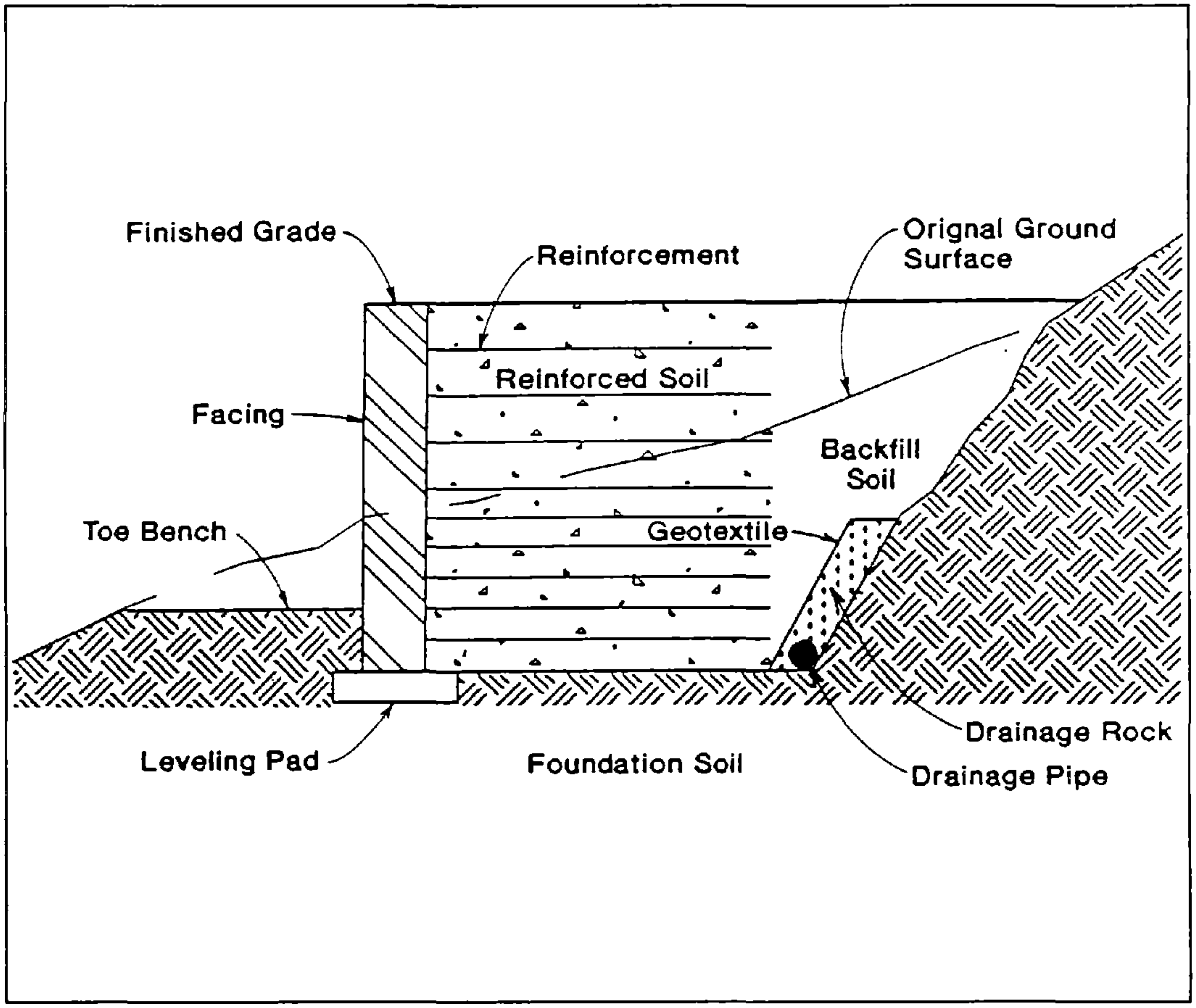


Figure 3-64.—Reinforced soil wall cross-section.

Other design considerations include the finished grades above and below the wall, surcharge loads, original site grade, limits of construction excavation, and leveling pad.

3K.3 Current State of the Practice

The state of the practice for design of reinforced retaining structures continues to evolve primarily because of increased understanding of the long-term behavior of existing walls. At the same time, manufacturers are developing new types of reinforcements and wall facing materials.

Several design manuals and design guides have been published in the last decade. Also, several manufacturers have produced design manuals that are product-specific. All these guidelines contain a tremendous amount of information, but there is a continuing need within the design community to develop a comprehensive design approach for evaluating the entire class of reinforced soil walls. Having recognized this, the FHWA is producing a design manual and a design software for reinforced structures. Other agencies have formed committees, such as the TRB A2K07 subcommittee, which is producing a document on the durability of geosynthetics.

The overall design process for reinforced soil retaining walls is very similar to that used for conventional steel-reinforced concrete retaining walls. The evaluation of the issues affecting external stability of reinforced soil walls, such as overturning stability, sliding stability, and bearing pressure, is virtually the same as for conventional retaining walls. The significant difference is the design approach for internal stability of the wall. In conventional concrete retaining wall design, the designer must specify the layout of the steel reinforcing bars, but in reinforced soil wall design, the designer specifies the layout of the reinforcing layers (type, length, and spacing of the reinforcing elements).

Generally, the various reinforced soil retaining wall systems (metallic-reinforced, polymer-reinforced, etc.) use the same potential modes of failure, soil pressures, and analytical methods for external stability. These systems do not necessarily use the same soil pressures, reinforcement strengths, and design methods for internal stability. Also, the greatest barriers to a comprehensive and consistent design method for evaluating internal stability are the issues of the actual load transferred to the reinforcements and the long-term design strength of the reinforcing elements. Also, no standards have been proposed that encompass all reinforcement types. The actual stress distribution for the reinforcing elements may not be known until long-term instrumentation programs are reported. The probable factors affecting long-term design strength, such as creep, durability, and survivability, have been identified. But some of the oldest structures are only about 30 years old.

3K.4 Assumptions and Limitations

The use of this design guide is limited by the following assumptions on reinforced retaining wall geometry, site conditions, and use. In cases where these limitations cannot be met, the design should be directed by a qualified geotechnical engineer.

(1) The structure is a noncritical temporary or noncritical permanent structure. Guidelines adapted from Allen (1991) limit noncritical structures to those meeting the following requirements:

- Design life less than 75 years.
- Wall height less than 16 feet.
- Loss of life risk resulting from failure is very low to low.
- The wall does not support other structures.

704.10 Select Granular Backfill. Select granular backfill is sound, durable, granular material free from organic matter or other deleterious material (such as shale or other soft particles with poor durability) and meeting the gradation in Table 704-4. The material shall also conform to the following quality and electrochemical requirements:

(a) Quality requirements.

(1)	Shear angle of internal friction, AASHTO T 236	≥ 34° (0.6 rad)
(2)	Sodium sulfate soundness loss (5 cycles), AASHTO T 104	15% max.
(3)	Los Angeles abrasion, AASHTO T 96	50% max.
(4)	Plasticity index, AASHTO T 90	6 max.

Note: Compact samples for AASHTO T 236 to 95% of the maximum density determined according to AASHTO T 99, Method C or D, and corrected for oversized material as set forth in Note 7 of AASHTO T 99.

(b) Electrochemical requirements.

(1)	Resistivity, AASHTO T 288, any method	3,000 Ω-cm minimum
(2)	pH, AASHTO T 289, any method	5.0 - 10.0
(3)	Sulfate content, AASHTO T 290, any method	1,000 ppm maximum
(4)	Chloride content, AASHTO T 291, any method	200 ppm maximum

Note: Tests for sulfate and chloride content are not required when pH is between 6.0 and 8.0 and the resistivity is greater than 5,000 Ω-cm.

Table 704 - 4
Select Granular Backfill Gradation

Sieve Size	Percent by Weight Passing Designated Sieve (AASHTO T 27 and T 11)
4 inch (100 mm)	100
3 inch (75 mm)	75 - 100
No. 200 (75 μm)	0 - 15

Figure 3-65.—FHWA FP-92 specifications.

(2) Soil conditions at the wall site are well documented, or a qualified geologist or geotechnical engineer has characterized site soils with the following results:

- The foundation soils have adequate bearing capacity to support the wall, including the influence of a slope beneath the toe of the wall, and they are relatively incompressible.

- The wall is not on or in the immediate vicinity of an existing landslide.

- Scour or erosion at the toe will not undermine the wall.

- The wall does not need to be analyzed for seismic loading conditions.

- Ideal soils used in the backfill and reinforced zones are free-draining granular materials conforming to section 704.10 of the FHWA FP-92 Standard Specifications (figure 3-65). Often, more fine-grained, marginal materials are used. See the discussion in Section 3E, "Backfill Considerations". If materials are not free-draining, provide a drainage system designed according to recommendations in the section on drainage in chapter 3.

- Soil conditions in the foundation zone, reinforced zone, and retained zone are approximately uniform along the profile of the wall.

(3) The wall has level backfill conditions or sloping and broken backfills can be modeled as a uniform surcharge of less than 500 psf (pounds per square foot).

(4) The wall has level toe conditions for a minimum distance of one-half the height of the wall.

(5) Surcharges are less than about 500 psf and are uniform.

(6) The same length of reinforcement will be used throughout the full height of the wall.

(7) The wall length will not exceed approximately 1,000 feet. The conservative assumptions in this manual can result in uneconomical designs for large walls and complex projects.

(8) Utilities and other subsurface lines will not be constructed within the reinforced zone.

3K.5 Design Methodology

The basic steps in the design of a reinforced retaining wall are:

(1) Satisfy design assumptions.

(2) Generate the wall profile.

(3) Determine the wall geometry, loading conditions and soil parameters for each cross-section to be analyzed.

(4) Complete the external stability analysis.

(5) Select the reinforcement.

(6) Select the facing.

(7) Complete the internal stability analysis.

(8) Develop construction drawings and specifications.

Step 1: Satisfy Design Assumptions

The design assumptions and limitations listed in 3K.4 should be met.

Step 2: Generate Wall Profile

A wall profile should be generated based on existing and proposed site grades at the wall. An example wall profile is shown in figure 3-66. The wall profile should include the final elevation at the top and the bottom of the wall, as well as the finish grades behind the top of the wall and in front of the toe. Most projects involve some variation in height along the length of the wall. Choose a convenient height interval, such as 2 feet, to develop design panels that have consistent cross-sections along the wall length. In gentle terrain (less than about 3 percent slope), the top of the wall can be parallel to the road grade.

Step 3: Establish Wall, Loading, and Soil Parameters

Wall geometry and loading conditions should be established for each of the wall panels generated for the wall profile described in step 2. The required wall and soil parameters are shown in figure 3-67, and listed as follows:

<u>Required wall parameters</u>

H = Total wall height (limited to 16 feet)

D = Wall embedment (normally 2.0 feet minimum for walls with solid facings such as concrete blocks. Normally 0 feet for walls with flexible facings, such as geotextile and wire mesh walls)

q = Uniform surcharge load (limited to 500 psf)

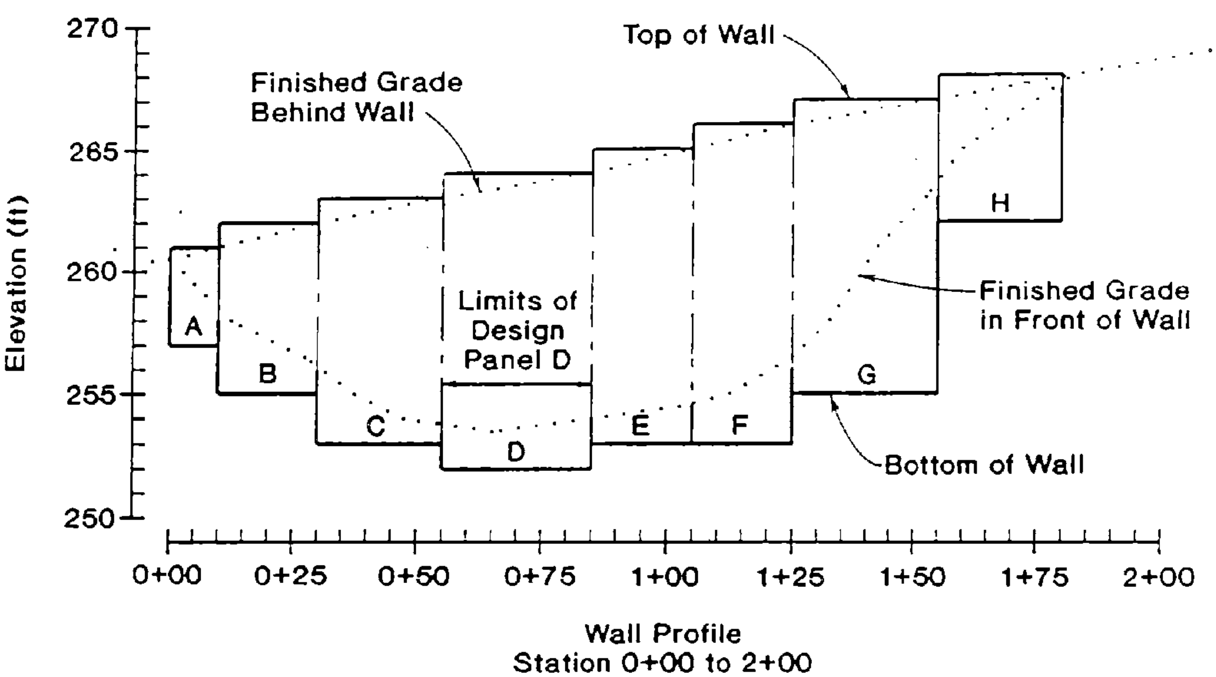


Figure 3-66.—Reinforced soil wall profile.

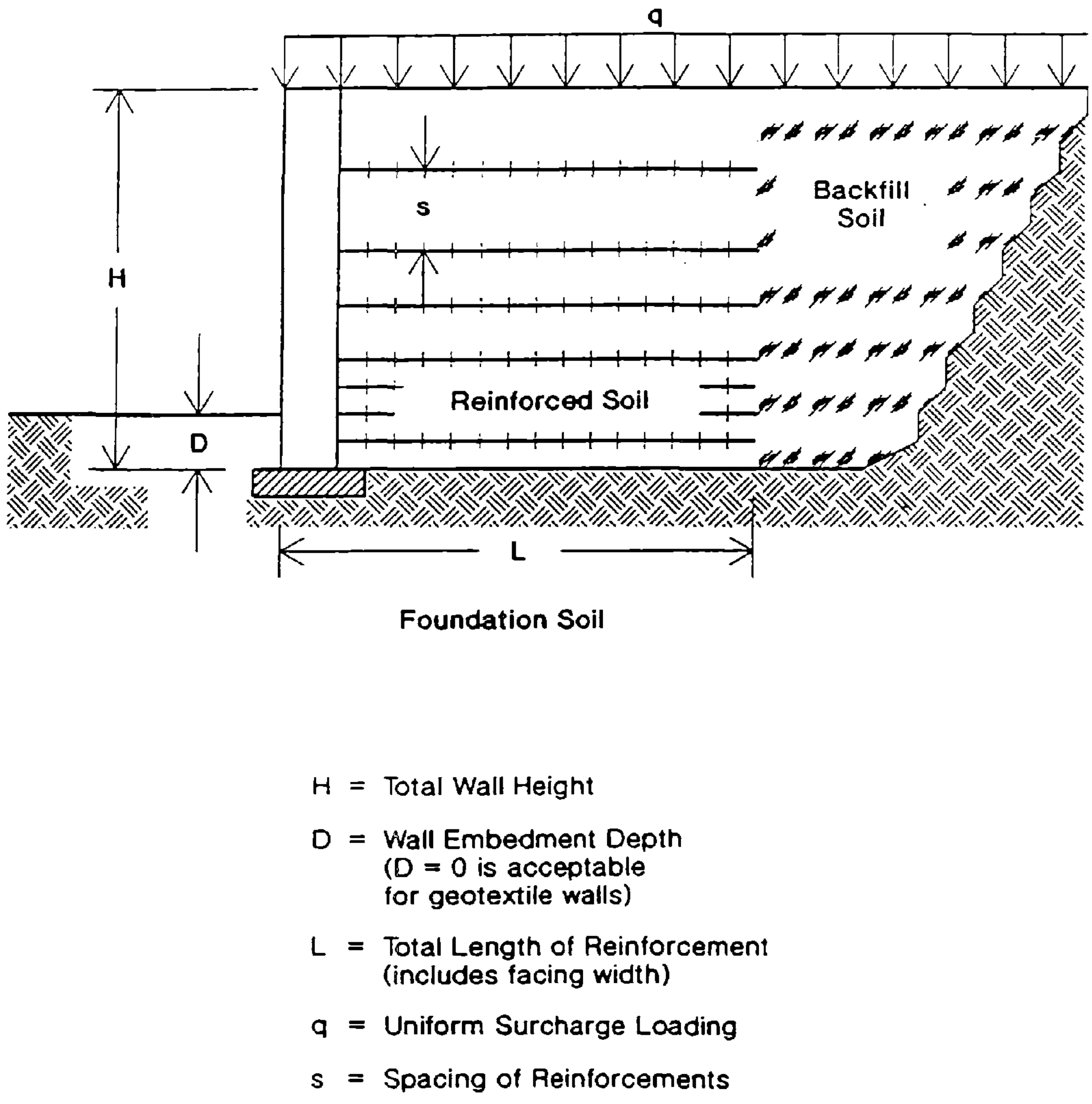


Figure 3-67.—Reinforced soil wall definition sketch.

Common live surcharge values for traffic on pavement:

$q = 50$ psf (automobile parking only)

$q = 100$ psf (car and light truck traffic)

$q = 250$ psf (tractor trailer traffic or fire lanes)

The following soil parameters should be determined for the wall:

Soil zone	Moist unit weight (pcf)	Internal friction angle (degrees)	Cohesion (psf)
Reinforced soil	γ_r	ϕ_r	0
Backfill soil	γ_b	ϕ_b	0
Foundation soil	γ_f	ϕ_f	C_f

pcf = pounds per cubic foot

The design soil parameters should be based on a site geotechnical investigation. The soil strength values should be effective stress parameters based on long-term (consolidated-drained) conditions. In this case, the cohesion intercept will be zero, which is consistent with conventional retaining wall design.

Step 4: Complete External Stability Analysis

External stability analyses ensure that the reinforced structure is stable against the action of the lateral pressures applied by the retained backfill. This analysis is the same as stability analyses completed for the design of gravity retaining walls. There are three modes of failure to be considered: sliding, overturning, and bearing capacity. These modes of failure are shown and described in figure 3-68. A fourth potential mode, global or overall stability, should also be addressed as discussed in Chapter 3, "External Stability."

Step 5: Select Reinforcement

The three predominant types of reinforcements currently in use are strip, grid, and sheet reinforcements.

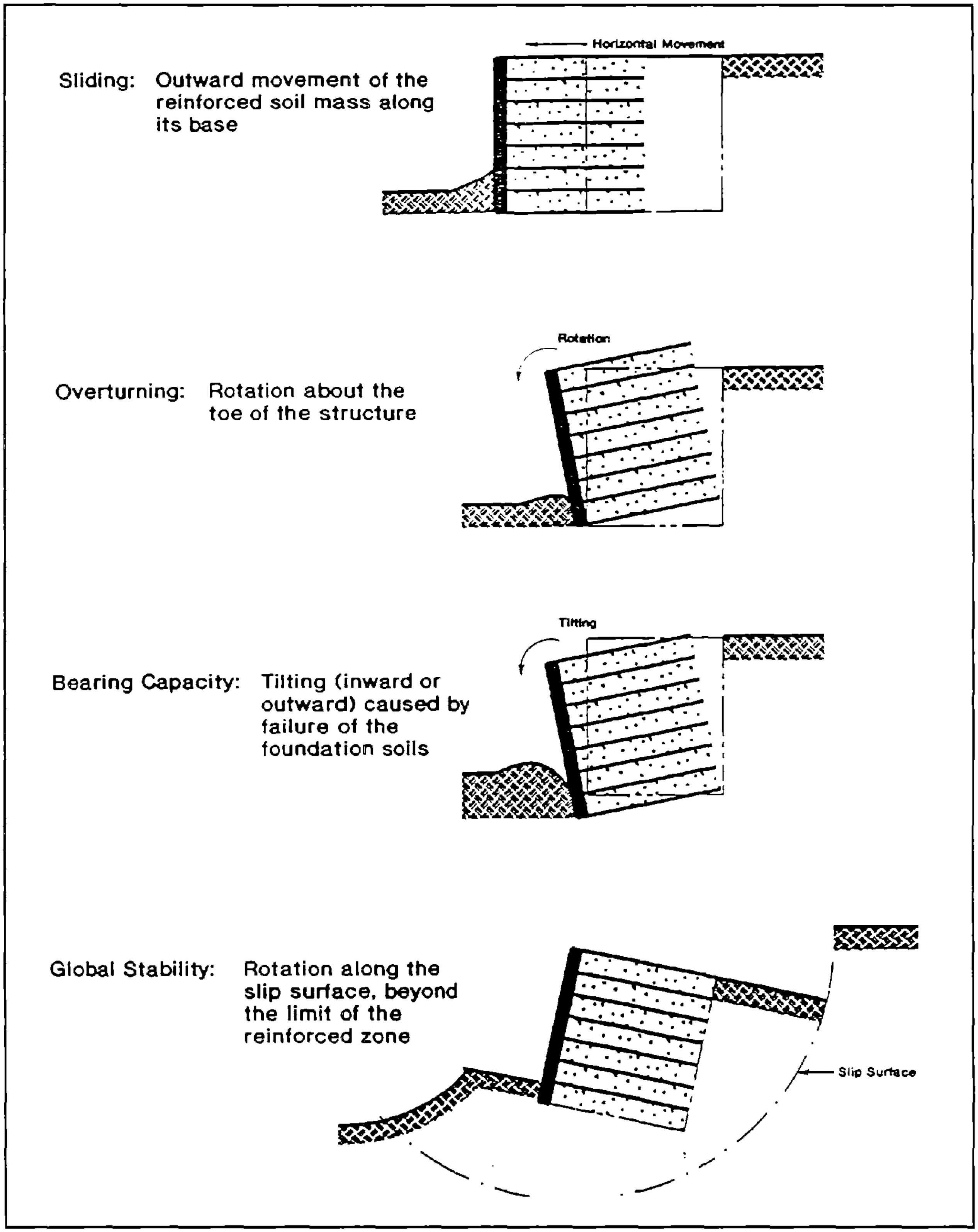


Figure 3-68.—Modes of external stability failures.

They differ according to geometry, stress transfer mechanisms, and extensibility of the reinforcement, as shown in table 3-22. Due to their different behavior, reinforcements are generally characterized as extensible or inextensible. The extensibility of the reinforcements compared to the deformability of the fill is an essential feature of the behavior of the wall, as it controls the state of horizontal stress in the reinforced soil zone. Extensibility depends on the material used for reinforcement (metal versus nonmetal), geometric form, and influence of confinement. Inextensible reinforcements create a relatively unyielding mass, such that the state of horizontal stress approaches an at-rest condition. In contrast, extensible reinforcements allow the fill to yield laterally so that an active state can be reached throughout the reinforced soil zone. The total load carried by extensible reinforcements can equal that of inextensible systems if the number of extensible reinforcements per unit volume is high enough so that the soil cannot yield. Inextensible reinforcements are generally metallic materials whereas extensible reinforcements are generally polymeric materials. More detailed information about reinforcement materials can be obtained from manufacturers. A partial list of manufacturers is given in appendix D.

The selection of reinforcements is influenced by the following factors:

- Size of the structure
- Life of the structure
- Purpose of the structure
- Criticality of the structure
- Chemical durability of reinforcement in the soil
- Survivability of the reinforcement during construction
- Availability of reinforcement
- Cost.

With proper attention to design, the majority of reinforcement types can be used for most applications. However, in practice, some reinforcement types have been consistently used for specific applications and are presumably better suited or more economical for those applications.

During the process of selecting reinforcement options, make sure that all product information is up to date. Many reinforced wall systems are patented or proprietary, and some companies provide services including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction supervision.

Table 3-22.—Comparison of reinforcing elements.

Reinforcement Type		Stress Transfer Mechanism[1]		Reinforcement Material[2]		Extensibility[3]		Failure Surface[4]		Earth Pressure Coefficient[5]		
		Surface Friction	Passive Resistance	Metal	Nonmetal	Extensible	Inextensible	Rankine	Bilinear	K_a	$> K_o$	Linear Reduction of K_o to K_a
Strip	Smooth	X		X			X		X			X
					X	X--------	--------X		X			X
	Ribbed	X--------	--------X	X			X		X			X
					X	X			X			X
Grids	Wire Mesh		X	X			X	X	X	X	X	X
	Geogrids	X--------	--------X		X	X--------	--------X	X		X		
Sheet (Geotextiles)		X			X	X----	----X	X		X		

Notes:

Adapted from NCHRP 290 and FHWA 89-043 reports.

[1]**Stress Transfer Mechanism**

Friction: The stresses are transferred between soil and reinforcement by shear along the interface.

Passive Resistance: The stresses are transferred between soil and reinforcement by bearing between the transverse elements against the soil.

[2]**Reinforcement Material**

Metallic reinforcements: Mild steel.

Non-metallic reinforcements: Polymeric materials including polypropylene, polyethylene, or polyester polymers.

[3]**Reinforcement Extensibility**

Inextensible: The deformation of the reinforcement at failure is much less than the deformability of the soil.

Extensible: The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

[4]**Failure Surface**

Rankine: The failure surface is linear and extends from the base of the wall to the ground surface at an angle of $45 + \phi/2$ from the horizontal.

Bilinear: The failure surface approximates a curved surface.

[5]**Earth Pressure Coefficient**

K_a: The active lateral earth pressure coefficient.

K_o: The at-rest lateral earth pressure coefficient.

$> K_o$: Welded wire walls use a value of $K = 0.65$, which is larger than K_o.

The following are comments about selecting reinforcement:

- Reinforcement type should be selected based on strength and stiffness requirements, which are generally governed by the lower reinforcing layers.

- Inextensible reinforcements are designed for at-rest earth pressures because they restrain the lateral movement of the backfill. Based on this, metallic reinforcements have been used successfully in critical and permanent applications that are sensitive to lateral movement of the backfill such as abutment wing walls and walls adjacent to concrete roadways.

- Metallic reinforcements appear to have a higher resistance to construction-induced damage where angular gravel or crushed rock is used for backfill. If the reinforcement is coated (such as with an epoxy coating), some damage of the coating may be accounted for by adding reinforcement or a sacrificial thickness of coating.

- The use of metallic reinforcements should be restricted where surface or ground water is contaminated by acid mine drainage, or where the water has a low pH, a high pH, or a high concentration of sulphates or chlorides (such as where deicing salts will be in use).

- The use of metallic reinforcements should be restricted where stray electrical ground currents are anticipated within 200 feet of the structure.

- Geotextile reinforcement has been used in the majority of temporary wall applications because of generally lower cost and easier installation resulting from the availability of wide roll widths.

- The use of geotextile reinforcement has been limited in permanent wall applications because of difficulties at the wall facing (discussed in step 6).

Step 6: Select Wall Facing

The wall facing provides structural confinement of the backfill and is the primary determinant of wall aesthetics. Common facing types include:

- Welded wire mesh

- Shotcrete

- Metal sheets and plates

- Gabions or wire baskets

- Geotextiles and geogrids
- Precast concrete panels
- Precast concrete modular units
- Wood lagging and panels
- Tires, logs, rocks, and so forth.

The selection of wall facing type is influenced by the following factors:

- Size of the wall
- Life of the wall
- Purpose of the wall
- Settlement/movement of the wall
- Strength of connection to reinforcement
- Type of backfill
- Desired aesthetics of the wall
- Structures above the wall, such as guardrails
- Availability of facing
- Access and constructibility
- Cost.

The following are comments about selecting wall facing types:

- The facing type and joint spacing should be compatible with the settlement expected at the project site.
- The facing type should be compatible with the curve radius and corner details required at the project site.
- The facing should provide adequate confinement of the backfill material. A filter material, synthetic or natural, may be required behind the facing to prevent migration of backfill through openings in the facing.
- The facing should be compatible with any structures that may penetrate the facing, such as drainage conduits.

- The facing-to-reinforcement connection should be able to develop adequate strength at each layer. Conservatively, the connection strength should equal or exceed the strength of the reinforcement layer.

- Facing should be able to survive any anticipated damage from vandalism, vehicle impacts, debris impacts, or fire.

- Metallic facing, including wire mesh and wire baskets, traditionally have been used together with wire mat reinforcements although polymer grid reinforcements also have been used with wire mesh facing.

- The facing should be able to survive the local environment, such as freeze-thaw cycles. For example, it is important to have a low absorption rate in modular concrete blocks used in higher latitudes.

- Precast concrete panels traditionally have been used together with metallic reinforcement.

- Problems of degradation have occurred with certain types of polymer reinforcements when they were cast directly into concrete.

- The use of metallic facing should be restricted where surface or ground water is contaminated by acid mine drainage, or where the water has a low pH, a high pH, or a high concentration of sulphates or chlorides (such as where deicing salts will be in use).

- The use of metallic facing should be restricted where stray electrical ground currents are anticipated within 200 feet of the structure.

- Geotextile wrap facing has been used (as a continuation of the reinforcing layers) in the majority of temporary wall applications because of its generally lower cost.

- The use of geotextile wrap facing has been limited on permanent wall applications because ultraviolet light exposure has caused degradation of geotextile exposed at the face of the wall. This problem has been addressed by coating the wall facing with shotcrete or other coatings. However, if a coating is used, the reinforcement manufacturer should be contacted to verify that the coating is compatible with the reinforcement. For example, some polymers degrade in the presence of asphalt coatings as well as in the presence of some admixtures and byproducts of concrete curing (chlorides and sulphates).

- Timber facing can be used for both temporary and semipermanent applications if the timber is treated.

Step 7: Complete Internal Stability Analysis

Internal stability analyses ensure that the reinforcement is not overstressed and that the reinforcement length is sufficiently embedded. The reinforcement must be selected and spaced to prevent tensile overstress, pullout from the soil mass, and connection strength. These modes of failure are shown and described in figure 3-69.

For the purposes of this guide, and considering that large and complex applications will be referred to a qualified geotechnical engineer, an ultimate strength approach will be used for overstressing of the reinforcement, with the long-term design strength computed using reduction factors based on reinforcement material type.

The tieback wedge method of analysis is recommended for analysis of retaining walls reinforced with extensible reinforcement. This method assumes that shear strength of the reinforced fill is mobilized and active or at-rest lateral earth pressures are developed. These pressures must then be resisted by the reinforcement tensile force. The assumed failure plane is shown in figure 3-70.

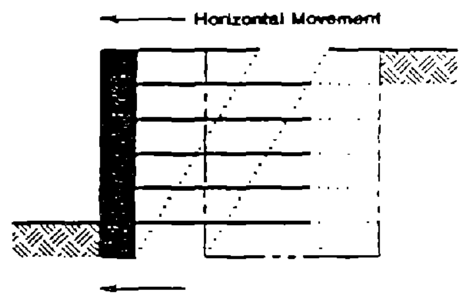
The tension (tensile force per unit width) in the reinforcement shall be calculated as a function of the vertical stress induced by gravity, uniform normal surcharges, and active thrust from the retained fill, multiplied by the earth pressure coefficient (K) (see example in Section 3K.6, "Design Calculations").

The required reinforcement tension, calculated using the tieback wedge method, should be less than or equal to the allowable reinforcement tension.

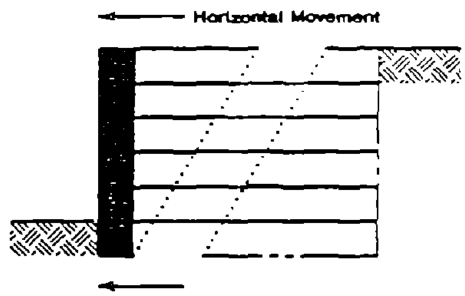
The reinforcement length should be the longer of the lengths required for external or internal stability. The length required for internal stability shall be determined by designing for a minimum factor of safety of 1.5 against pullout failure (i.e., at each reinforcement level the pullout resistance should be at least 1.5 times the required reinforcement tension). They differ according to geometry, stress transfer mechanisms, and extensibility of the reinforcement, as shown in table 3-22. Only the effective pullout length extending beyond the failure surface into the resistant zone shall be used in the computation of the required pullout resistance. Reinforcement length should be uniform throughout the wall.

Minimum geotextile or geogrid length in the resistant zone shall be 3 feet. The total reinforcement length at any level is equal to the sum of the lengths in the active and resistant zones. Reinforcement length shall be uniform throughout any section of the wall (AASHTO Task Force 27 Report).

a) Pullout: Excessive movement of the reinforcement through the soil



b) Tensile Overstress: Applied stress to the reinforcement exceeds the allowable working stress level



c) Connection Strength: Rupture of the connection between the reinforcement and the facing

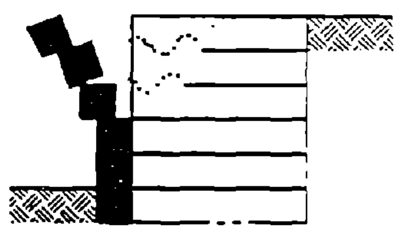


Figure 3-69.—Modes of internal stability failures.

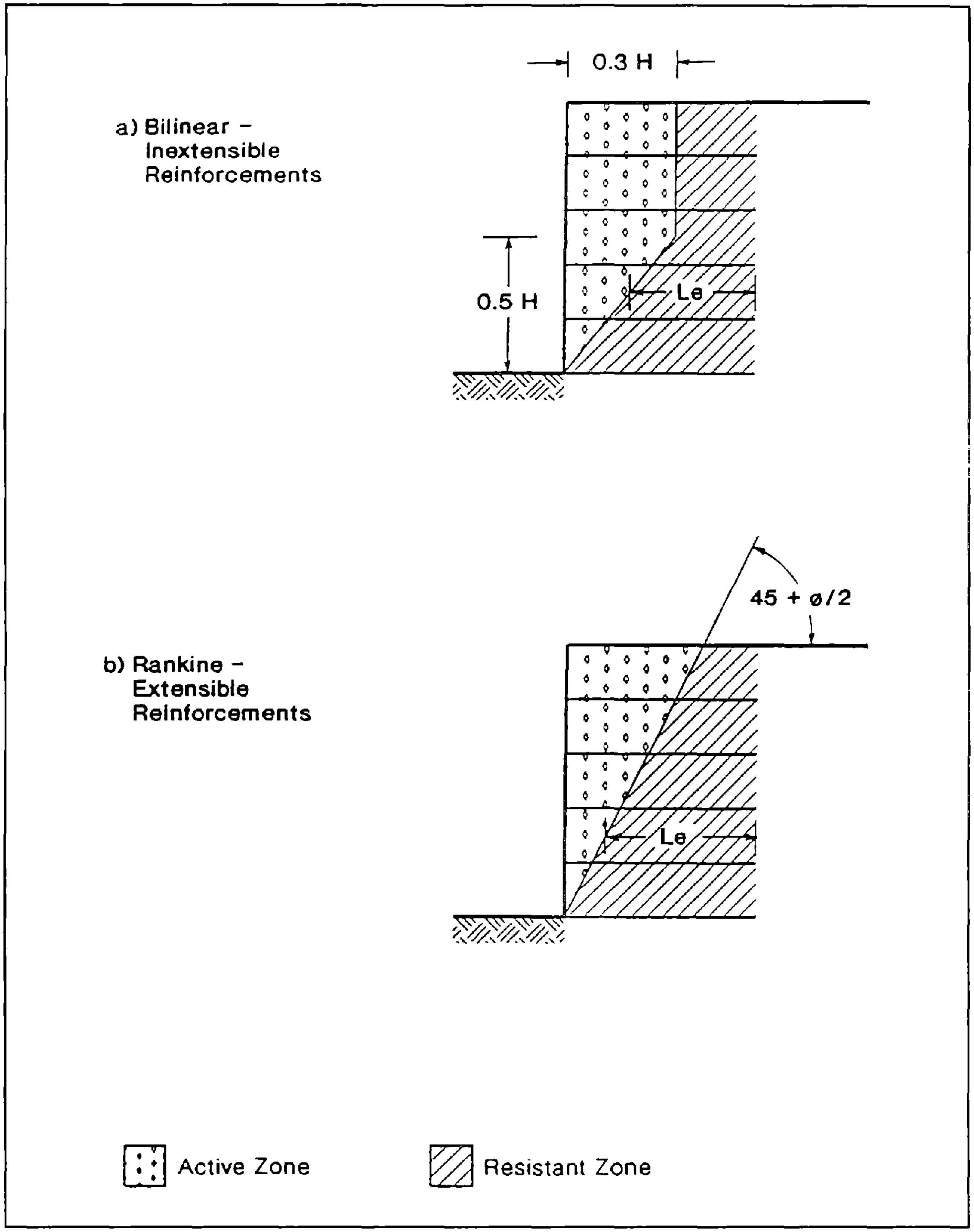


Figure 3-70.—Bilinear and Rankine failure planes.

Step 8: Develop Construction Drawings and Specifications

Construction drawings and specifications should be developed for the specific project. These documents can be as simple as a cross-section with specified products and construction notes. For large jobs, the construction documents can be complex. Many resources are available to aid the designer in preparing drawings and specifications. The manufacturers have examples available. Technical specifications are included or referenced in most of the design manuals listed in section 3K.3, "Current State of the Practice." Appendix F contains example specifications, and appendix C contains example drawings.

3K.6 Design Calculations

This section provides the design steps, narrative, and calculations required to design a reinforced soil retaining wall. The steps listed are a detailed version of the steps previously presented. The left-hand column lists the steps and suggests guidelines when choosing required parameter values. The right-hand column supplies the necessary equations to design the reinforced soil wall. In the internal stability step, step 7, the guidelines and equations provided are general. The user may need to refer to detailed technical references for equations developed to analyze internal stability of specific types of reinforcement. Example calculations are provided in chapter 4.

Design Steps and Narrative

Step 1: Satisfy Design Assumptions

See Section 3K.4, "Assumptions and Limitations," for additional comments. If level toe conditions cannot be met, deeper embedment may be required.

Calculations

- Noncritical structure.
- Wall heights are limited to 16 feet.
- Wall has level backfill conditions or sloping and broken backfills can be modeled as a surcharge of less than 500 psf (pounds per square foot).
- Wall has level toe conditions for a minimum distance of one-half the wall height.
- Competent foundation below wall.
- Soils in the foundation, and reinforced and backfill zones are uniform and have low plasticity.
- Soils used in the reinforced and backfill zone are free-draining or drainage is provided.
- Surcharges are limited to 500 psf and are uniform.
- Seismic loading is not a concern.

- Same length of reinforcement through full height of wall.
- See other calculations in Section 3K.4, "Assumptions and Limitations."

Step 2: Generate Wall Profile

Calculation

- See step 2 in Section 3K.5, "Modern Methodology."

Step 3: Determine Wall, Loading, and Soil Parameters for Each Wall Section

See figure 3-67. Subscripts "r", "b", and "f" refer to reinforced soil, backfill soil, and foundation wall.

Calculation

- Determine values for the following:

H	ϕ_r	γ_r
D	ϕ_b	γ_b
q	ϕ_f	γ_f
c_f		

Step 4: Complete External Stability Analysis

External stability analysis ensures that the reinforced structure is stable against the action of the lateral pressures applied by the retained soil and surcharge loading.

The minimum reinforced zone width, L, is selected when the minimum factor of safety against each of the three potential external failure modes is satisfied. This minimum reinforced zone width must be maintained by all reinforcement layers within the reinforced soil wall.

Calculations

- Calculate K_a, the active earth pressure coefficient. For external stability calculations, K_{ab} should be calculated for the soil backfill.

$$K_{ab} = \tan^2(45 - \frac{\phi_b}{2}) \qquad (3\text{-}106)$$

Step 4.1: Check Sliding

The width, L, of the reinforced soil zone must be great enough to ensure an adequate shear capacity along the base of the reinforced zone to prevent sliding of the structure along its base. The horizontal driving forces are due to the backfill weight and any surcharge loading acting at the surface of the backfill behind the reinforced soil zone.

Calculations

- The factor of safety against sliding, FS_{SL}, could be calculated using the following equations. See figure 3-71 for the free body diagram of the forces acting on the wall.

$$FS_{SL} \leq \frac{horizontal\ resisting\ forces}{horizontal\ sliding\ forces}$$

$$\leq \frac{[qL\gamma_r\ (H)L]\ \tan\phi}{q(H)K_{ab} + 0.5\gamma_b(H^2)K_{ab}} \qquad (3\text{-}107)$$

- Use the smallest ϕ value of ϕ_r and ϕ_f for calculations.

- By rearranging the previous equation, the required length of reinforcement, L, can be determined. Use a minimum FS_{SL} equal to 1.5 in the calculation.

$$L \geq \frac{FS_{SL}\ (\ K_{ab}\ )H\ [q + 0.5(\gamma_b\ )H]}{[q + \gamma_r(H)]\ \tan\phi} \qquad (3\text{-}108)$$

Step 4.2: Check Overturning

The reinforced soil zone must have a sufficient total mass to prevent the reinforced soil zone from overturning about the toe of the wall.

Calculations

- The factor of safety against overturning, FS_{OT}, can be calculated using the following equations. See figure 3-71 for the free body diagram of the forces acting on the retaining wall.

$$FS_{OT} \leq \frac{moments\ resisting}{moments\ overturning}$$

$$\leq \frac{3L^2[q + \gamma_r(H)]}{K_{ab}(H^2)[\gamma_b(H) + 3q]} \qquad (3\text{-}109)$$

- By rearranging equation 3-109, the required length of reinforcement, L, can be determined. Use a minimum FS_{OT} equal to 2.0 in the calculation.

$$L \geq \frac{FS_{OT}(K_{ab})H^2\ [\gamma_b(H) + 3q]}{3[q - \gamma_r(H)]} \qquad (3\text{-}110)$$

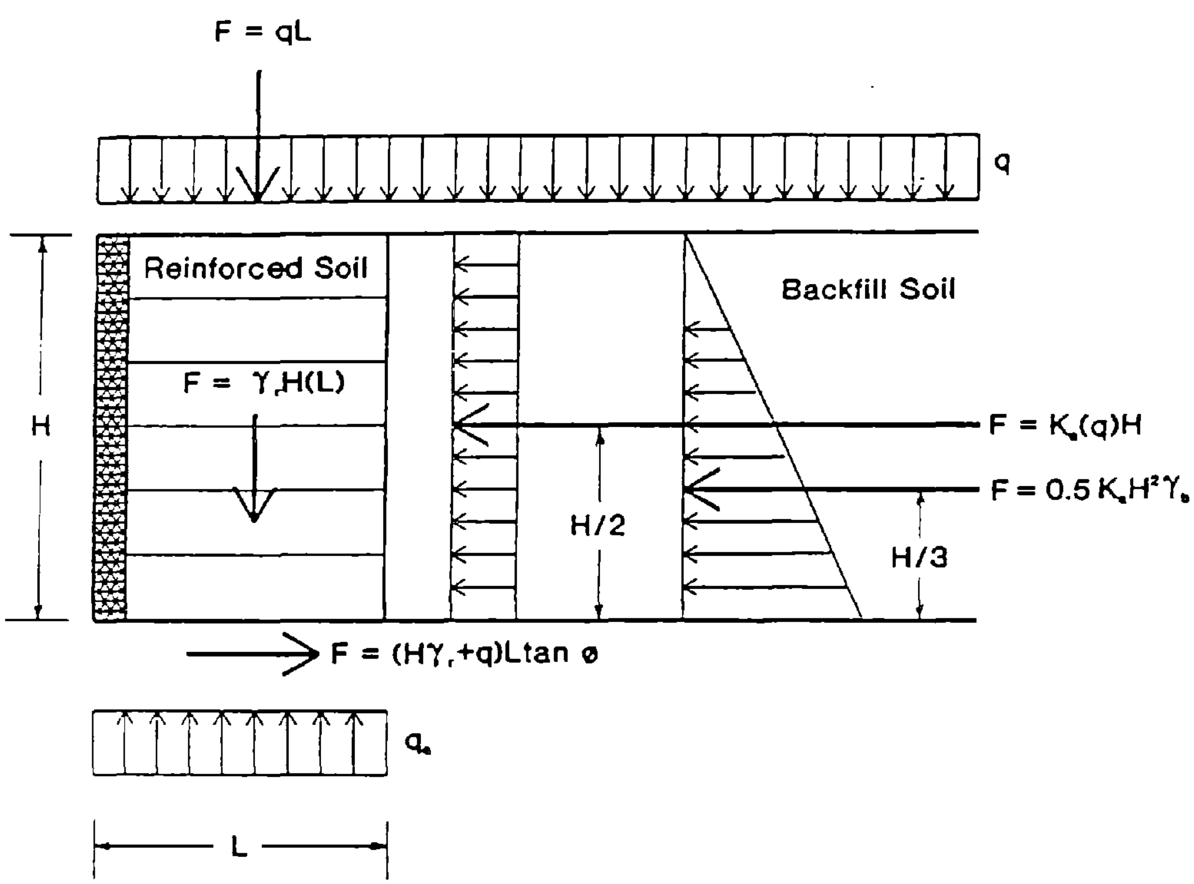


Figure 3-71.—Free body diagram of reinforced soil wall.

Step 4.3: Check Bearing Capacity

The shear strength of the underlying foundation soils must not be exceeded. A Meyerhof type of pressure distribution is used to estimate the applied bearing pressure on the foundation. The applied pressure distribution is assumed to be uniform over an effective width of footing B equal to $L - 2e$.

Calculations

- Use the largest L value obtained from steps 4.1 and 4.2 for calculations in step 4.3.
- Calculate eccentricity, e, of bearing pressure resultant force by summing moments about the center, $L/2$, of the reinforced soil zone considering counterclockwise movement positive (+).

$$e = \frac{K_{ab}(H^2)\ [\gamma_b H + 3q]}{6L[\gamma_r H + q]} \qquad (3\text{-}111)$$

A uniform bearing pressure is assumed to exist over a length equal to L-$2e$, where e is the eccentricity of the bearing pressure resultant from the vertical center line of the wall fill. The maximum allowable eccentricity is typically limited to $L/6$ to prevent possible tension at the base of the reinforced zone. The reinforcement length, L, should be increased if e exceeds $L/6$.

- Ensure that no tension develops in the foundation soil.

  If $e > L/6$, increase L and recalculate e.
  If $e < L/6$, continue.

- Calculate the equivalent footing width, B.

$$B = L - 2e \qquad (3\text{-}112)$$

- Calculate the applied bearing pressure, Q_a.

$$Q_a = \frac{\gamma_r(H)L + q(L)}{B} \qquad (3\text{-}113)$$

The geotechnical site investigation should determine the allowable bearing capacity q_a, of the foundation soils. The allowable bearing capacity should be calculated with a factor of safety against bearing capacity failure, FS_{BC}, of 2.0 to 3.0.

- The allowable bearing capacity, q_a, is defined in equation 3-114. The ultimate bearing capacity, Q_{ult}, should be determined during the site investigation.

$$q_a = \frac{Q_{ult}}{FS_{BC}} \qquad (3\text{-}114)$$

  in which Q_{ult} equals the ultimate bearing capacity of the foundation soil, and FS_{BC} equals 2.0 to 3.0.

- To prevent bearing capacity failure, the allowable bearing capacity, q_a, must be greater than the previously determined applied bearing pressure, Q_a.

  If $q_a \geq Q_a$, go to step 5.0; if $q_a \leq Q_a$, increase L and redo step 4.3.

Step 5: Select Reinforcement

Calculation

- Table 3-22 compares the various reinforcement systems, and step 5 in Section 3K.5, "Elements of a Reinforced Retaining Wall," provides guidelines on selecting the appropriate reinforcement type.

Step 6: Select Facing

Calculation

- Many facing types correspond with the type of reinforcement selected, such as welded wire facing mesh with wire mesh reinforcement. Step 6 in Section 3K.5, "Elements of a Reinforced Retaining Wall," provides guidelines on selecting the appropriate facing type.

Step 7: Complete Internal Stability Analysis

Select reinforcement type, number of reinforcement layers, and their location to ensure that the reinforced soil zone does not fail internally due to pullout, tensile overstress, and corrosion.

The tieback wedge method of analysis is recommended for analysis of retaining walls reinforced with extensible reinforcement. This method assumes that shear strength of the reinforced fill is mobilized and active or at-rest lateral earth pressures are developed. These pressures must then be resisted by the reinforcement tensile force. The assumed failure plane is defined in figure 3-70.

The tension (tensile force per unit width) in the reinforcement should be calculated as a function of the vertical stress induced by gravity, uniform normal surcharges, and active thrust from the retained fill, multiplied by K, the earth pressure coefficient. The required reinforcement tension, calculated using the tieback wedge method, should be less than or equal to the allowable reinforcement tension.

The reinforcement length should be the longer of the lengths required for external stability or internal stability. The length required for internal stability shall be determined by designing for a minimum factor of safety of 1.5 against pullout failure (i.e., at each reinforcement level the pullout resistance should be at least 1.5 times the required reinforcement tension). Only the effective pullout length extending beyond the failure surface into the resistant zone shall be used in the computation of the required pullout resistance. Reinforcement length should be uniform throughout the wall.

The minimum length of reinforcement in the resistant zone L_e is 3.0 feet. The total reinforcement length at any level is equal to the sum of the lengths in the active and resistant zones. Reinforcement length shall be uniform throughout any section of the wall (AASHTO Task Force 27 Report).

Step 7.1: Select Failure Surface and Earth Pressure Coefficient

The reinforced soil mass can be divided into two regions, the active zone and the resistant zone. The active zone is located immediately behind the face of the wall. In this region, the soil is trying to move away from the soil behind it. The stresses produced by this movement are directed outward and must be resisted by the reinforcements. The forces in the reinforcements are transferred to the resistant zone, where the soil shear stresses are mobilized in the opposite direction to prevent the pullout of the reinforcements. The reinforcements hold these two zones together, making a coherent soil mass. Figure 3-72 shows the assumed variation in earth pressure coefficients with depth for different wall types.

Calculations

- The appropriate potential failure surface should be selected from table 3-22 for the reinforcement type to be used.

- Inextensible reinforcements, including metallic strips or grids, can be evaluated for internal stability using K_o, the lateral earth pressure coefficient at rest. The failure surface is assumed to be bilinear, as shown in figure 3-70.

$$K_o = 1 - \sin \phi_r \tag{3-115}$$

  in which ϕ_r equals the internal friction angle of the reinforced soil.

- Extensible reinforcements, including polymeric reinforcements, could be analyzed using a tieback wedge method of analysis. The full shear strength of the reinforced fill is assumed to be mobilized and active lateral earth pressures are developed. The failure surface is assumed to be defined by the Rankine active earth pressure zone. This zone is defined by a straight line passing through the wall toe and oriented at an angle of $45° + \phi_r/2$ from the horizontal.

$$K_{ar} = \tan^2 (45 - \phi_r/2) \tag{3-116}$$

Reinforcement manufacturers can provide recommendations on the spacing of reinforcement and location of connections. The general guidelines listed below can be used to estimate required reinforcement spacings. The guidelines provided should be verified by the manufacturers to obtain the most current available information.

Calculations

- Choose vertical spacing measurements, S_z. Metallic reinforcement applications may also require selection of horizontal spacings S_x and S_y. More specific guidelines for reinforcements can be obtained from the manufacturers.

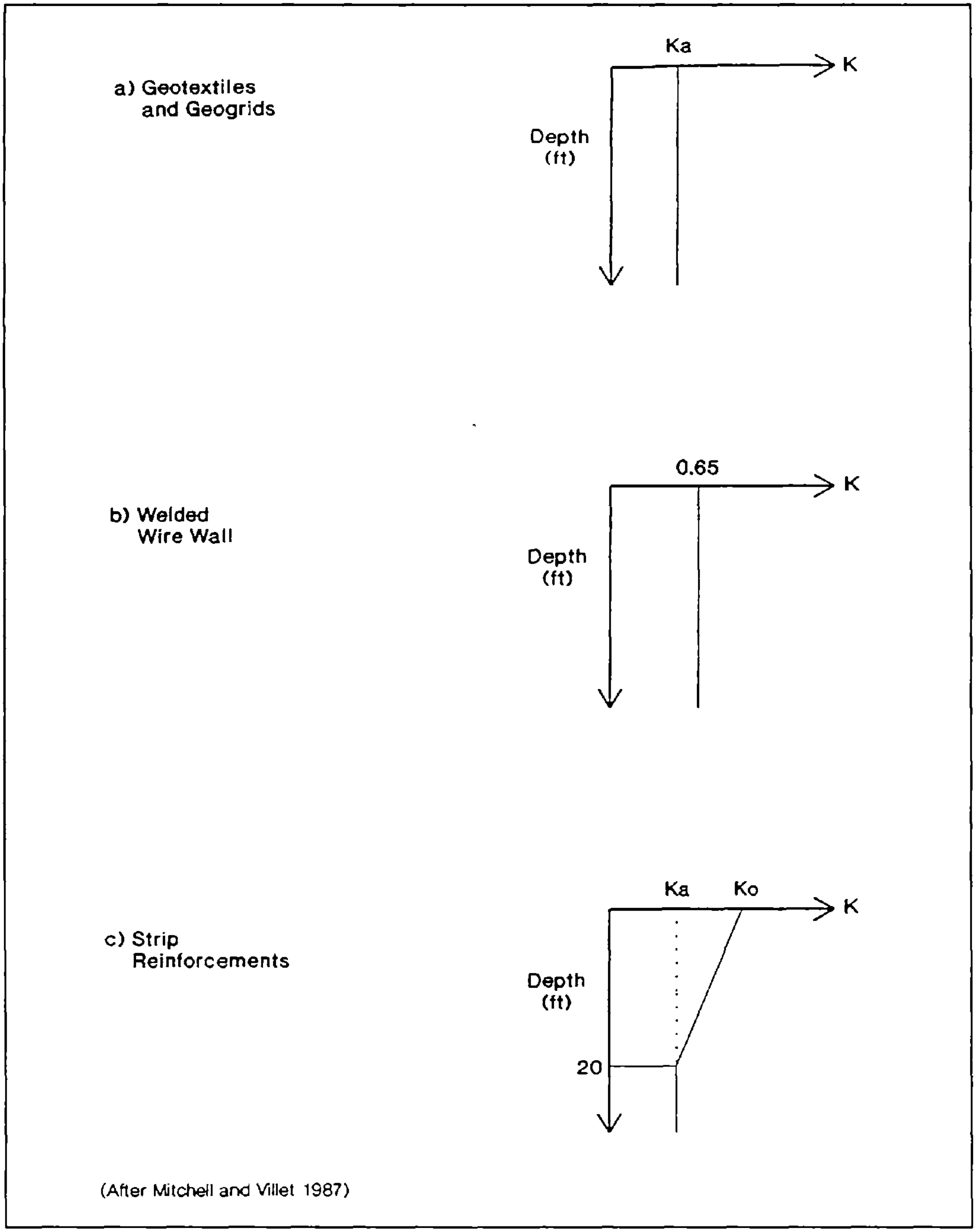


Figure 3-72.—Earth pressure coefficients.

- Choose strength of reinforcement. The stronger the material, the larger the spacing can be. The strength of the material varies according to the cross-sectional area, spacing of members, and/or thickness.

Step 7.2: Select Spacing Measurements

Strip Reinforcement for Reinforced Earth

To increase internal stability, vertical spacings are normally maintained constant, but the number of strips is increased or stronger strips are used.

The reinforcement spacing both vertically and horizontally is predetermined by the connector tab spacings on the precast facing elements. Usual minimum values for these spacings are 30 inches for vertical spacing and 40 inches for horizontal spacing.

Grid Reinforcement for Welded Wire Wall

To increase internal stability, vertical spacings are normally maintained constant, but stronger wire mats are used. Vertical spacing between mats can also be reduced.

The standard vertical spacing is 1.5 feet. Two feet of fill are typically used above the top reinforcement.

Grid Reinforcement for Geogrid Reinforced Wall

To increase internal stability, vertical spacing between layers is reduced or stronger reinforcement is used.

Vertical spacing must be determined for each wall depending on applied horizontal stress and design strength. For an initial estimate, use a vertical spacing, S_Z, of 1 foot.

Sheet Reinforcement for Geotextiles

Vertical spacing must be determined for each wall depending on allowable long-term tension in geotextile and lateral pressure applied in the middle of each layer. The most common way of varying the reinforcement density is to change the vertical spacing. The vertical spacing should be a multiple of the compacted lift thicknesses required for the fill (typically 8 to 12 inches). The range of acceptable spacings is a minimum value of 6 inches and maximum value of 2.0 feet.

The use of a constant reinforcement spacing for the full height of the wall usually gives more reinforcement near the top of the wall than is required for stability. Therefore, following the internal stability analysis, alternative spacings can easily be evaluated by analyzing the reinforcement strength requirements at different wall depths and modifying spacing accordingly, or by changing the strength of the reinforcement to match the spacing requirements.

Step 7.3: Calculate the Tensile Stress in Reinforcement and at Connection Between the Facing and Reinforcement

The tensile stress in each reinforcement layer must be calculated. These values will be used in later calculations. Tensile stresses and connection strength should be determined for walls having stiff facing elements. Tensile stresses at the facing generally are not considered for wrap-faced walls. It is assumed that the tensile stress of the wall at the facing corresponds to the maximum tensile stress in the reinforcement layer.

Calculations

- Calculate the tensile stress to be distributed to the reinforcement.

$$\sigma_x = K\,\sigma_z = K(\gamma_r z + q) \tag{3-117}$$

in which σx equals the horizontal tensile stress, σ_z equals the vertical compressive stress, and z equals the depth below the top.

- Calculate the total horizontal load, P_H, to be resisted by a given layer.

$$P_H = \sigma_x (S_x) S_z \tag{3-118}$$

or

$$P_H = K\sigma_z S_x (S_z) \tag{3-119}$$

in which S_x equals the horizontal spacing, S_z equals the vertical spacing, and K equals the earth pressure coefficient.

Step 7.4: Check Pullout of Reinforcement

Pullout is the excessive movement of the reinforcement through the soil without rupture of the reinforcement. The pullout capacity is dependent on the type of reinforcement used. There are three types of pullout resistance utilized by the reinforcement, including frictional, passive, and a combination of frictional and passive. The equations used to determine this resistance are usually empirical and can be obtained from several of the referenced technical documents. This guide uses a reduced friction angle of the reinforced soil to compute the pullout resistance.

Calculations

- Check factor of safety against pullout, FS_p, of the reinforcement.

$$FS_p = P/P_H \tag{3-120}$$

in which P equals the ultimate pullout capacity = 2 (tan ϕ_{sr}) $L_e\,\sigma_z$ (equation 3-121), and ϕ_{sr} equals the angle of soil/reinforcement friction, dependent on the reinforcement type.

Material	ϕ_{sr}
ribbed strips	ϕ_r
wire mesh	$2/3\phi_r$
geogrids	$2/3\ \phi_r$
Geotextile	$2/3\ \phi_r$

If $FS_p \geq 1.5$, continue to step 7.5; if $FS_p < 1.5$, increase L or decrease S_x and S_z and redo steps 7.3 and 7.4.

Step 7.5: Check Overstressing of Reinforcement

Overstressing occurs when the applied stress felt by the reinforcement exceeds the allowable working stress level. Overstressing may lead to excessive strain or movement in the reinforcement. This guide uses reduction factors applied to ultimate strength of polymer reinforcements and applied to the yield strength of metallic reinforcements.

Manufacturers can recommend ultimate and design material strength values. AASHTO recommends an allowable tensile stress for steel grid reinforcements and connections of $0.48F_y$, in which F_y is the yield strength.

Manufacturer-supplied independent test results, such as long-term stress-strain-time behavior of the materials, may be substituted to evaluate project-specific strength values.

Calculations

- Check factor of safety against overstressing, FS_R, at the end of the design life.

$$FS_R = \frac{F_a}{P_H} \qquad (3\text{-}122)$$

in which F_a equals $f(F_y)$ for metal, and F_a equals $f(T_u)$ for geosynthetics; and in which f equals reduction factor, dependent on material type; T_u equals the ultimate material strength from ASTM method 4595 for geosynthetics; and F_y equals the yield strength of steel.

The following guidelines are suggested:

Material type	Reduction factor (f)
Steel	0.48 (requires corrosion evaluation)
Polyester	0.13
Polyethylene	0.10
Polypropylene	0.08

If $FS_R \geq 1.0$, continue; if $FS_R < 1.0$, increase cross-sectional area, strength, or thickness of the reinforcement and/or decrease the spacing of S_x and S_z and return to step 7.3.

Step 7.6: Evaluate Corrosion Rates for Reinforcement Material

Corrosion rates for the reinforcements should be evaluated over the design life of the structure (75 years).

<u>FHWA corrosion rates</u>:

= 15 µm/yr
= 0.059 mil/yr for first 2 years
= 4 µm/yr
= 0.016 mil/yr for subsequent years

Carbon steel loss

= 15 µm/yr
= 0.059 mil/yr after zinc depletion
where 1 mil = 0.001 inch

Environmental conditions, such as temperature, microbial attack, construction damage, UV degradation, and chemical attack can directly affect polymeric materials. Product-specific laboratory data may be evaluated when determining the F_a value for the reinforcement.

Calculation

- For metallic reinforcements:

$$T_c = T_n - T_s \tag{3-123}$$

  in which T_c equals the thickness at the end of design life, T_n equals the nominal thickness at construction, and T_s equals the thickness lost due to corrosion during design life.

- For polymeric reinforcements, environmental conditions have been conservatively accounted for in the reduction factor, f.

Step 8: Develop Construction Drawings and Specifications

Calculations

- See example specifications from different public agencies and manufacturers in appendix F.

- See example drawings for the welded wire, geotextile, and Keystone walls in appendix C.

3L Reinforced Embankments

3L.1 General

Reinforced embankments (reinforced fills) or mechanically stabilized earth slopes consist of an embankment fill built up in compacted lifts with layers of a reinforcing material, such as a geogrid, welded wire, or geotextile, placed throughout the embankment. The reinforcing material adds tensile resistance to local (face) and deep-seated shear failure in the embankment. In granular soils, reinforced fills placed with a 1H:1V or steeper face slope can offer an economical alternative to retaining structures for those sites where the ground is too steep to catch a conventional 1-1/2:1 (horizontal:vertical) fill slope, yet is flat enough to catch an oversteep reinforced fill. In poor plastic soils, reinforcement can steepen the stable slope angle of an embankment. Reinforced fill heights commonly range from 25 to 50 feet on most projects, and the highest fill built to date is a 115-foot-high, 1:1 fill in Southern California.

The spacing of the primary reinforcement is chosen to add the tensile strength needed to support the oversteepened fill slope and to prevent a deep-seated slope failure. Spacing typically varies between 2 and 5 feet, depending on soil parameters, the height of the fill, and the strength of the reinforcement. Intermediate reinforcement, placed between the primary reinforcement, typically consists of narrow (usually 4- to 5-foot wide) strips of low strength reinforcement placed along the fill face on a 1-foot vertical spacing or with each construction lift to prevent local fill surface failure. Figures 3-73 and 3-74 show typical reinforcement applications and layout.

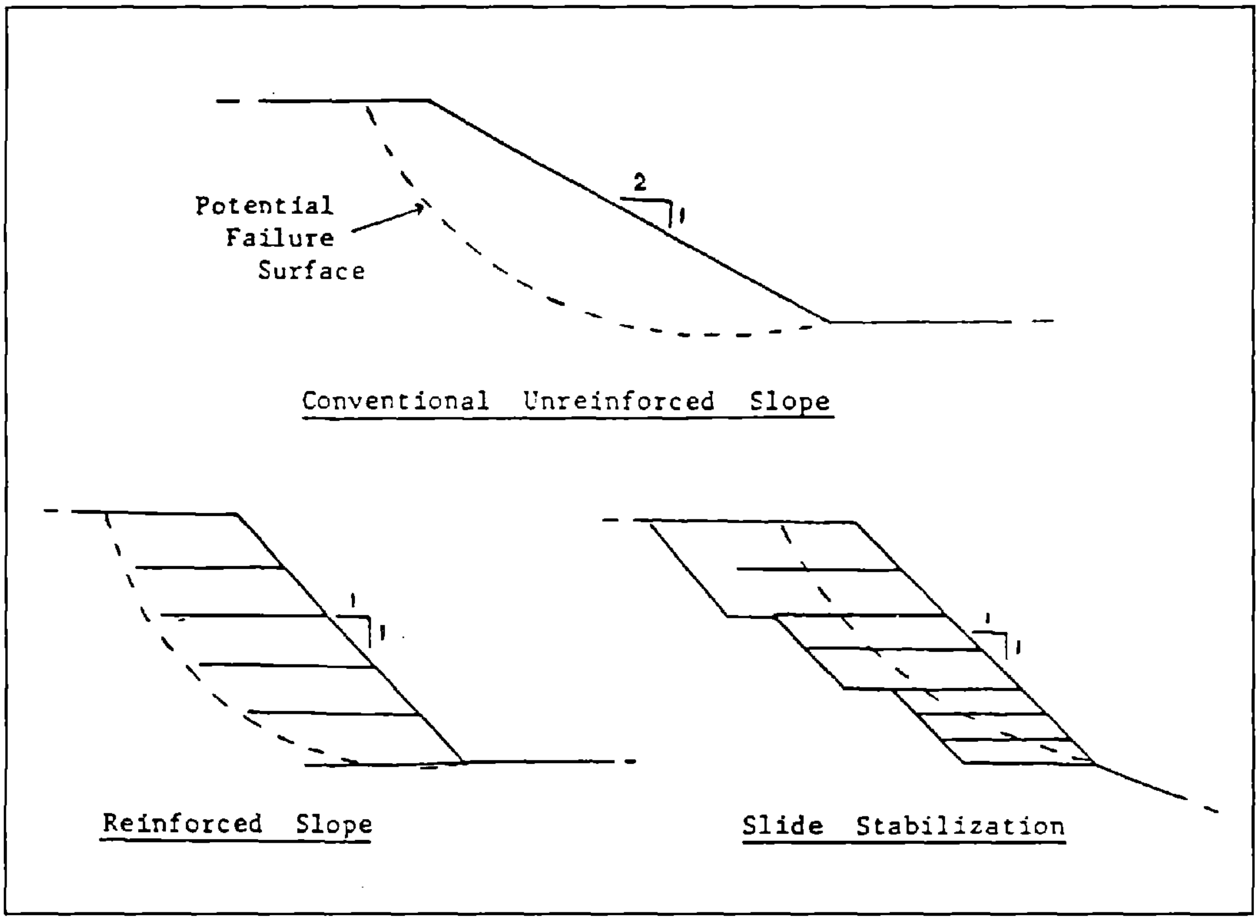


Figure 3-73.—Typical reinforced slope applications.

The primary application of reinforced embankments is to construct an embankment with an oversteepened face slope to fit the space constraints of a particular site and avoid the use of a retaining wall. This is primarily a cost consideration. Other applications include sites where the area of disturbance must be minimized, or the toe of a slope is constrained to a particular elevation or limit, or additional fill surface width is needed (with a constrained toe location). Another purpose for slope reinforcement is to provide improved compaction on the edge of a slope, thus decreasing the tendency for surface sloughing. The addition of intermediate reinforcement on the oversteep face between the primary reinforcement layers also improves construction safety by reducing the likelihood of failures due to construction equipment loading. Also oversteepened reinforced slopes can be used for landform contour grading, where variably steep slopes are designed for aesthetic purposes to simulate local natural topography (Miyake et al., 1993). Reinforced embankments built into a roadcut have also been used as a drained "buttress" in a limited space application.

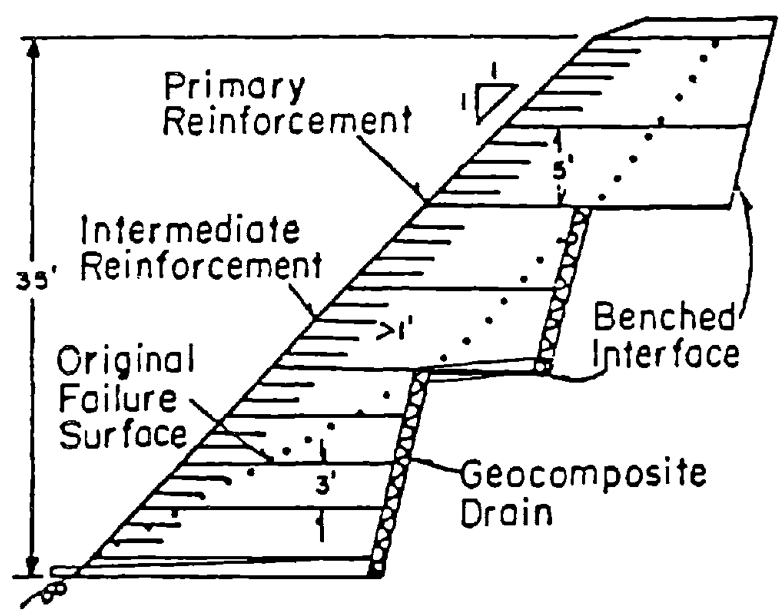


Figure 3-74.—Reinforced fill cross-section with drainage and a bench fill-native soil interface to prevent sliding failure.

Reinforcement may be used in the base of embankments to improve the overall embankment foundation stability by creating a "wide footing" over a soft subgrade or poor foundation soils. This application is discussed in publications such as the *Federal Highway Administration (FHWA) Geotextile Engineering Manual* (Christopher and Holtz, 1984). It is not further discussed in this guide.

Although reinforced embankments are typically constructed with select frictional backfill, successful projects have also been constructed using marginal, silty backfill materials and clay-rich plastic materials. The use of poor backfill material requires special design and construction considerations, which should be carefully and individually evaluated.

The reinforced embankment concept can also be applied in biotechnical slope stabilization where layers of brush or vegetation, such as willows, are built into the fill. This concept is commonly known as "brush layering." Vegetation adds slope reinforcement from the woody debris, it breaks up the slope for erosion control, and can offer long-term root strength for slope stability. This concept should be regarded as a surficial slope stabilization and erosion control measure, rather than a "designed" deep-seated slope reinforcement and stabilization measure. However, the concept of primary reinforcement with material such as geogrid, plus the use of vegetation in the outer several feet of the lift as secondary reinforcement, is likely the "best of both worlds" for long-term stabilization.

3L.2 Design Information

Information needed for design of reinforced fills includes site and materials information similar to those needed for most retaining structure designs. This includes knowledge of the local site geology, ground water conditions, mechanism of failure, and having an accurate site cross-section. Information on backfill materials strength, including frictional and cohesive strength parameters and characteristics (gradation, plasticity, pH, and so forth), are typically necessary. Methods of obtaining this information are discussed elsewhere in this design guide.

Additional information is needed on the reinforcing material used in the slope. This material typically is a geogrid, although geotextiles, welded wire mesh, chain link fencing, and other materials have been used for reinforcement. Material properties needed include design long-term strength, creep characteristics, stress-strain relationships, durability, corrosion resistance, and so forth. Much of this information is available from manufacturers, and the topic is the subject of ongoing research.

Pullout resistance properties of the reinforcement material in contact with the soil are particularly important for the function of the structure. Pullout resistance is mobilized through some combination of cohesion, interface friction, and passive soil resistance against transverse elements of a reinforcing grid. Geogrids with a high percentage of open area and some thickness to the grid offer the highest passive resistance, whereas a geotextile has dominantly frictional resistance. Pullout resistance design values or factors are typically obtained from pullout tests performed by manufacturers on a range of soil types or on backfill to be used on a large project. Standardized pullout test methods are currently being developed.

3L.3 Design Process

The stability of reinforced slopes and fills is typically analyzed using different versions of conventional limit equilibrium analyses that are modified to account for the needed or added tensile strength of the reinforcing material. The reinforcing material-soil interaction and pullout resistance is similar to the approach used in earth reinforced retaining structures. A circular- or wedge-shaped potential failure surface is assumed for stability analysis, and the relationship between resisting and driving forces or moments determines the factor of safety.

Detailed design procedures for reinforced fills have been summarized in a reference published by the FHWA (Christopher et al., 1990), and outlined in texts by Koerner (1990). Tall reinforced structures, roughly over 20 feet high, and those in critical areas should be designed using comprehensive slope stability analysis. Several reinforced slope computer programs are commercially available, such as PCSTABL6, TENSLO, STABGM, and UTEXAS II, and a comprehensive design program is currently being developed by the FHWA.

The XSTABL stability analysis program (Sharma, 1992), developed for the Forest Service, can also be used for reinforced fill analysis. In the XSTABL program the magnitude of external horizontal resisting force needed to maintain stability, or achieve moment equilibrium for the given embankment geometry, loading conditions, and factor of safety, is determined. The engineer must then select an appropriate reinforcing material and distribution of that reinforcement material throughout the embankment to equal the required resisting force, as described herein and by others (Christopher, 1990; TENSAR, 1986).

Simplified hand solutions have been developed and published by Schmertmann (1987), and step-by-step design charts are available from various manufacturers, such as the TENSAR Corporation (1986), MIRAFI (1991), or AMOCO (1988). Figure 3-75, published by FHWA (Christopher, et al. 1990) presents the chart solutions developed by Schmertmann for conditions with no pore pressure. A more detailed explanation of the use of these design charts, with an example and treatment of reinforcement spacing and distribution in the embankment, is presented by the TENSAR Corporation (1986). TENSAR's design procedure is used in the sample problem in chapter 4.

Jewell et al. (1984) have developed design charts which can account for a water table in the embankment through the use of a pore pressure coefficient, *Ru*. Values include *Ru* equal to 0, 0.25, and 0.50. These charts have been presented by Mitchell and Villet (1987) and are shown in figures 3-76 through 3-78, along with a design chart example problem.

Design charts, without detailed stability analysis, are commonly used to design small projects. This approach appears conservative but appropriate for rural, noncritical applications on relatively small fills under 25 feet high, and in good soils. Also, the cost of reinforcing materials is a relatively small percentage of the total repair cost of a site, so optimization of a design on small projects is not critical and may not be cost-effective.

Often it is desirable to use both procedures: the simplified design chart method for a preliminary design, then checking and refining the design with the stability analysis. More detailed information on reinforced embankments is available in the Forest Service's *National Slope Stability Guide*.

The conventional design of a reinforced fill involves determining the following specifications:

- The desired final slope geometry and loading conditions.

- The selection of fill material, determination of fill, backslope, and foundation materials properties, and evaluation of site conditions. Look for unusual site conditions, cause of failures, stratified materials, drainage needs, etc.

- The performance requirements (factors of safety, reinforcement strength, and durability criteria) for the system and materials.

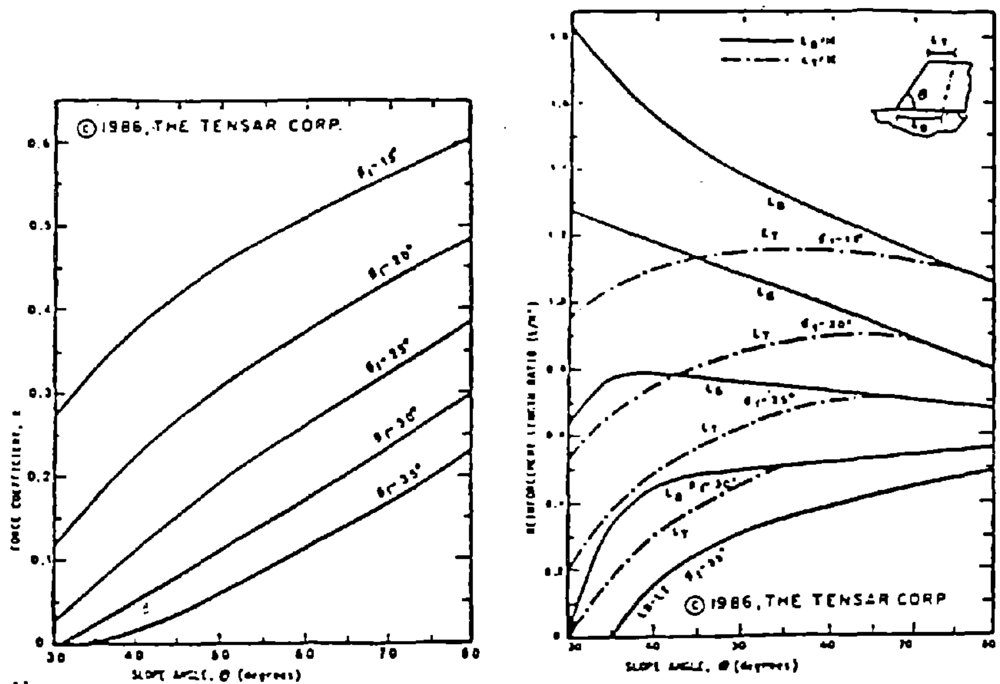
- The unreinforced stability of the slope using conventional stability analysis methods to determine the unreinforced slope factor of safety (usually less than or equal to 1.0).

- The forces acting on the oversteepened slope, which must be resisted with reinforcement to achieve internal stability. In other words, the total reinforcement tensile force is determined which is necessary to resist the driving forces and achieve stability with the desired factor of safety for the slope (usually *FS* = 1.3). Note that this total tensile force (Ttot) is determined from limit equilibrium analysis, as shown in figure 3-79 or by using previously developed design charts such as figure 3-77. Note that extensible geogrids and geotextiles and inextensible wire mesh reinforcing materials develop somewhat different tensile force components.

- The required number and type strength of reinforcement layers needed to equal the total reinforcement tensile force (Ttot). The number of layers depends on the allowable design tensile capacity of each layer of reinforcing material chosen.

- The reinforcement distribution of the number of layers needed throughout the embankment. This distribution is typically uniform in embankments lower than 20 feet. Higher embankments should be divided into two or four zones, with the reinforcement placed with a spacing inversely proportional to the depth. For instance, with two zones, the bottom zone receives three-fourths of the total reinforcement, and the top zone receives one-fourth. With three zones, the bottom third receives one-half of the reinforcement; the middle receives one-third, and the top zone receives one-sixth.

- The maximum allowable vertical spacing between reinforcement layers is determined to prevent reinforcement material rupture, and it depends on the allowable materials tensile strength and horizontal stress developed at a given depth and reinforcement spacing. The actual spacing of layers is chosen as a multiple of lift thickness.



A) B)

Chart Procedure

Limiting Assumptions:

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil (c=0).
- No pore pressures within the slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge no greater than $0.2\gamma H$.
- Relatively high soil/reinforcement interface friction angle $\phi_{sr} = 0.9\ \phi_r$ (may not be appropriate for some geotextiles).

1. Determine force coefficient K from Fig. A above where $\phi'_f = \tan^{-1}(\tan\phi_r/FS_R)$.

2. Determine $T_{max} = 0.5K\gamma_r\ H'^2$

   where $H' = H + q/\gamma_r$
   q is a uniform surcharge.

3. Determine length of reinforcement L_T and L_B required from chart B.

Figure 3-75.—Chart procedure for confirming reinforced slope design.

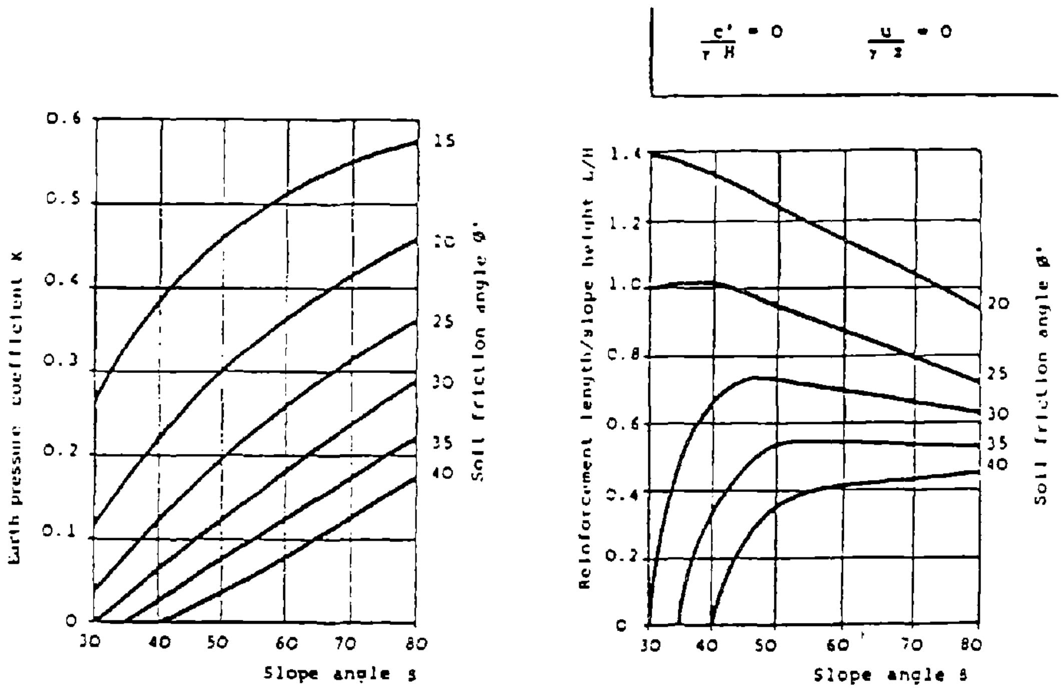


Figure B-83. Simplified design chart with pore pressure coefficient equal to zero. [Jewell et al., 1984b]

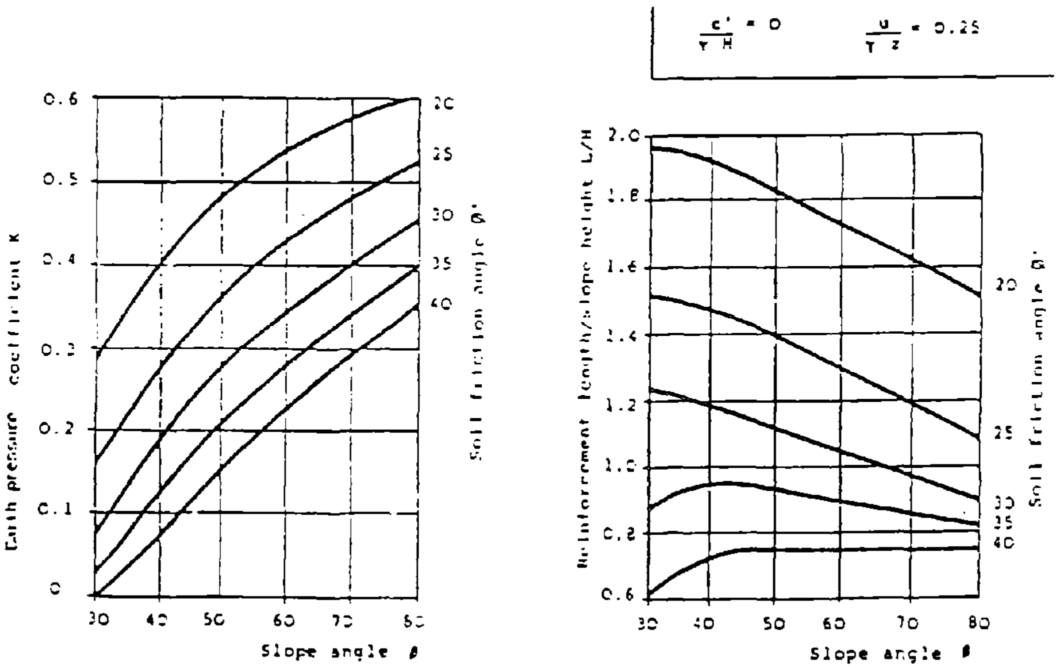


Figure B-84. Simplified design chart with pore pressure coefficient equal to 0.25. [Jewell et al., 1984b]

(from Mitchell and Villet-NCHRP 290, 1987)

Figure 3-76.—Simplified design chart with pore pressure coefficient equal to 0.25.

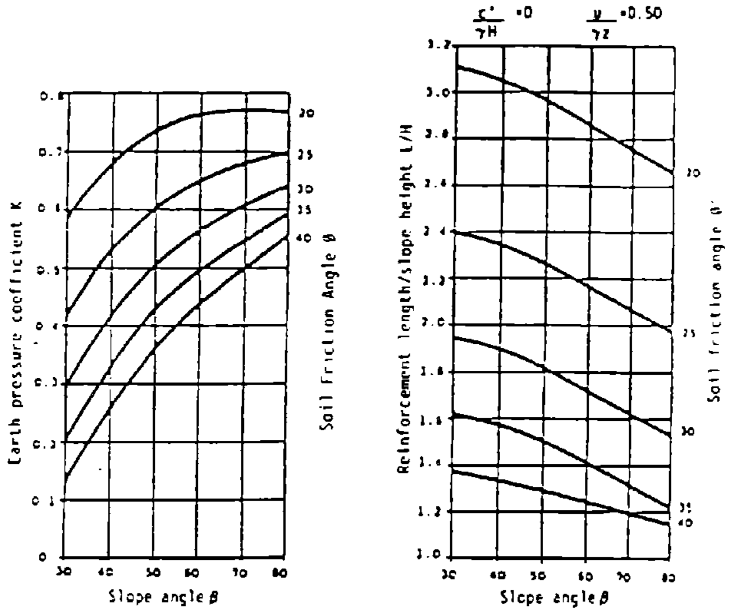


Figure B-85. Simplified design chart with pore pressure coefficient equal to 0.50 [Jewell et al., 1984b]

7.2.6 Design Chart Example

The example is taken from the technical guidelines for the design and construction of steep reinforced embankments over stable foundations issued by Netlon Limited [1984b].

The problem is to determine a suitable reinforcement layout to provide equilibrium in a 20-ft high embankment with a slope angle 70 deg built from compacted granular soil.

1. The embankment dimensions are shown in Figure B-86 and there is no surcharge loading.

2. The large strain value of shear strength is taken to be $c' = 0$, $\phi' = 29°$, and the maximum density $\gamma = 121$ pcf. The slope is fully drained and $r_u = 0$.

3. The earth pressure coefficient from Figure B-83 is $K = 0.25$. The reinforcement length to embankment height ratio $L/H = 0.69$ from Figure B-83, giving a reinforcement length $L = 13.5$ ft.

4. The in-service characteristic strength suggested by the manufacturer of Tensar SR2 in granular soils is the ideal laboratory value (2,000 lb/ft) divided by a partial factor 1.1 to 1.4 dependent on soil type (see Sec. 4.4.3) to account for possible construction damage, creep, and long-term loss of strength. In this case taking a partial factor 1.3 gives an in-service design strength, $\frac{2{,}000}{1.3} = 1{,}530$ lb/ft.

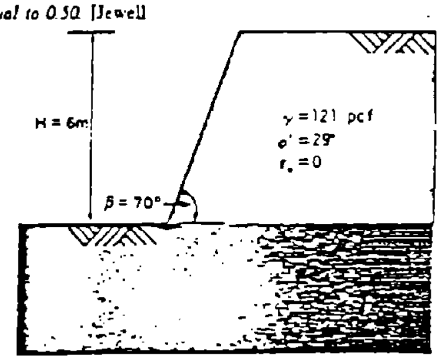


Figure B-86. Slope geometry and soil properties for the design example. [Netlon Ltd., 1984b]

5. Selecting an overall design safety factor 1.35 gives a factored design strength $P = \frac{1{,}530}{1.35} = 1{,}130$ lb/ft

6. An assumed minimum spacing for the reinforcement $S_{v,min}$ = 9 in gives a value for the spacing constant $Q = \frac{P}{K\gamma S_{v,min}} =$ 50.6 ft.

7. The depths and thickness of the zones of equal reinforcement spacing may be calculated (see Fig. B-82) and these are shown in Figure B-87.

8. The total horizontal force required to provide equilibrium is $T = \frac{1}{2} K\gamma H^2 = 5{,}860$ lb/ft

9. The calculated number of reinforcement grids is 8 (Table B-7, Fig. B-88), giving the check $\frac{5{,}860 \text{ lb/ft}}{8} = 732$ lb/ft < 1,130 lb/ft, which is sufficient.

Figure 3-77.—Simplified design chart with pore pressure coefficient equal to 0.50.

Spacing of Grids in Zone (ft)	Depth to Bottom of Zone (ft)	Thickness of Zone (ft)
Svmin=0.74	Q=50.6	—
2Svmin=1.48	Q/2=25.3	20-16.9=3.1
3Svmin=2.22	Q/3=16.9	16.9-12.7=4.2
4Svmin=2.96	Q/4=12.7	12.7-0=12.7
		Height=Σ=20.0

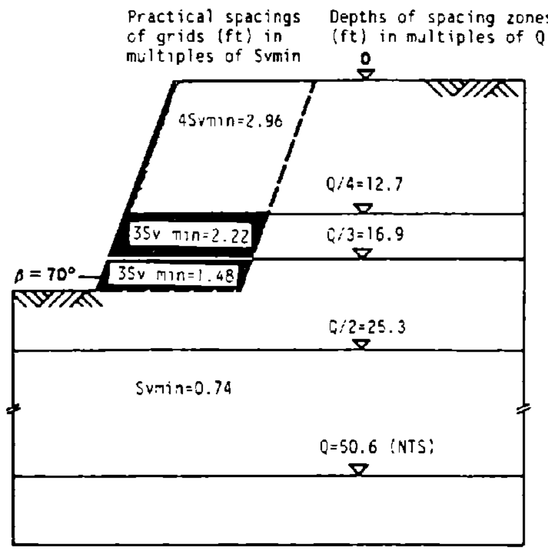


Figure B-87. Depth and thickness of reinforcement spacing zones for the design example. [Netlon Ltd., 1984b]

Table B-7. Number of reinforcement layers in each zone as drawn in Figure B-88. [Netlon Limited, 1984b]

Spacing in zone (ft)	Thickness of zone (ft)	Calculated number of grids	Chosen number of grids	Remainder (ft)
		Initial grid at base	1	
1.48	2.79	2.79/1.48 = 1.89	1	0.59 x 1.48 = 1.32
2.21	4.23	(4.23 + 1.32)/2.21 = 2.50	2	0.8 x 2.21 = 1.10
2.95	12.70	(12.7 +1.10)/2.95 = 4.67	4	0.67 x 2.95 = 2.0
		Total	8	

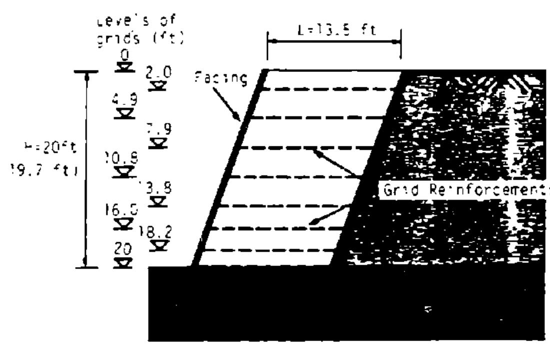


Figure B-88. Positions of eight reinforcement layers as calculated in Table B-7. [Netlon Ltd., 1984b]

Figure 3-78.—Depth and thickness of reinforcement spacing zones for the design.

- Reinforcement length of each reinforcing layer is the sum of the length to the critical failure plane plus the embedment length. Embedment length is the length needed to provide adequate pullout resistance in each layer of reinforcement. The tensile capacity of any reinforcement layer is the minimum of either its allowable pullout resistance behind the potential failure plane or its allowable design strength. Embedment length is determined either using the pullout resistance formulas based on pullout tests, or total reinforcement length at the top and bottom of the embankment is determined using the chart solutions (see figure 3-75). The lengths are then interpolated between these values. Note that the minimum recommended embedment length behind the critical failure plane is 3 feet.

- The overall external stability of the reinforced mass against sliding, deep seated failure, foundation settlement, and seismic events. On relatively tall, shallow fills, such as for debris slide repairs, the design of the reinforced zone may be controlled by overturning or sliding considerations. In this case, additional benching or terracing of the soil-fill interface may be needed.

Appropriate minimum factors of safety for the various aspects of internal and external design are available in the FHWA design procedures. Commonly accepted factors of safety for design are:

- External sliding = 1.5

- Internal/external slope stability = 1.3

- Pullout resistance = 1.5–2.0

- Internal geosynthetic materials strength factors

    Installation damage = 3

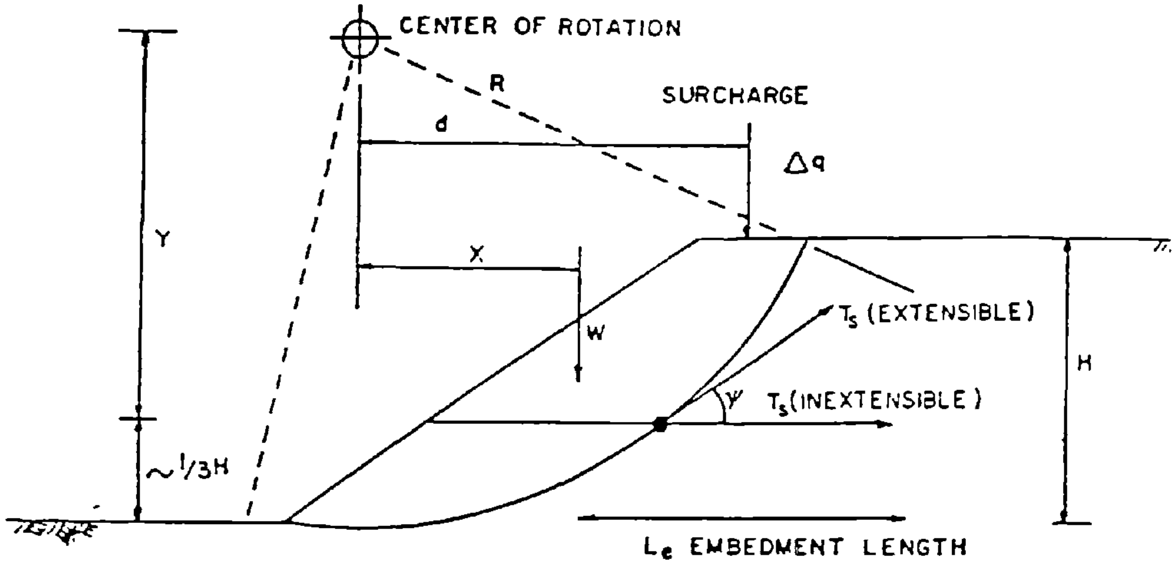
    Creep deformation = 5

    Chemical degradation = 2

    Biological degradation = 1.3

    Connection strength = 2

Recommended seismic design procedures should utilize a pseudostatic analysis as outlined by the FHWA (Christopher et. al., 1990) or discussed by Bonepart (1986). A horizontal pseudostatic force, which is some percentage of the slice weight, is applied to each slice or volume in the analysis. Internal, external, and compound failure modes are then analyzed with the additional seismic forces. A minimum factor of safety of 1.1 is typically used.



Factor of safety of unreinforced slope:

$$F.S._u = \frac{\text{Resisting Moment } (M_R)}{\text{Driving Moment } (M_D)} = \frac{\int_0^{L_{SP}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)} \qquad (58$$

where: W = weight of sliding earth mass
L_{SP} = length of slip plane
Δq = surcharge
τ_f = shear strength of soil

Factor of safety of reinforced slope:

$$F.S. = F.S._u + \frac{T_s \cdot D}{M_D} \qquad (59$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers
D = moment arm of T_s about the center of rotation
= R for extensible reinforcement
= Y for inextensible reinforcement

Figure 3-79.—Rotational shear approach to required strength of reinforcements.

3L.4 Materials

3L.4.1 Internal Reinforcement Materials

Internal reinforcement material most commonly used in reinforced fills is a uniaxial or biaxial geogrid material made of polymers such as polyester, polypropylene, or polyethylene. Also welded wire mesh, chain link fencing, and geotextile materials have been used for reinforcement. Each have their own tensile strength, soil interaction, creep, and elongation characteristics which should be considered in the design process. Stress-strain characteristics of polymer geogrids or geotextiles should ideally be as similar to soil as is practical. Materials characteristics used for design are commonly available from the manufacturer, but test methods should follow those developed as industry standards, such as Task Force 27 (AASHTO, 1990) and Geosynthetic Research Institute (GRI, Drexel University, 1990) standards of practice and test methods.

Allowable design tensile strength of reinforcing materials, particularly geogrids, is determined from an ultimate strength laboratory value modified by several factors of safety, including factors for creep deformation, installation damage, chemical and environmental degradation, and seams and joints weaknesses. Creep tests should be conducted for a minimum duration of 10,000 hours and extrapolated to a minimum design life of 75 years.

Pullout resistance, which reflects the specific soil-reinforcement interaction, is a significant design consideration. An advantage of geogrids or a wire mesh is its high percentage of soil-to-soil contact through the grid. Long-term pullout performance is a function of pullout capacity (preferably determined from pullout tests), allowable displacement, and long-term displacement (depending on creep characteristics).

Welded wire mesh or other metals such as chain link fencing, which are considered inextensible, will have less creep and long-term displacement than extensible polymers (geogrids), different load-deformation characteristics, and will be more susceptible to corrosion. Galvanized coating or additional sacrificial metal are commonly used to retard or compensate for corrosion in metal reinforcement. Corrosion and durability issues both for metals and polymers are thoroughly addressed in FHWA-RD-89-186, *Durability/Corrosion of Soil Reinforcement Structures*, by Elias (1990).

Expanded fiberglass strands mixed pneumatically with backfill soil have been used both as primary internal reinforcement for the entire reinforced fill and for reinforcement of the zone at the fill face. The method, originally developed in Europe under the name "TEXSOL" (Leflaive, 1985), has been used by the Forest Service in small fill face applications (Burke, 1988), and laboratory testing of its soil-interaction properties has been conducted by the California Department of Transportation Materials Laboratory. Presently, the design methodology is evolving, but the concept appears promising and relatively inexpensive.

3L.4.2 Backfill Material

Backfill gradation requirements recommended by the AASHTO Task Force T27 for reinforced slopes are:

Sieve size	Percent passing
4 inches	100–75
No. 4	100–20
No. 40	0–60
No. 200	0–50

These gradation requirements are broad and can include excellent to marginal quality material. Plasticity index should not exceed 20, and soil pH range should be 3 to 9 to avoid excessive corrosion.

Coarse material (rock) should be limited to a size of approximately one-half the lift thickness. When rock coarser than 3/4 inch is present in the backfill, the "installation damage" factor of safety may need to be increased over the commonly recommended factor, depending on the type of reinforcement used.

Local materials are commonly used in low volume road projects for backfill in retaining walls and reinforced embankments. Local "marginal" materials used on projects have varied from clayey sands to silts (SM, SC, and ML—unified soil classifications). The use of marginal backfill has been acceptable, but it can present problems in construction and long-term performance. Fine-grained soils are moisture sensitive, so close compaction control is needed to meet specified densities and the desired strength, and drainage is more important and difficult to achieve. A density of 95 percent of ASSHTO T-99, which is typically specified, generally produces satisfactory results.

3L.5 Fill Face Slope and Facing Needs

With reinforcement, the final fill face most commonly designed and achieved has a 1H:1V slope. However, this slope can vary between 1-1/4H:1V and 1/2H:1V, depending on the soil type used and extra measures taken. Approximately, a 1H:1V slope is the steepest slope face achieved using a dominantly granular, low plasticity backfill material typical of mountainous terrain. Because the outer edge of the fill face is unsupported, good compaction in this area is very difficult to achieve. Without adequate density, soils placed on a steeper slope will typically not hold, and local fill face instability will occur. Attempts to construct an unsupported 1/2H:1V fill face have failed, even using a slightly clayey soil, and a 1H:1V slope was the end result.

Use of a material with some cohesion may allow a somewhat steeper slope to be constructed, but performance is probably still controlled by construction limitations at the fill face. In very granular, nonplastic material, such as decomposed granitic soil, experience has shown that a 1 1/4H:1V face is the most appropriate stable slope. A steeper slope will either ravel or it will need some additional type of support, such as a reinforcement material wrapping around the face. Wrapping material has included either the geogrid being used for reinforcement or a heavy erosion control matting or netting. The TENSAR Corporation has constructed 3/4H:1V-slopes using the primary reinforcement grid wrapped around the face in up to 3-foot lifts. However, if abundant wrapping is needed, a retaining structure may become more economical. Eight-inch (200 mm)-diameter quarry rock has been used on fill faces to achieve a stable 1:1 slope. Also, a variety of prefabricated concrete blocks is available for facing material. Note, however, that if much wrapping or facing material is needed, a retaining structure may be more economical.

Promise exists for achieving a 1/2H:1V slope face on low fills with use of biotechnical measures, such as vegetal stabilization. Experience with fills up to 11 feet high has shown that a 1/2H:1V slope could be constructed, without forms, by placing a mixture of straw, clay-riched soil, manure, and seed in 1-foot lifts along the outer couple feet of the fill face between geogrid reinforcing layers (Burke, 1988). The straw provides tensile strength to support the steep slope and erosion protection until the seed germinates, which further adds root support.

A 3/4H:1V slope face has been achieved by pneumatically shooting expanded fiberglass strands into the outer couple feet of the fill face as fill material was being dumped with a backhoe. This method, described previously, appears promising and inexpensive.

On any newly constructed fill face, particularly when oversteepened, both surface water control and erosion protection are needed. Surface water should be collected above the slope and channeled around it. At a minimum, some type of erosion control blanket or matting should be placed on the slope. The reinforcement material, such as geogrid, can be used to wrap around the face. This technique provides a very durable but expensive erosion control layer, and ideally it should still be used in conjunction with some finer erosion control matting. Erosion control and revegetation measures to protect the fill face should be integral parts of the reinforced slope design, and they should not be left to the discretion of the contractor.

3L.6 Drainage Requirements

Most reinforced fills already built have had drainage provisions added, either to remove local ground water or to ensure that the backfill will remain in a drained condition (as assumed for typical retaining structure design). Typically, chimney drains or geocomposite drains are installed behind the fill. Layers of drainage material, alternatively could be incorporated into the fill.

3L.7 Construction Sequence

Construction of reinforced fills is relatively simple and fast once the design is laid out to the following specifications:

- First the area is excavated to grade and the subgrade smoothed and compacted. To ensure that a suitable foundation exists, the excavation should be inspected by a geologist or a geotechnical engineer.

- Typically a horizontal layer of the reinforcing material is initially placed on the subgrade to the dimensions and orientation as shown on the drawings or as directed by the engineer. Correct orientation should be confirmed by the manufacturer to ensure that the direction of maximum tensile strength is towards the embankment face. The material may be secured with staples, pins, stakes, or backfill material.

- Drainage, typically a chimney drain using gravel or geocomposite material, is placed against the back of the excavation and brought up as necessary to stay above the lifts of backfill material. Perforated collection pipes and nonperforated drain pipes are installed as required on the drawings to remove the water.

- Placing and compacting layers of backfill material are done similar to normal earthwork lift operations. Backfill should be spread and compacted so that the reinforcing fabric or geogrid does not move, and that equipment does not operate directly on the reinforcing material. Compaction on clay-rich soils should be in 6- to 8-inch lifts and granular soils should be 9- to 12-inch lifts. Lift thickness may be governed by reinforcement spacing. Soil is normally compacted to 95 percent of the AASHTO T-99 maximum density. Large, smooth drum vibratory or rubber tire rollers should be used; sheepsfoot rollers, however, may damage the reinforcement.

- Facing elements and/or erosion control measures are either brought up or wrapped around as the lift sequence progresses, or facing is added upon completion of the basic structure.

- The sequence, spacing, and length of reinforcement layers and fill material lifts are continued to the dimensions shown on the drawings until the structure is completed. Secondary reinforcement strips are commonly placed along the edge of the fill face as each lift (or two lifts) are placed.

Correct orientation of the reinforcement is critical to the design of the structure, so it should be double-checked and confirmed. Also, since it is almost impossible to work with a single reinforcement roll width, either overlapping or splices must be considered. Splices should be avoided along the reinforcement in the direction parallel to the face. The type of splice or overlap distance will depend on the type of reinforcing material and direction of splice. They should be installed in accordance with the manufacturer's directions.

3L.8 Cost

Because no facing materials are needed, the cost of a reinforced embankment should be less than the cost of a retaining structure built to stabilize that same site. The cost will be greater than that of conventional fill construction because of the addition of the reinforcement material. Because of contractors' general limited familiarity with this design concept, and typically small project applications, construction costs to date have not been as low as this concept should dictate. On large commercial projects, savings of 30 to 40 percent have been realized over the use of retaining structures. Fortunately, the use of reinforced embankments is becoming more widespread and accepted.

Bid prices between 1987 and 1992 have averaged from $10 to $15 per cubic yard for controlled compaction of the material, and between $4 and $8 per square yard for installed reinforcing geogrid, the primary and intermediate reinforcement members, and drainage.

3M Low Cost Shoulder Repair

3M.1 Introduction

Many thousands of miles of low volume roads have been constructed using lower standards which have allowed for high, long-term economic risk. Typically, this involves an old embankment which was constructed using a side-cast construction technique, that is, little or no control of compaction and/or deleterious materials. On such roads, it is often eventually common for subsidence to develop along the roadway surface, particularly along the shoulder. Subsidence may be due to the following reasons:

- Densification of particles due to vibration from traffic or seismic loads.
- Plastic deformation of soil under its own weight.
- Deterioration of soil/rock due to weather conditions resulting in rearrangement of particles.
- Hydraulic transport of soil particles within the embankment due to ground water and/or change in drainage conditions.
- Deterioration of organic debris resulting in voids that allow for the caving-in or the washing of embankments into the voids.

In many cases, the consequences of this type of lower initial cost construction technique has been higher maintenance needs, or if sufficient justification exists, high cost reconstruction involving treatments, such as reinforced embankments or retaining walls. The purpose of this section is to present a simple, cost-effective alternative to shoulder subsidence repair that meets the following criteria:

- Treatment can be accomplished by a force account maintenance crew or another crew with similar skills.
- Simple design procedures which can be performed by an experienced civil engineering technician or journeyman civil engineer without specialized knowledge of soil mechanics.
- Treatment extends less than 5 feet below the road surface for excavated treatments; 30 feet for launched soil nails.
- Qualified professionals judge that the consequences of the potential failure is low.

The primary advantages of the alternative treatment presented in this section are as follows:

- Low life-cycle cost effectiveness.
- Minimal ground disturbance and, hence, low planning costs.

- Minimal traffic disruption.
- Simple design procedures and, hence, low design and contract preparation costs.

3M.2 The Geosynthetic Reinforced "Deep Patch"

The typical road prism for which the geosynthetically reinforced deep patch is appropriate consists of old, loose, side cast embankment that often contains decomposing organic debris. The typical site has minor subsidence which may be a safety hazard and requires frequent patching, crack sealing, and/or a leveling course of surfacing. Settlement is often accelerated by a crack on the road's surface, allowing water to infiltrate into the embankment. The problem is often chronic, but it is not severe enough to require conventional reconstruction techniques. Figure 3-80 shows the general situation.

The following two cases, case 1 and case 2, are variations on the theme; that is, they vary according to the judged mode and severity of potential failure.

Case 1: Settlement of Side Cast Embankment

In case 1, the risk that the embankment will proceed rapidly downslope is low and is often a chronic maintenance problem. The primary cause of subsidence can be characterized as settlement. Settlement could be due to a combination of many factors previously mentioned.

In case 1, it is judged that stability is not a significant concern and that the problem can be treated as settlement.

A very important part of the design is to determine the nature of any drainage problem(s) and provide an effective solution, if applicable.

In case 1, a simplified geosynthetic reinforced deep patch design could be used. A single layer of geosynthetic material is placed approximately 3 feet below the road surface, between two cushioning layers of compacted aggregate. Figure 3-81 shows a typical construction drawing. If open cracks are encountered in the bottom of the excavation, they should be filled with a slurry grout applied under low pressure.

Since nothing is being done to improve the settlement characteristics of the underlying material, it is expected that the outside portion of the patch foundation will continue to settle—but at a slower rate and without the redevelopment of the surface crack. Since the settlement continues after construction and stress is applied to the geosynthetic, the materials must be designed for tensile strength and for the pullout.

A simplified geosynthetic design for case 1 is shown in figure 3-82. The tensile strength of the geosynthetic is designed to meet the long-term allowable design load per length of road, S_l, such that $S_l = C_f X_l d_\gamma$.

in which

C_f = Friction coefficient of geosynthetic/soil interface
X_l = Geosynthetic portion outside of projected failure surface
d = Geosynthetic depth below road surface
γ = Unit weight of backfill material on top of geosynthetic

Note: this equation is conservative due to the sloped surface at the embankment face.

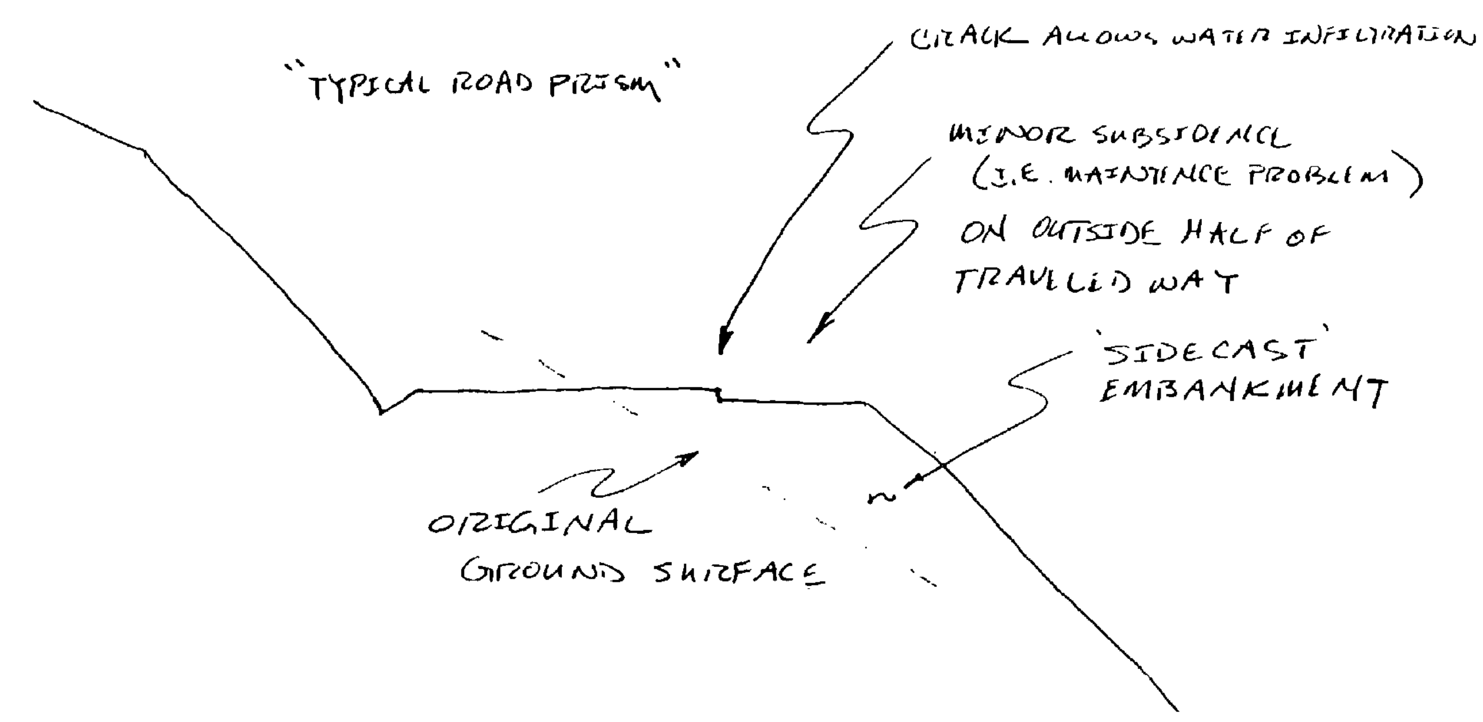


Figure 3-80.—General problem description.

The factor of safety against pullout, *FS* (pullout), can be

$$FS\text{ (pullout)} = S_2/S_1 > 1.0 \qquad (3\text{-}124)$$

in which S_2 equals the pullout resistance per length of road; S_1 equals the pullout load (allowable design load per length of road).

By substituting values and simplifying, it can be determined that

$$FS\text{ (pullout)} > 1.0 \text{ when } X_2 > X_1$$

in which X_2 equals the portion of geosynthetic outside of the projected failure surface.

Figure 3-83 shows the construction sequence for the repair scheme.

Case 2: Settlement of Side Cast Embankment as an Incipient Failure

In case 2, the risk that the embankment will proceed rapidly downslope is judged to be sufficiently high to warrant a more detailed stability analysis and design. Therefore, if the consequences of a potential slope failure are significantly high, then a case 2 analysis should be performed by a qualified professional.

A case 2 situation is shown in figure 3-84 and generally has similar physical characteristics to those in case 1. In addition to the simplified geosynthetic design in case 1, a slope stability analysis is performed, as shown in figure 3-85.

The stabilizing component resulting from the geosynthetic is limited by the frictional resistance mobilized along the geosynthetic/soil interface between the potential failure surface and the embankment face. Note that the frictional strength should be expected from shear development only on the underside of the geosynthetic and may, therefore, be less than frictional coefficients due to geosynthetic pullout and the available published data.

In the case of an incipient failure analysis, a modified Bishop or other circular arc search routine can be used. To model an incipient failure condition, strength values, C and ϕ, are chosen so that the minimum factor of safety for the critical failure arc is equal to 1.0, using the expected critical ground water condition (refer to figure 3-85). A resisting moment due to the force developed in the geosynthetic is used to calculate an increase in the factor of safety above the incipient failure condition. This equation for the increase is

$$\Delta FS = L(min) \times S(geo)/DM \qquad (3\text{-}125)$$

in which *L(min)* equals the minimum moment arm between the center of curvature and the geosynthetic/failure surface intersection; S(*geo*) equals the strength developed from the geosynthetic/soil interface, and *DM* equals driving moment determined in the modified Bishop analysis.

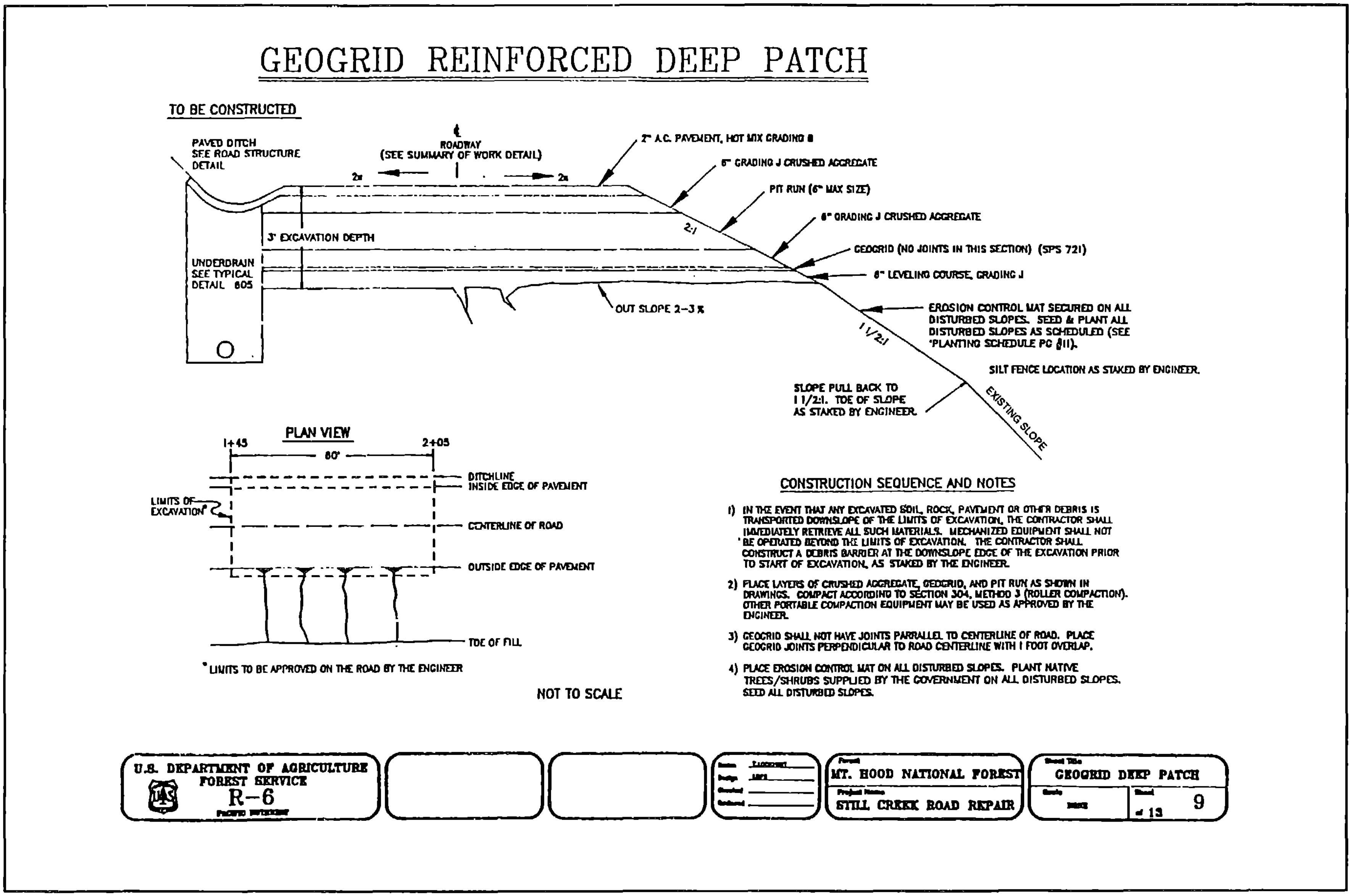


Figure 3-81.—Sample contract drawing.

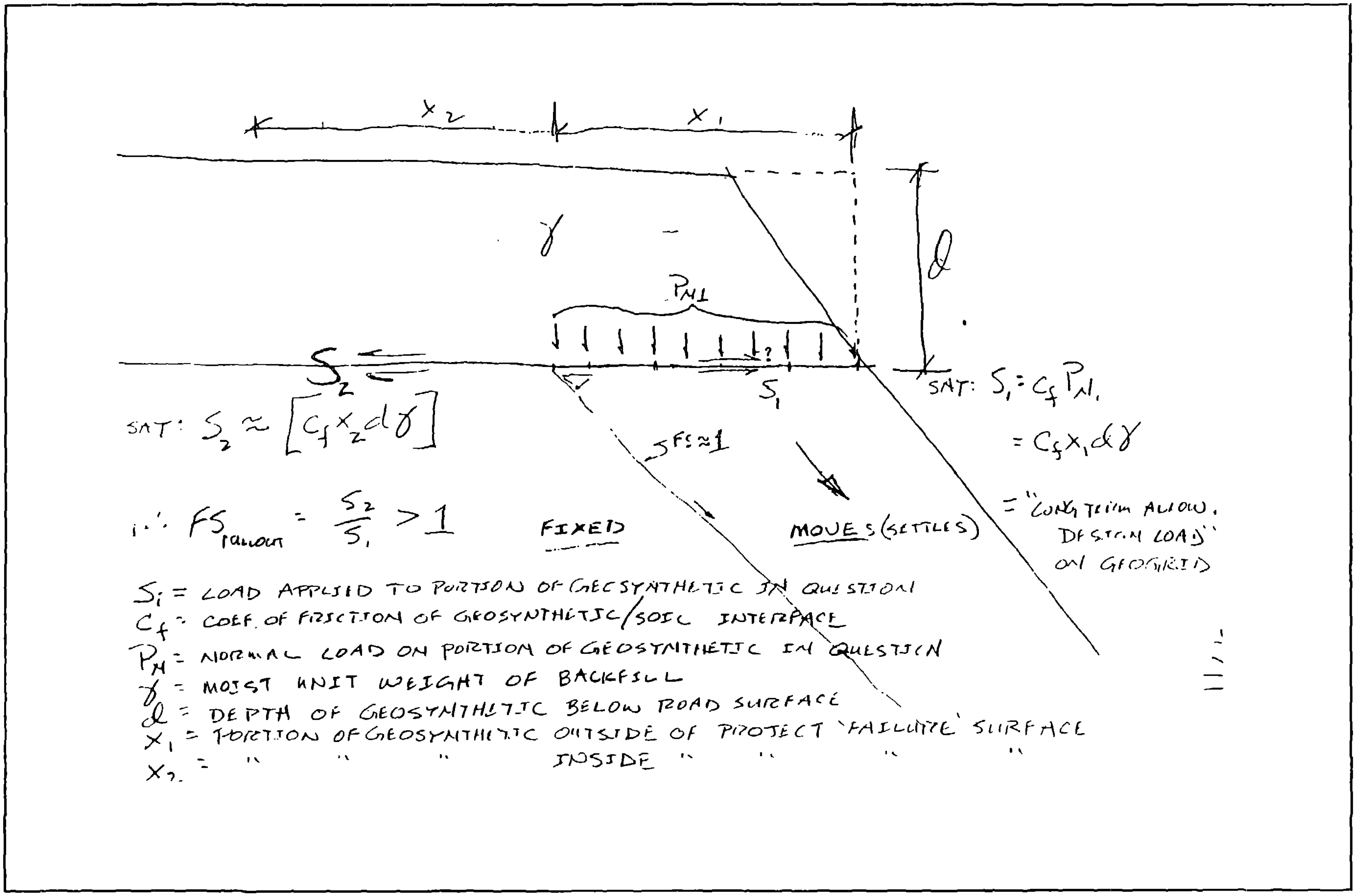


Figure 3-82.—Simplified geosynthetic design.

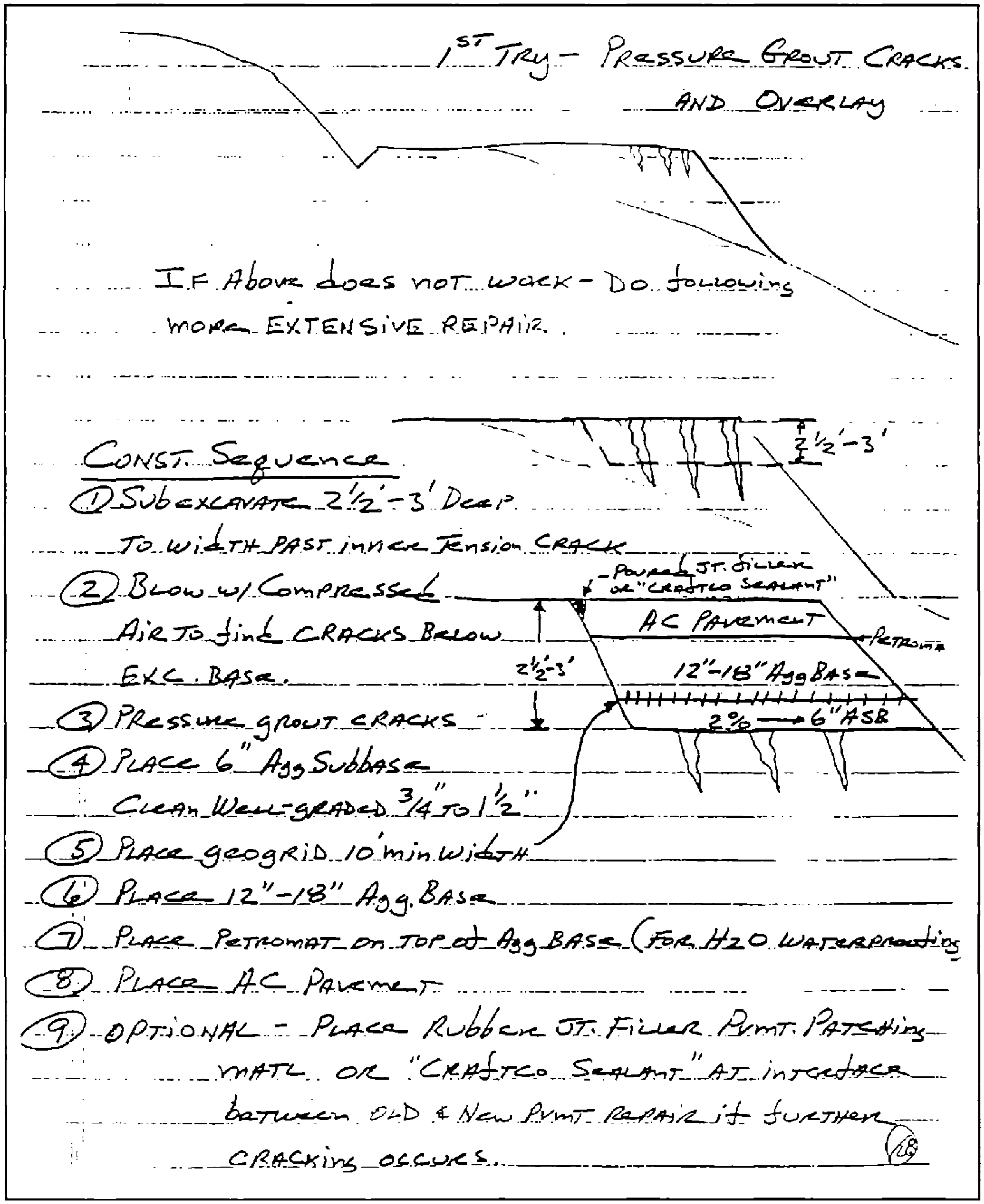


Figure 3-83.—General comments—shoulder fill settlement (repair scheme).

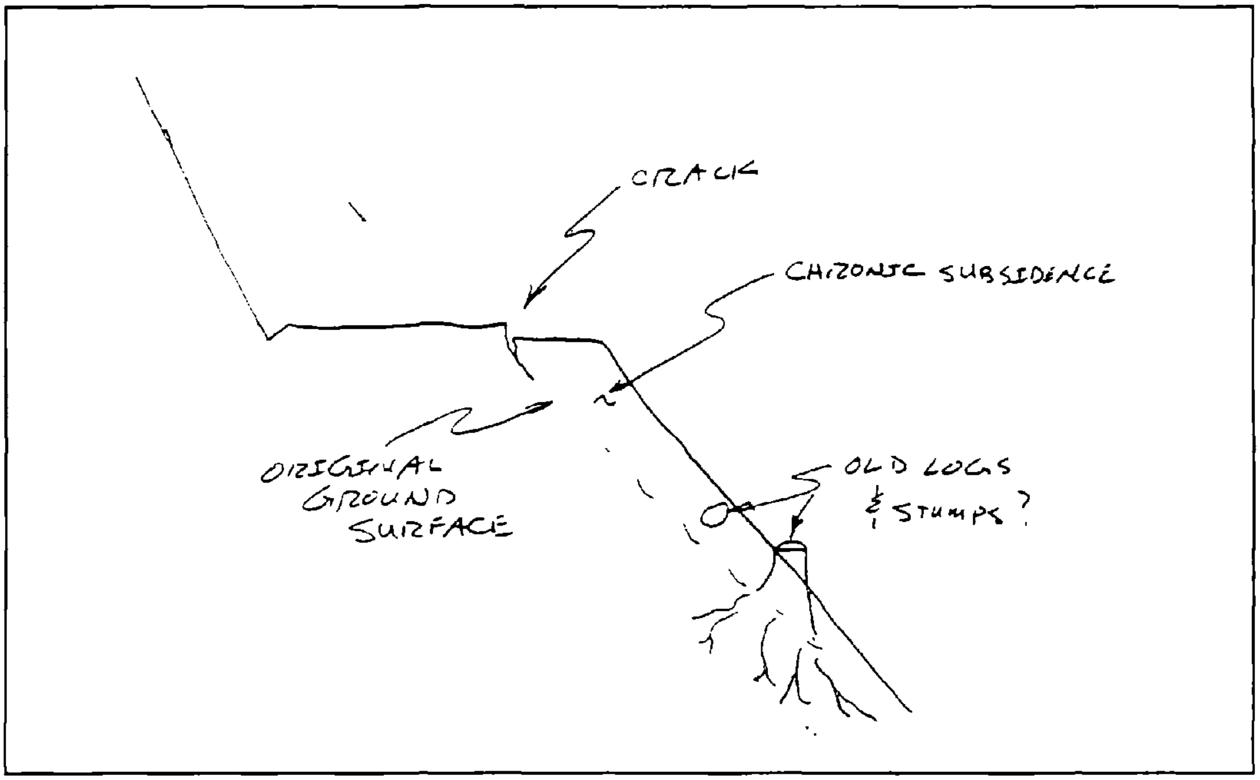


Figure 3-84.—Case 2 description.

3M.3 Launched Soil Nails

A new technology called launched or "ballistic" soil nailing was succesfully demonstrated in the Western United States in 1992 in a demonstration project facilitated by the USDA Forest Service. The technology appears most suited to reinforcing and stabilizing shallow landslides (less than 15 feet deep) on cut and fill slopes of highways and low volume roads. The 1992 project demonstrated the capability of the equipment to install 1.5-inch-diameter by 18-foot-long galvanized steel reinforcing nails in a wide variety of soil and field conditions. Design charts for stabilizing roadway shoulder failures/embankments were developed as part of that demonstration. The design charts are in appendix C. Figure 3-86 shows a schematic of the equipment (Steward, 1994).

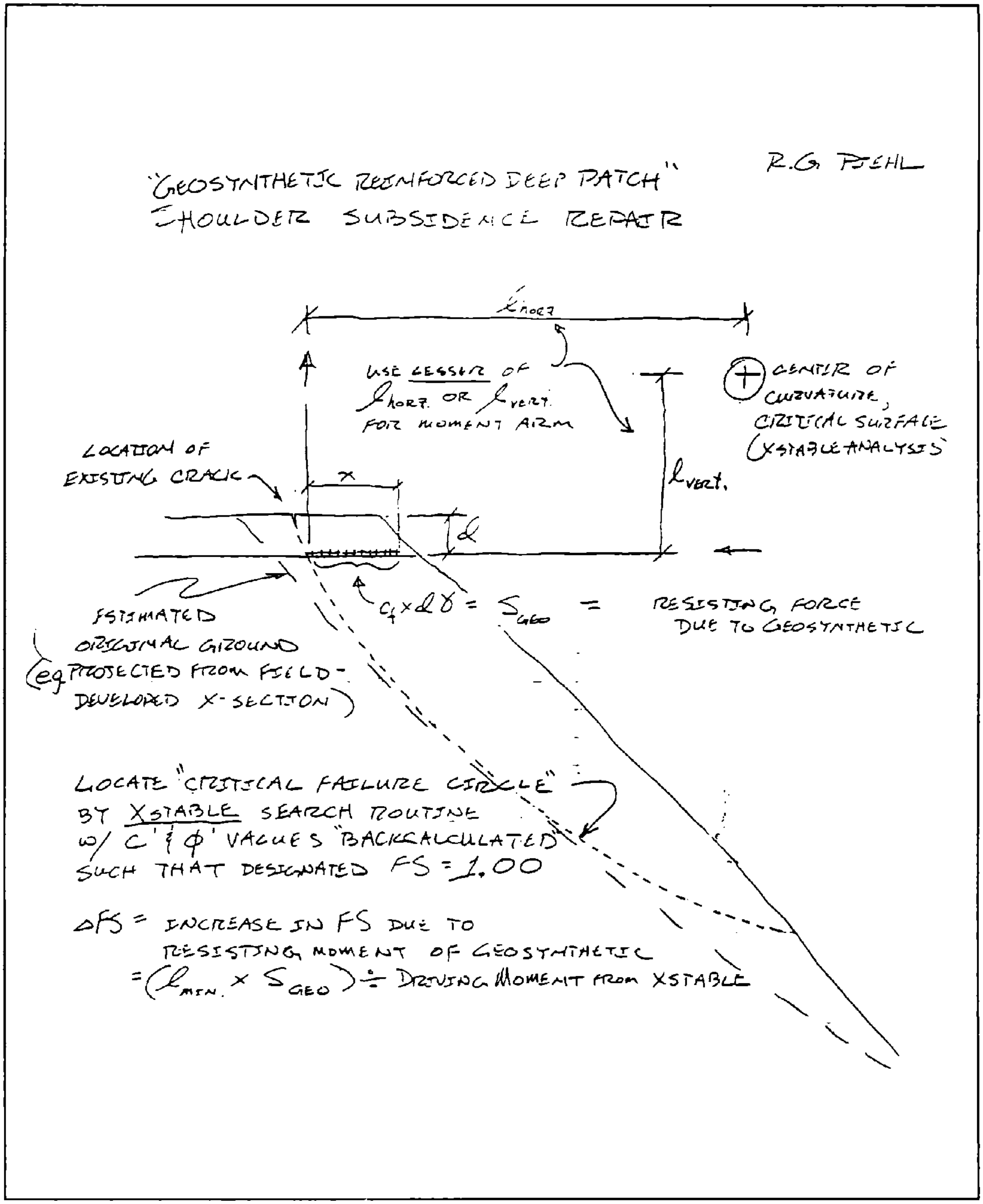


Figure 3-85.—Stability analysis of geosynthetic reinforced deep patch.

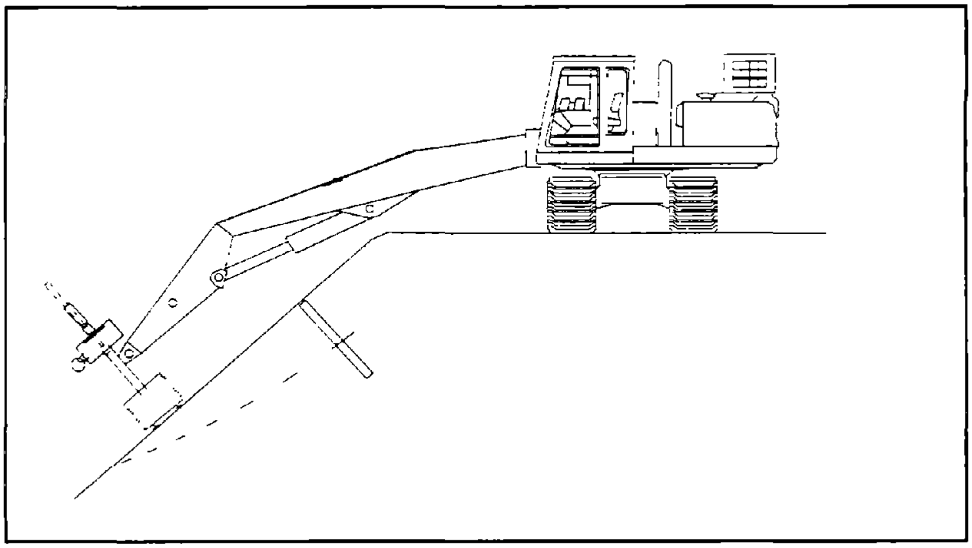


Figure 3-86.—Illustration of the soil nail launcher attached to a hydraulic excavator.

3N Soil and Rock Anchors

3N.1 General

Retaining wall anchorages are fixed objects embedded in soil and/or rock behind a potential failure zone. The anchors develop their holding capacity either by mobilizing the passive resistance of the soil or from the bond strength of a grout column or mechanical anchorage with the surrounding rock. Passive earth anchors may take the form of piles, reinforced concrete or sheet pile walls, smaller structural bars and plates, deadmen (embedded concrete blocks), or injection-grouted soil anchors. A factor of safety of 1.5 to 3 is common for passive anchors. The factor of safety of grouted rock anchors depends on rock quality, and it varies from 3 to 10.

The forces are transferred from the wall to the anchorage through a system of tie rods. The tie rods are generally steel bars, tendons, or straps that have been treated to protect them from oxidation and corrosion. Failures of anchored walls are typically the result of the progressive deterioration of the tie rod and/or tie rod connection.

Full length grouting or a wrapped grease or bitumen coating may be used to protect anchorage. Some engineers prefer to use a reduced design stress or to over-size the anchor bar to allow long-term reduction of the section area due to oxidation. The allowable tensile stress in an anchor rod is generally taken to be 0.6 of the steel's yield stress. For threaded bars, the rod area is taken at the thread root. The tie rods are generally located at distance, *d*, measured down from the top of the wall in which *d* is generally in the range of H/4 to H/3 (see figure 3-87).

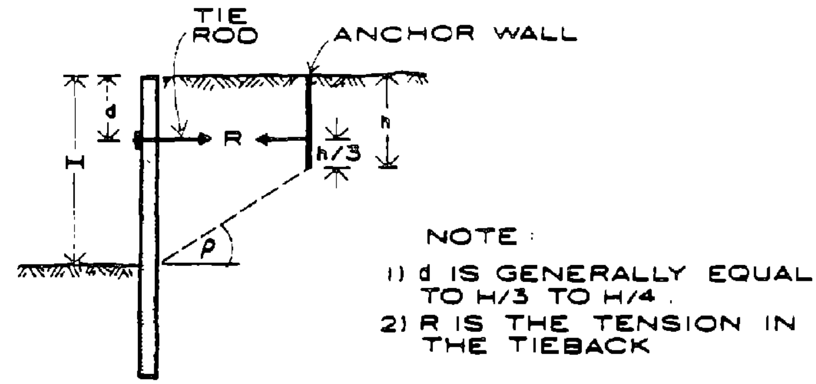


Figure 3-87.—Schematic anchored wall.

Loads are transferred from the wall face to the anchor rods through horizontal structural steel shapes called "walers." On occasions, rolled steel channels are used in a back-to-back position, as shown in figure 3-88, so that the tieback may fit between the channels. If a wide flange section is selected as a waler, eccentric loading may result in significant torsal stresses. In this case, web stiffeners may be required at the anchor connection.

Walers may be designed as simple spans and the maximum bending moment computed using

$$M_{max} = \frac{R}{8} l \tag{3-126}$$

in which R is the tension in the tieback and l is the horizontal spacing between tiebacks. Analysis of more complex beam loadings may result in lighter waler sections. The required section modulus of the waler is

$$S = \frac{M_{\max}}{F_b} \tag{3-127}$$

in which F_b is 0.66 of the steel's yield stress (for A36 steel, F_b is 24 ksi). To complete the waler-anchor rod assembly, the connection must be designed. The waler must be checked for web crippling, and the bearing plate must be of adequate thickness to distribute the load.

It is important to note that if the anchor rod or tie bar is not acting perpendicularly to the wall, a significant vertical force will be applied to the wall.

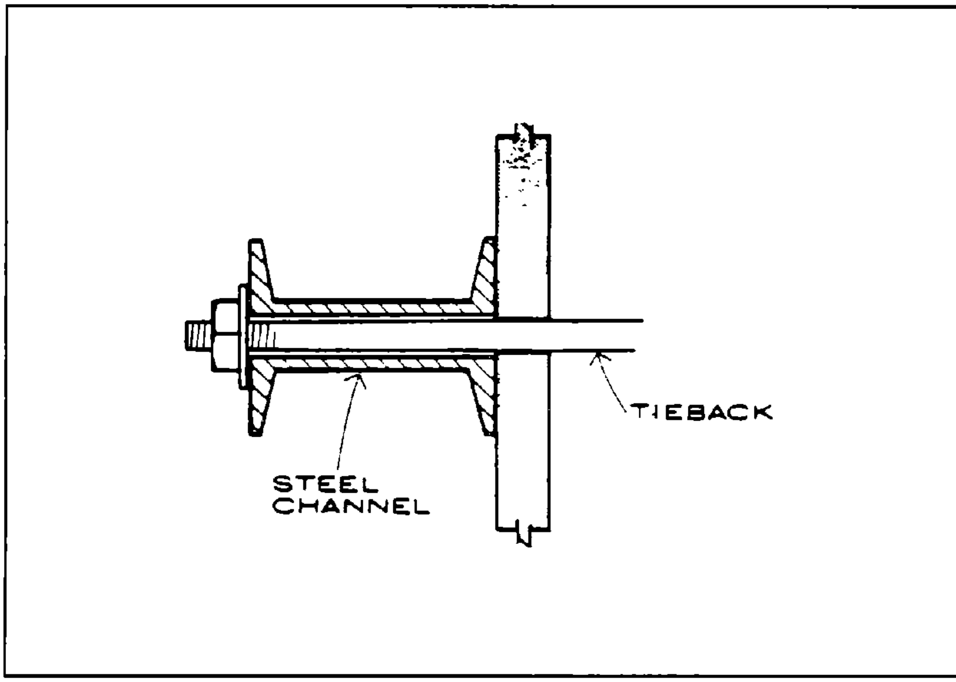


Figure 3-88.—Typical waler detail.

3N.2 Anchor Walls

Continuous walls designed as anchors should be analyzed in accordance with figure 3-89, using the Rankine method of analysis.

As shown in example (a) of figure 3-88, if the active and passive failure zones do not intersect, the wall is analyzed in a conventional manner and the maximum anchor tension that the anchor wall (soil) can support is

$$A_{pmax} = p_p - p_a \tag{3-128}$$

If the active failure wedge of the retaining wall intersects the passive wedge of the anchor wall (see example (b) in figure 3-88), then A_{pmax} of equation 3-128 must be reduced, as shown in

$$A'_{pmax} = A_{pmax} - 1/2\, h_2^2\, (K_p - K_a) \tag{3-129}$$

in which h_2 is the depth to the intersection of the failure wedges (see example (b) in figure 3-88).

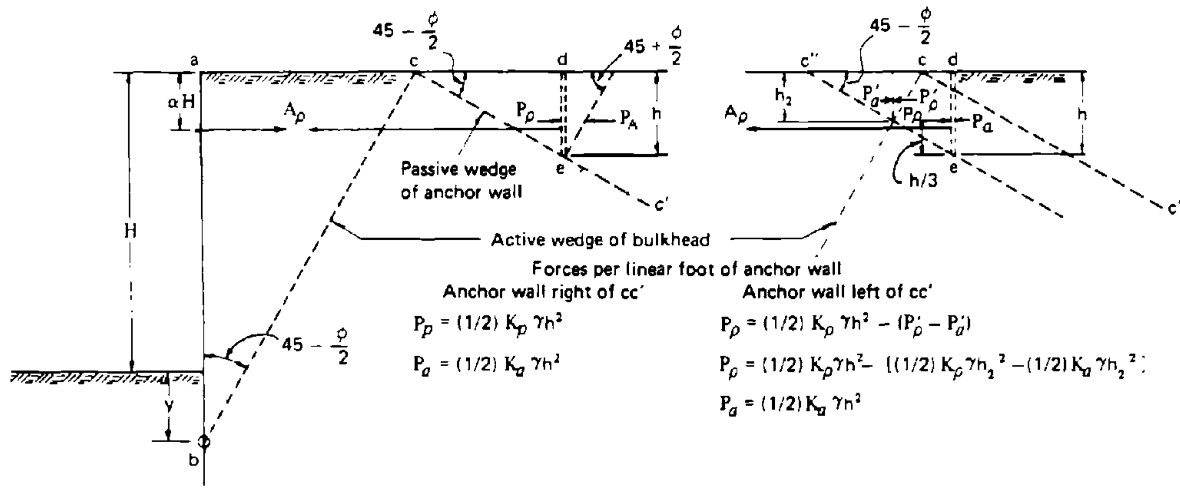


Figure 3-89.—Continuous anchor wall–earth pressure distribution (after NAVFAC, 1982).

3N.3 Short Deadman Near Ground Surface

In this analysis it has been assumed that the passive wedge of the deadman does not interact with the active wedge of the wall. Experiments have shown that the maximum capacity of the deadman (see figure 3-90), is

$$A_p \leq L(P_p - P_a) + \tfrac{1}{3} K_o \gamma (\sqrt{K_p} + \sqrt{K_a}) h^3 \tan\phi \tag{3-130}$$

in which

L equals the length of deadman,
h equals the height of deadman, and
K_o equals the at-rest earth pressure (granular soils).

For cohesive soils, A_p is

$$A_p \leq L(P_p - P_a) + 2\, ch^2 \tag{3-131}$$

in which c is the cohesion of the soil.

3N.4 Small Structural Plates and Bars

The anchors discussed in this section are typically used on horizontal sheet pile or vertical culvert pipe walls. It is generally accepted that the mode of failure of these anchors is, in effect, a localized bearing capacity failure rather than a generalized mobilization of the traditional passive wedge. Hence, the ultimate capacity of this type anchor is taken as equal to the bearing capacity of a footing, whose base is located at a depth equal to the distance from the ground surface to the proposed anchor.

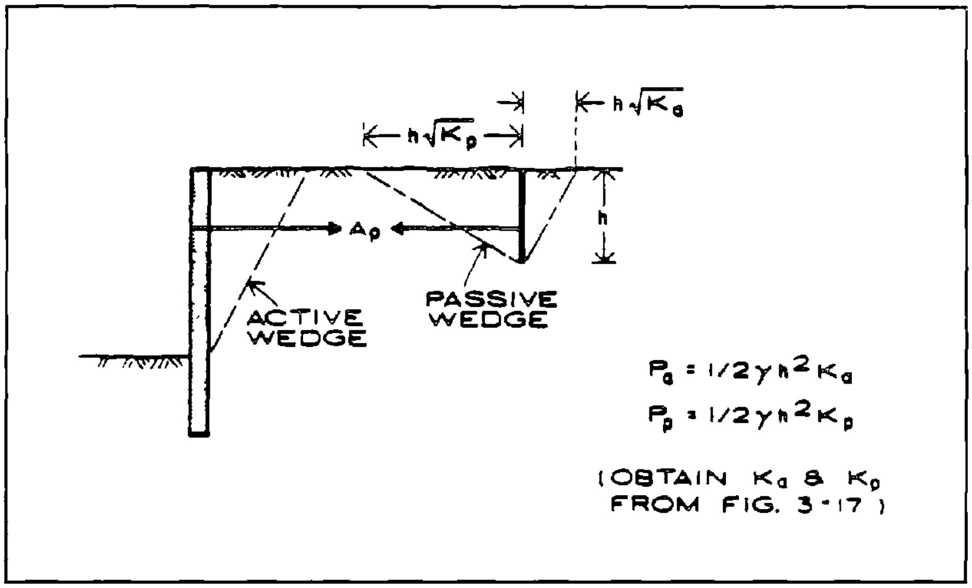


Figure 3-90.—Short deadman near ground surface.

The Terzaghi bearing capacity equations for local shear are recommended for computing the anchor capacity (see equations 3-40, 3-43, and figure 3-35). A factor of safety of 3.0 should be applied to the ultimate anchor capacity previously computed.

The designer should note that for a constant factor of safety against pullout and constant anchor size, the anchor spacing should decrease with increasing depth. However, the anchor spacing, *S*, should be greater than or equal to 3B (B is the width of the anchor plate or bar) to prevent the overlapping of stress bulbs.

The thickness of continuous bar anchors is determined by conventional structural analysis, assuming that there is a uniform soil pressure between the tie straps. The anchor should be analyzed as a continuous beam with moment redistribution for plastic bending. The frictional resistance developed on the tie straps between the critical failure plane and the anchor may be considered in this analysis. When analyzing the frictional forces on the tie straps, the designer should use the horizontal stress at a given depth, not the vertical stress, unless the strap is lying flat as in reinforced earth.

3N.5 Rock Anchors

The information in this section is taken from NAVFACS DM7. It is presented here for the convenience of the user.

There are two general categories of anchors:

(1) Grouted anchors where load is transferred from tendon to grout then from grout to soil. Load transfer is by either friction along a straight shaft or bearing against an underream or both.

(2) Mechanical anchor where load is transferred to soil by an expanding bit or other means.

The basic components (see figure 3-91) of a grouted ground anchor are:

(1) The prestressing steel, which may be one or more wire cables or bars; the bond length of the steel is the grouted portion of the tendon which transmits force to the surrounding soil or rock; the stressing length of the tendon is the portion which is free to elongate during stressing.

(2) The stressing anchorage, which permits the stressing and anchoring of the steel under load.

(3) The grout and vent pipes required for injecting the anchor grout. Secondary grouting of the stressing length is often done for corrosion protection.

Rock anchors have a wide variety of applications and may be installed in most rock types. Figure 3-91 shows the basic components of a rock anchor. Rock anchor design must consider the following failure modes:

(1) **Failure of steel tendon**. Design stress within the steel is usually limited to 50 to 60 percent of the ultimate stress (50 percent for permanent installations).

(2) **Failure of grout-steel bond**. The bond capacity depends on the number and length of the tendons, or steel bars (plain or deformed) and other factors. For guidance see *Rock Anchors, State of the Art*, by Littlejohn and Bruce in the reference section.

(3) **Failure of grout-rock bond**. The bonding capacity between the rock and the grout may be determined using

$$P_u = \pi d_s L_o \delta \mathrm{skin} \qquad (3\text{-}132)$$

in which

P_u equals load capacity of anchor,
d_s equals diameter of drilled shaft,
L_o equals length of grout-anchor bond, and
δskin equals grout-rock bond strength.

Typical grout rock stresses for various rock types are presented in table 3-23.

Table 3-23.—Typical values of bond stress for selected rock types.

Rock Type (Sound, Non-Decayed)	Ultimate Bond Stresses Between Rock and Anchor Plus (δ_{skin}), psi
Granite & Basalt	250 - 450
Limestone (competent)	300 - 400
Dolomitic Limestone	200 - 300
Soft Limestone	150 - 220
Slates and Hard Shales	120 - 200
Soft Shales	30 - 120
Sandstone	120 - 150
Chalk (variable properties)	30 - 150
Marl (stiff, friable, fissured)	25 - 36

Note: It is not generally recommended that design bond stresses exceed 200 psi even in the most competent rocks.

(4) **Failure of rock mass**. The criterion for failure in rock mass is based on the weight of rock contained within a cone emanating from the bonded zone. Figure 3-93 shows design criteria. Actual anchor failure in this mode would be controlled by discontinuity patterns and weathering of the rock.

The capability of epoxy grout to bond with rock is so great that the rock strength is the controlling strength factor. Figure 3-93 shows a graph of compressive strength in ksi versus ultimate anchor capacity in kips per foot of grouted length. The figure should be used for preliminary estimates of the required grouted length of anchor. A suitable factor of safety should be applied to the grouted lengths determined from figure 3-93; that is, in the range of 3 to 10, depending on the rock quality.

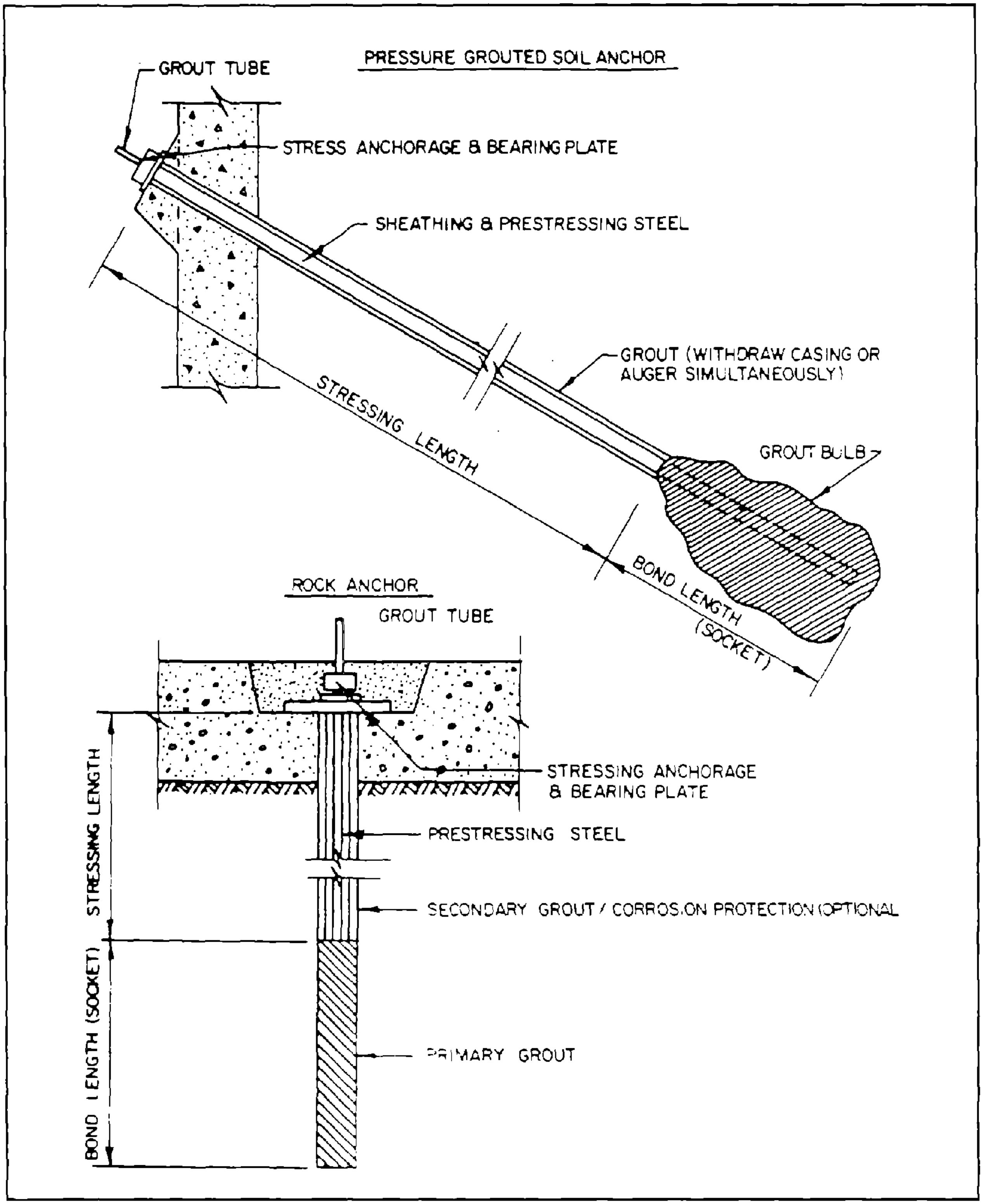


Figure 3-91.—Basic components of ground anchors.

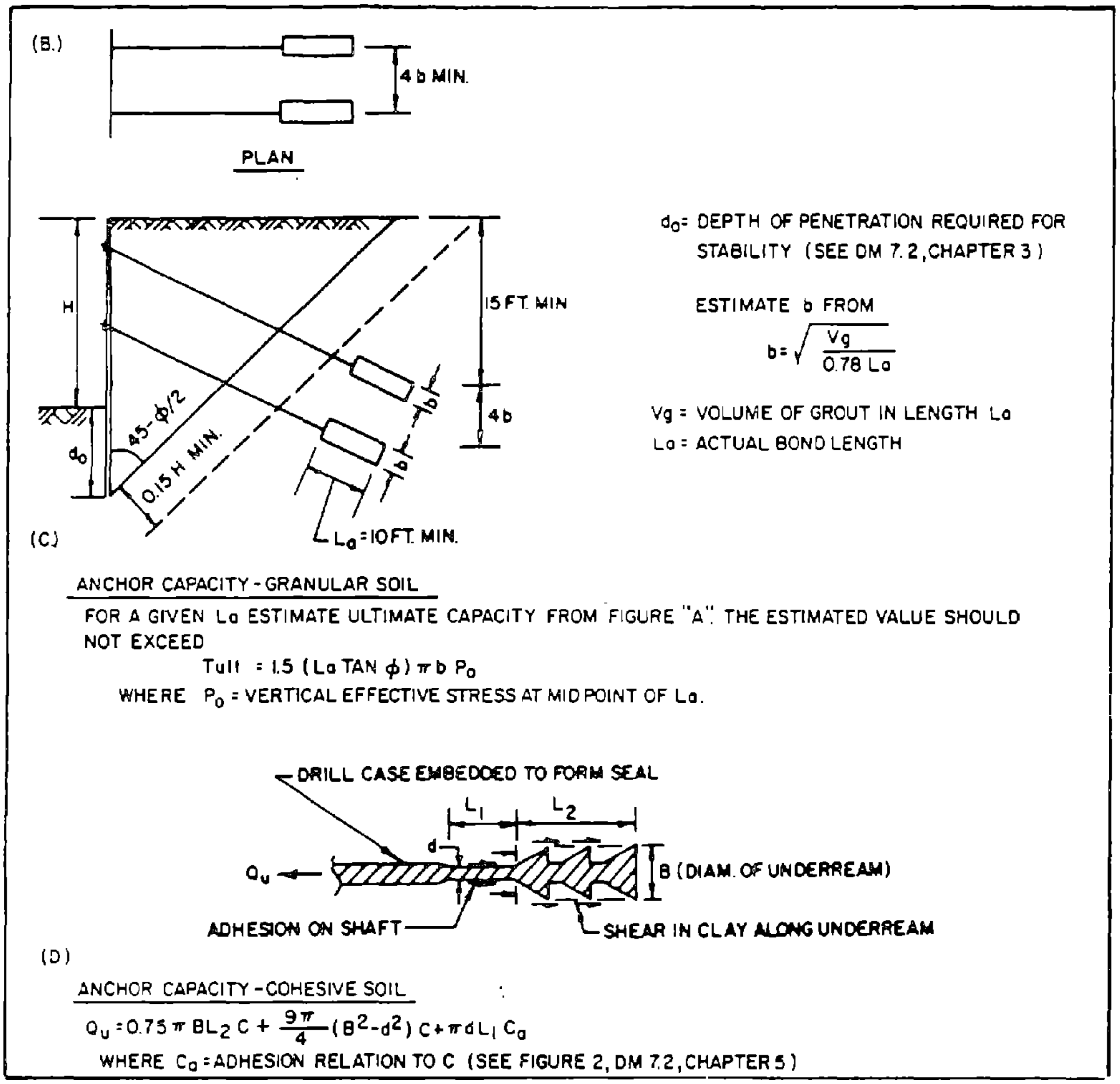


Figure 3-92.—Estimate of anchor capacity.

When using cement grout, a rule of thumb for determining grouted length, l_g, in rock with compressive strengths in excess of 3000 psi is

$$l_g \geq 60d \tag{3-133}$$

in which *d* is the diameter of the tie rod. For anchors designed in accordance with equation 3-133, the tie rod will fail before it pulls out of the grout plug. Regardless of the design method or grout type, the minimum grouted length should be 4 feet.

Expandable shell rock anchors in hard rock should be designed on 40 percent of the compressive strength over the surface area of the shell. In soft rock, the design should be based on 18 percent of the compressive strength.

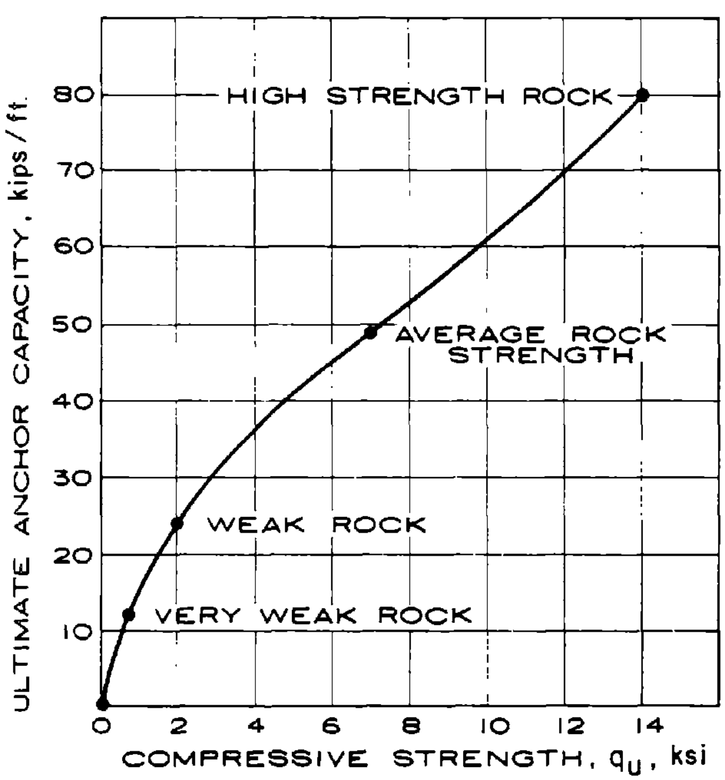


Figure 3-93.—Preliminary resin anchor design.

Suggested procedures and specifications for rock anchor installation and testing are presented in the following manual: *Post Tensioning Manual*, 5th Edition, 1990, Post-Tensioning Institute, 1717 West Northern Ave, Phoenix, AZ 85021, telephone (602) 870-7540.

3N.6 Soil Anchors

Soil anchors can be installed in nearly all types of soil. Types of anchors and applicable soils are presented in table 3-24. Anchor capacity depends on various factors, such as soil type and grout penetration. Estimate of anchor capacity should consider past experience, pullout testing of anchors, soils data, and consequences of failure. In some cases, field testing of all anchors is necessary. Anchors in coarse sands and gravels have had working loads up to 80 tons (factor of safety, F_s = 1.5) where the fixed anchor has had about 40 feet of overburden on it. Anchors in medium sands, with the fixed anchor below 20 to 30 feet of overburden, have been installed with working loads up to 40 tons (F_s = 2). Anchors with working loads up to 60 tons (F_s = 3) have been installed in stiff clays.

Anchorages in granular soils are formed by the injection of grout under high pressure so that a grout bulb forms along the bond length of the anchor. Figure 3-94 presents a graph of anchor capacity versus bond length for granular soil types of various densities. The figure may also be used for pretest estimate of bond length (free or stressing length of anchor is normally a minimum of 20 to 25 feet).

Guidance is given in figure 3-94 for pretest estimating and pullout capacity of anchors in cohesive soils.

Because of the large number of variables affecting anchor performance, anchors are normally proof-loaded to at least 115 to 125 percent of the design load with selected anchors tested to higher loads and for long-term creep characteristics. Permanent anchors should be tested to 150 percent of the design load.

Anchors in soil should be designed using a minimum factor of safety of 2.0; a higher factor of safety is used for permanent or critical structures. All production anchors should be proof-loaded to 115 to 160 percent of the design load. Additional testing to higher capacities and to determine creep characteristics may be justified for permanent installations, or where the design conditions warrant.

3O Materials

3O.1 General

Timber, concrete, and steel are commonly utilized as structural components in retaining walls on low volume roads. Geosynthetics and epoxy are also rapidly gaining acceptance as construction materials. The following subsections will include discussions of each of the construction materials.

3O.1.2 Timber

Timber is used in the construction of crib walls, as lagging and piling in lagged cantilever pile walls, and as anchors for tieback systems.

The advantages of timber as a construction material relate to its ease of handling and its ability to field-modify if required. Aesthetically, timber also tends to blend with its surroundings. Some of the disadvantages of timber are its lack of permanence. Timber rots, if attacked by insects and other pests, is combustible. Timber is also subject to vandalism by chain saws, axes, and so forth.

Table 3-24.—Types of soil anchors.

Method	Diameter (inches)		Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
	Shaft Type	Bell Type				
1. LOW PRESSURE						
Straight Shaft Friction (Solid stem auger)	12 - 24"	NA	A	NA	Very stiff to hard clays Dense cohesive sands	Friction
Straight Shaft Friction (Hollow stem auger)	6 - 18"	NA	NA	30 - 150	Very stiff to hard clay Dense cohesive sands Loose to dense sands	Friction
Underreamed Single Bell at Bottom	12 - 18"	30 - 42"	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
Underreamed Multi-bell	4 - 8"	8 - 24"	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft Rock	Friction and bearing

Table 3-24.—Types of soil anchors (cont'd.).

Method	Diameter (inches)		Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
	Shaft Type	Bell Type				
2. HIGH PRESSURE - SMALL DIAMETER						
Non-regroutable (2)	3 - 8"	NA	NA	150	Hard clays Sands Sand-gravel formations Glacial till or hardpan	Friction or friction and bearing in permeable soils
Regroutable (3)	3 - 8"	NA	NA	200 - 500	Same soils as for non-regroutable anchors plus: a) Stiff to very stiff clay b) Varied and difficult soils	Friction and bearing

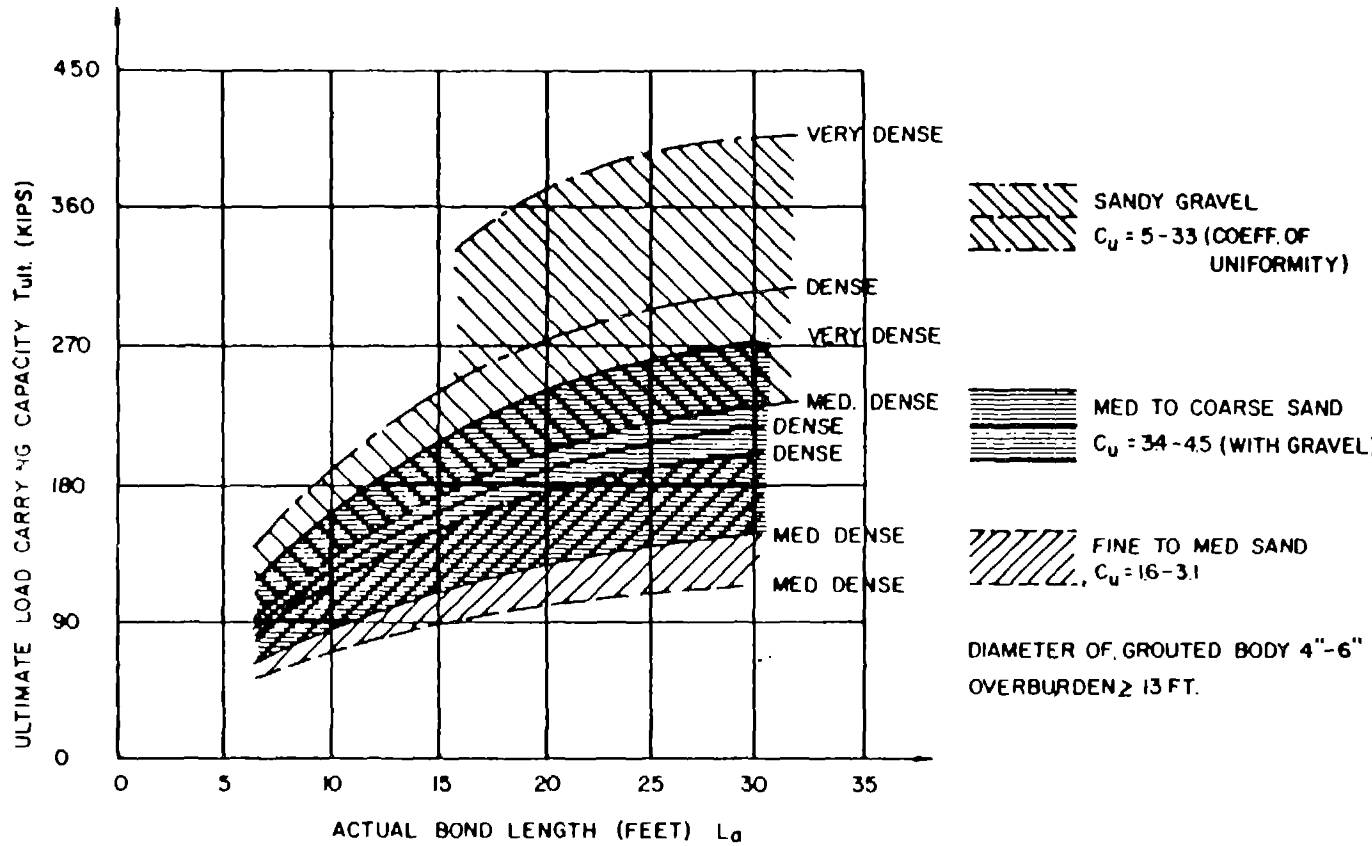
(1) Grout pressures are typical

(2) Friction from compacted zone having locked in stress. Mass penetration of grout in highly pervious sand/gravel forms "bulb" anchor."

(3) Local penetration of grout will form bulbs which act in bearing or increase effective diameter.

A - Applicable

NA - Not applicable



(A) EMPIRICAL ULTIMATE CAPACITY OF ANCHORS IN GRANULAR SOILS SHOWING INFLUENCE OF SOIL TYPE (COEFFICIENT OF UNIFORMITY), DENSITY AND BOND-TO-GROUND LENGTH.

Figure 3-94.—Estimate of anchor capacity.

Except for walls intended to have a very short life (less than 2 years), all timber exposed to soil or weather should be pressure-treated. Treatment types fall into three general categories: creosote, pentachlorophenol, and water-borne salts. All are effective and the designer should become familiar with the local availability of each process. Pressure treatment should be in accordance with the American Wood Preservers Association (AWPA) C-14 standard, except for marine piling.

To increase its useful life, precut all timber to length and have any other fabrication done prior to pressure treatment, unless required to be done in the field. Damaged areas, field cuts, and holes in all pressure-treated timber should be treated in accordance with AWPA standard M-4.

Marine (saltwater) piling should be costal Douglas fir pressure-treated in conformance with AWPB standard C-18 and specified in accordance with ASTM D-25.

Piling for nonmarine use should be Douglas fir specified in accordance with ASTM D-25. Pressure treatment should be required in accordance with AWPB standard C-3.

Design stresses for piling and lumber depend on the size and shape of the piece as well as on the species. Shape descriptions are shown on table 3-25. In addition, the commonly used grades for various uses are presented. As there are several grades of timber for each use, care must be taken when selecting structural members to assure that the proper grade is chosen. Higher grades than those shown in the table are available; they are, however, expensive and frequently unavailable.

Table 3-25.—Timber shape descriptions.

Use Grade	**Common**	**Shape Description**
Piling	Class A	Round in section
Decking	Commercial	2″ to 4″ thick, 4″ to 12″ wide
Posts and timbers	no. 1	5x5 and larger—width not more than 2″ greater than thickness
Beams and stringers	no. 1	5″ and thicker—width more than 2″ greater than thickness
Structural joists and planks	no. 2	2″ to 4″ thick—5″ and wider

For reliable design stress values for lumber, consult the current edition of the *Uniform Building Code* or the *WWPA Grading Rules for Western Lumber*. For piles, consult the current edition of the *AASHTO Standard Specifications for Highway Bridges*. Generally, WWPA-tabulated values are for sawn lumber and dry stresses (moisture content of wood no greater than 19 percent). If the moisture content of the wood exceeds 19 percent, the wood is considered wet and the allowable stress must be reduced as described in the grading rules. This is the case for most retaining structures. Different reductions in allowable stress for wet conditions are given for each type of stress (compression, bending, shear, tension, and so forth).

The allowable design stresses for wood are also affected by several other factors such as the duration of the load. Generally, a reduction of the maximum allowable design load is recommended for permanent or dead loads. However, for transient load conditions such as seismic and other short-term loadings, the allowable design stress, reduced for wet conditions, may be increased. Refer to the current edition of AITC's *Timber Construction Manual* for a discussion of each factor and application examples.

The AITC manual should also be used as a reference for other design considerations, such as the combined bending and axial stresses being subject to inspection with the "interaction formula"

$$\frac{f_b}{F_b} + \frac{f_a}{F_a} \leq 1 \qquad (3\text{-}134)$$

in which f_b and F_b are the actual bending stress and allowable bending stress respectively, and f_a and F_a are respectively the actual and allowable axial stress.

30.3 Structural Steel

Structural steel is used in the construction of retaining walls as reinforcement for concrete, soldier and anchor piles, sheet piles, anchor bars and tendons, and bins and cribs.

The major advantages of steel as a construction material are its high unit strength, toughness, durability during construction, ease of field modification, and generally ready availability. Major disadvantages are its corrodibility, and often the need for special handling equipment.

The design stress, F_b, of tie anchors or bars is generally taken to be 0.6 F_y, which is the yield stress of the steel.

Reinforcing steel in reinforced concrete should be placed and tied as designed. AASHTO M31, M42, and M53 give specifications for acceptable deformed bars and grade 60 should be normally specified. When available, grade 40 will suit most low-stress applications, because it is readily available and might be most economical in walls. ASTM A185 gives specifications for welded wire fabric. Reinforcing should be designed and called out to be 3 inches clear of the ground or other unformed surfaces and 1-1/2 or 2 inches clear of formed surfaces.

3P Corrosion

3P.1 Introduction

The corrosion and deterioration of structural components in retaining walls is an inevitable process. The rate of this process is determined by the environmental and structural characteristics of the materials in the project. By understanding the corrosivity of the site and the corrosion and deterioration susceptibility of the construction materials, the engineer can select the most cost-effective design to meet the objectives of the project.

Deterioration occurs in all of the materials in a retaining wall structure, whether they are metal, geosynthetics, or treated wood. This section will discuss evaluation of the site conditions in relation to corrosion and deterioration of the structural components, corrosion protection for metallic elements, durability of geosynthetics used in retaining walls, and touch briefly on treated timbers.

3P.1.1 Corrosion

Corrosion is the deterioration of metal by chemical or electrochemical reaction with its surroundings. Corrosion factors include the site, the atmosphere, imported materials, incompatible materials, or maintenance activities. Testing of the materials in the environment determines the degree and type of environmental corrosion. The major parameters in determining soil/water corrosivity are resistivity; moisture content (soil); soluble salts such as chlorides, sulfates, and sulfides; pH; redox potential; organic material; and soluble iron content.

Once the degree of environmental corrosion is determined, the engineer has several options in selecting structural components. Materials differ in their susceptibility to environmental conditions. Some techniques in fighting onsite corrosion are: changing backfill material, using different structural components, designing "sacrificial" thickness into the structural components, adding coatings to components, using cathodic protection, and controlling drainage. These options can be used separately or in combination.

Evaluation of site corrosion and the susceptibility of the structural components to corrosion are part of the design process. Selection of method to address the corrosion problem at the site should be part of the risk analysis process for the project.

3P.1.2 Geosynthetics

The use of geosynthetics is becoming more common in retaining wall structures. While geosynthetics do not corrode, they do deteriorate. It is important to evaluate the characteristics of the project environment as they affect the durability of the geosynthetics. As with other materials, individual geosynthetics differ in their susceptibility to environmental conditions. Durability and survivability during the construction process are also key factors in the use of geosynthetics.

3P.1.3 Treated Timber

It is important that during the construction process the integrity of the treated timber not be impugned.

3P.2 Corrosion Parameters

Several parameters are used to rate a soil's corrosivity. Direct relationships between any one parameter and rates of corrosion are not yet known. However, soil resistivity has emerged as the most accurate indicator of a soil's corrosivity potential. The other parameters are moisture content, soluble salts (chlorides, sulfates, and sulfides), pH, redox potential, organic material, and soluble iron content.

Corrosivity is usually rated in five categories: noncorrosive, mildly corrosive, moderately corrosive, corrosive, and very corrosive. Table 3-26 shows the parameters and the range of values in relationship to the degree of corrosivity of the soil.

3P.2.1 Soil Resistivity

Resistivity is an indirect measurement of the soluble salt content of a soil. Higher soluble salt contents result in higher conductivity, or lower resistivity. As the resistivity decreases, the corrosion potential of a soil increases. Resistivity appears to be the best indicator of a soil's corrosivity potential. Resistivity is usually expressed as ohm/cm. There are several methods to measure soil resistivity. For scanning purposes, FHWA recommends the laboratory procedure on saturated paste in ASTM G57-78. In the FHWA 1992 "Standard Specifications," they require that resistivity be measured by AASHTO T288 (any method).

Table 3-27 shows the ranges of resistivity values for different soils and rocks.

3P.2.2 Moisture Content

There is an important relationship and interaction between compaction, water content, and resistivity. This relationship is usually under-emphasized. As moisture content increases, resistivity decreases. Data suggests that maximum corrosion rates occur at saturations of 60 to 85 percent. For granular materials, this is similar to the moisture content range in the field for required compaction. Other electrochemical parameters being equal, the placement compaction requirements for soil- reinforced structures result in the highest corrosion potential conditions.

The moisture content of a soil is determined by the soil structure, its permeability, and its porosity. When the moisture content is greater than 25 to 40 percent, the rate of general corrosion is enhanced. Below this level, corrosion by pitting is more likely to occur.

In the TRRL Supplementary Report 316, the importance of alternate wetting and drying on corrosion of buried metals is discussed. It also suggests that corrosion behavior may be affected by the time of year of construction.

Table 3-26.—Corrosivity table.

INDICATOR	Non-Corrosive	Mildly Corrosive	Moderately Corrosive	Corrosive	Very Corrosive
Resistivity (ohm/cm)	>10,000	5,000–10,000	2,000–5,000	700–2,000	<700
pH	4.5–9.5 -	- - - - - -	- - >		
Redox potential (mV)	>400	200–400	100–200		<100
Chloride (ppm)			60–180		
Sulfates (ppm)			90–280		
Organic content			$\leq$0.01%		
Moisture content					*60– 85%

*Maximum corrosion rates seem to occur within this saturation range.

Table 3-27.—Resistivities of different soils.

Soil	Resistivity range (ohm/cm)
Surface soils, loam, etc.	100–5,000
Clay	200–10,000
Sand and gravel	5,000–100,000
Surface limestone	10,000–1,000,000
Limestones	500–400,000
Shales	500–10,000
Sandstone	2,000–200,000
Granites, basalts, etc.	100,000
Decomposed gneisses	5,000–50,000
Slates, etc.	1,000–10,000

3P.2.3 Soluble Salts

Dissolved salts such as chlorides and sulfates decrease resistivity, promoting the flow of corrosion currents and inhibiting the development of protective layers. Only carbonate forms a scale on most metals and reduces corrosion. Chlorides and sulfates are considered the major promoters of corrosion. Analysis of field tests in France indicates that chlorides are more corrosive than sulfates at equal concentrations at 50 percent saturation levels.

ASTM D-4327-88 is a recently adopted standard to measure anions by ion chromatography. It is the most accurate and reproducible of the methods. It can be used to determine both chloride and sulfate simultaneously. FHWA construction specifications for select granular backfill designate the AASHTO T-290 test for determining sulfate content, and AASHTO T-291 for chloride.

Sulfides in soils may cause severe deterioration of steel and concrete. Freshly exposed sulfidic materials do not give indication of acid sulfate conditions when tested in the laboratory. However, the occurrence of sulfidic materials is usually limited to geologic formations derived from marine sediments or associated with coal strata. Samples having Munsell color chromas of ≤ 1 (black) in areas known to have sulfidic soils should be screened for sulfides. Only screening for sulfides is recommended because no simple quantitative method exists.

3P.2.4 PH

PH values are not as important a parameter as soil resistivity. Also, they may be misleading as the total acidity or alkalinity of the soil may be more important. Some agencies, however, have found pH to be a good indicator of local corrosion.

The corrosion rates of metallic reinforcement materials generally increase as the pH value decreases from neutral. Extremely acidic soils (< 4) or very strongly alkaline soils (> 10) are generally associated with significant corrosion rates. Zinc is attacked in strongly acidic and alkaline soils so that galvanized coatings will have a much reduced life expectancy in those regions. However, only very low values will have a significant effect on high alloy steels.

The most widely used method for measuring pH is the soil survey laboratory procedure 8Cla. There are other methods for testing if a more detailed evaluation is needed.

3P.2.5 Redox Potential

Oxidation reduction potential, or redox potential, is a primary indicator of anaerobic bacterial corrosion. This is generally associated with reinforced structures containing organic matter, such as dredged fills or coastal marine sediments.

At present, there are no standard methods for measuring redox potential.

3P.2.6 Organic Material

Organic material in soils can be reduced to organic acids which produce pitting corrosion when in contact with metals. Inclusion of backfill with organics may lead to the development of anaerobic pockets, which could be contaminated with sulphate-reducing bacteria (SRB), leading to severe pitting corrosion.

The AASHTO test method T-276-86 is usually used to determine organic content.

3P.2.7 Soluble Iron Content

Soils containing more than 125 mg Fe/gram soil have been found to be aggressive, while soils with less than 50 mg Fe/gram soil are not aggressive. Very aggressive biogenic iron sulphides may develop in high soluble iron soils.

3P.3 Design Considerations

Once the type and degree of corrosion of the soils has been determined, then the structural components for the project may be selected and designed. Risk analysis plays a major role in this evaluation. The criticality of the structure, the planned design life, the cost, and the corrosion of the site are all part of the analysis.

There are several ways to deal with corrosion problems in a reinforced earth structure. These include changing the materials in the project—either backfill material or structural components, using "sacrificial" materials, coatings on structural components, cathodic protection, and control of drainage.

3P.3.1 Change in Materials

Backfill. In corrosive environments, one method of reducing corrosion is to replace native backfill with imported, select backfill. The FHWA's *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects* (1992) has a specification for select granular backfill (see figure 3-65).

Structural components. Susceptibility to corrosion varies among different structural components. Due to costs of materials, this option may be limited. Table 3-28 shows corrosion resistance of various materials.

Figure 3-95 shows the underground corrosion rate for steel for various resistivity and pH conditions.

Tendons and tie rods are commonly protected from corrosion by greasing and wrapping, or by full-length grouting. Additional protection may be provided by bedding the tendons or rods in an envelope of free-draining material. A commonly overlooked detail of corrosion protection is hardware, which must be designed to resist corrosion.

In order to provide positive corrosion protection, copious construction inspection must be done to ensure that the structure is constructed as designed and that none of the protective coatings or shields are broken or ruptured.

3P.3.2 Sacrificial Materials

The principle of "sacrificial" materials states that materials corrode at generally predictable rates. By knowing the design life of the project and the predicted corrosion rate of specific structural components, the engineer can design a thickness for the structural components sufficient to satisfy structural requirements until the end of the design life. The thickness of the material, which is greater than the structural requirements, is thus "sacrificed" to the corrosion process.

Table 3-28.—Corrosion resistance of reinforcement materials.

Soil property	Mild steel	Galvanized steel	Aluminum coated steel	Lead coated steel	Low alloy steel	High alloy steel	Aluminum and alloys	Glass reinforced plastic (Grp)	Copper	Copper steels
Resistivity < 700	3	2	2	1	2P	1	3		1	2
(ohm/cm) 700–4000	2	2	1	1	1P		2		1	1
> 4000	0	0								
Redox > 400										
potential 200–400	1	1	2	1	1		2	?	1	1
(mV NHE) < 200	3	3	3	2	3P	2?	3	?	2	2
Moisture > 80%	1	1	1	1	1	1	1	1	1	1
content (%) 10–80 %	2	2	2	2	1	1	2	1	1	2
< 10%										
Dissolved salts	2	2	2	1	1	1	3	1	2	2
Chlorite	2	2	3	1	3	2	3	1	2	2
pH acidic < 6	2	3	3	1	2	1	3		2	3
neutral 6–8										
alkaline > 8	1	2	3	2			3		1	
Organic acids	2	2	2	3	1	1	3	?	3	3

<--- COATING INTACT --->

□ generally unaffected in this environment

0 - Generally unaffected3 - Markedly affected
1 - Affected only slightlyP - Pitting attack pronounced
2 - Affected? - Behavior unknownfrom NCHRPR 290, p. 77 (table 21)

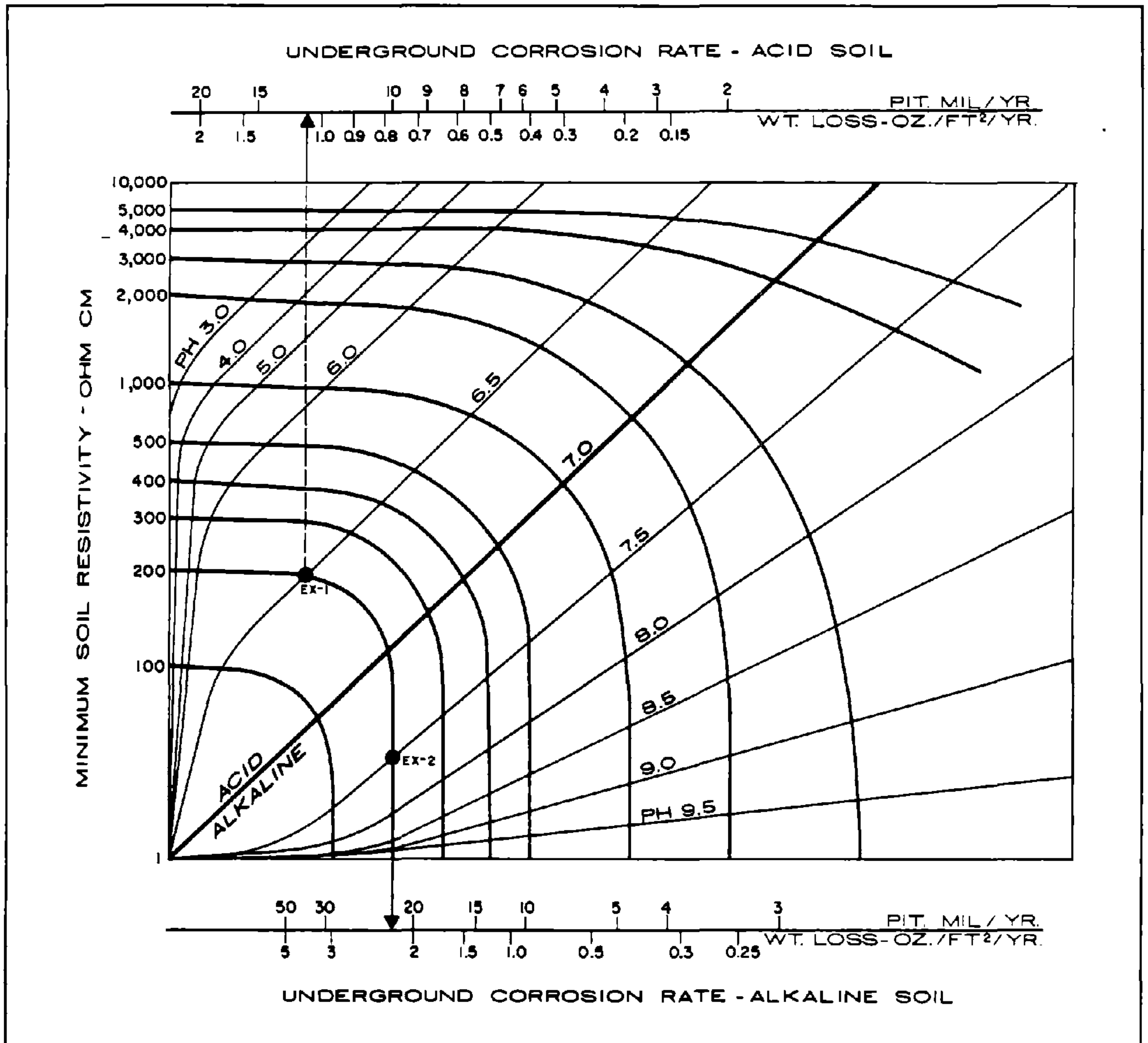


Figure 3-95.—California Transportation Laboratories—corrosion rate.

3P.3.3 Coatings

Coatings work somewhat like sacrificial thickness. The difference is that coatings are different materials layered onto the structural component. The rate of corrosion of the coating is predictable, and the thickness is designed for the corrosion potential of the site and the design life.

Some materials used as coatings include zinc (galvanizing), epoxy, asphalt tape, asphalt mastic, and polyvinyl (pvc). The coating type selection is generally based on type and degree of corrosion and cost.

3P.3.4 Cathodic Protection

Corrosion of steel reinforcing bars in concrete occurs by an electrical process in a moist environment. A voltage difference develops between bars or between areas of bars during corrosion, creating an electron flow.

Corrosion occurs at the anode, where the electrons from the iron are given up.

Cathodic protection results in this electrical current being reversed, slowing or stopping the corrosion. This is accomplished by use of an electrical DC rectifier supplying electrical current from local electrical power lines to a separate anode embedded in the concrete. This anode is usually a wire mesh embedded just under the concrete surface.

When the impressed current enters the separate anode, the voltage on the rebars is reversed, turning the network into a cathode. Since natural corrosion occurs only at the anode, the rebars are protected. With impressed current cathodic protection, the corrosive action which would occur at the separate anode does not, because electrons are supplied by the external DC rectifier. Thus, the artificial anode is also protected from corrosion.

3P.3.5 Control of Drainage

Control of drainage in a reinforced earth structure is another means of reducing the corrosion rate. Reduction of moisture content tends to reduce corrosion potential. Also, elimination of the seasonal fluctuations in ground water is generally accepted to be desirable.

3P.4 Construction Considerations

The construction process can have a large impact on the corrosion resistance of the structural components of a reinforced earth structure. Damage to materials may reduce or, in some cases, eliminate corrosion protection in the structural components. Areas of concern are: transportation, on-site storage, environmental protection prior to use, and construction damage.

Materials should be transported and stored so that coatings are not scratched or damaged. Any damage should be repaired according to established construction specifications prior to installation. Components need to be protected from environmental damage due to water, UV (in the case of geosynthetics), and petroleum products (geosynthetics and asphaltic coatings).

To keep from damaging the reinforcing members, use caution during the compaction process. Backfill material should be placed, with no rocks being dropped on the structural components. Construction equipment should not be operated directly on any structural material.

It is also important that compaction be completed in a uniform manner. This results in consistent resistivity in the backfill and generally corrosion reduction.

3P.5 Monitoring

The first and most important part of any monitoring program is to determine why one wants to monitor a structure. After determining the reason for the monitoring, one goes on to deciding what to monitor and how. Additionally, a monitoring program can be simple or complex, depending on the needs of the program. If there are no questions about a structure, then monitoring is unnecessary.

In the FHWA publication *Reinforced Soil Structures (volume I) Design and Construction Guidelines* (FHWA-RD-89-043), p. 240, table 14, shows the steps in designing a monitoring program:

(1) Define the purpose of the monitoring program.

(2) Define the project conditions.

(3) Predict the mechanisms that control behavior.

(4) Select the parameters to be monitored.

(5) Predict magnitudes of change.

(6) Devise remedial action, should measurements exceed warning levels.

(7) Assign monitoring tasks for design, construction, and operation phases.

(8) Select instruments, based on reliability and simplicity.

(9) Select instrument locations.

(10) Plan recording of factors that may influence measured data.

(11) Establish procedures for ensuring reading correctness.

(12) Prepare budget.

(13) Write instrument procurement specifications.

(14) Plan installation.

(15) Plan regular calibration and maintenance.

(16) Plan data collection, processing, presentation, interpretation, reporting, and implementation.

(17) Write contractual arrangements for field instrumentation services.

(18) Update budget.

Pages 241 through 242 of the FHWA-RD-89-043 contains a list of important parameters for reinforced structures. The list includes the following parameters:

(1) Horizontal movements of the face.

(2) Vertical movements of the surface of the overall structure.

(3) Local movements or deterioration of the facing elements.

(4) Drainage behavior of the backfill.

(5) Performance of any structure supported by the reinforced soil, such as approach slabs for bridge abutments or footings.

(6) Horizontal movements within the overall structure.

(7) Vertical movements within the overall structure.

(8) Lateral earth pressure at the back of facing elements.

(9) Vertical stress distribution at the base of the structure.

(10) Stresses in the reinforcement, with special attention to the magnitude and location of the maximum stress.

(11) Stress distribution in the reinforcement due to surcharge loads.

(12) Relationship between settlement and stress-strain distribution.

(13) Stress relaxation in the reinforcement with time.

(14) Total horizontal stress within the backfill and at the back of the reinforced wall section.

(15) Aging condition of reinforcement such as corrosion losses.

(16) Pore pressure response below structure.

(17) Temperature (often a cause of real changes in other parameters, and it may also affect instrument readings).

(18) Rainfall (often a cause of real changes in other parameters).

(19) Barometric pressure may affect readings of earth pressure and pore pressure measuring instruments.

3P.6 Geosynthetics

The durability of geosynthetics in reinforced earth applications is important and variable. Table 3-29 shows resistance of various polymers to soil environments.

3P.7 Geosynthetic Construction Survivability

Stresses applied to geosynthetics during construction often exceed those encountered by the materials during actual use. For that reason, selection of geosynthetics is generally governed by expected construction impact. The idea is that the geosynthetic must survive construction in order for it to perform its intended function.

Table 3-30 shows the relationship between the elements of construction to the severity of loading on the geotextile. Use these tables with prudent judgment.

Table 3-29.—Anticipated resistance of polymers to specific soil environments.

Soil environment	PETP	PA	PE	PP	PVC
Acid sulphate soils	?	o	NE	?	?
Organic soils	NE	?	NE	NE	o
Salt affected soils	?	NE	NE	NE	NE
Ferroginous	NE	?	NE	o	o
Calcareous	o	?	NE	NE	?
Modified soils	o	?	NE	NE	?

PETP = Polyethylene Terephthalate
PA = Polyamides
PE = Polyethylene
PP = Polypropylene
PVC = Polyvinylchloride

NE = No effect
? = Questionable use
o = Not recommended

From: *Durability/Corrosion of Soil Reinforced Structures*, FHWA, 1990 (p. 111).

Table 3-30.—Relationship of construction elements to severity of loading imposed on geotextile in roadway construction severity category.

Variable	Low	Moderate	High to Very High
Equipment	Light dozer (8 psi) wheeled equip. (8–40 psi)	Medium weight dozer; light (>40 psi)	Heavy weight dozer; loaded dump truck
Aggregate	Rounded sandy	Coarse angular	Cobbles, blasted rock
Lift thickness	18"	12"	6"

Table 3-31 addresses the necessary strength required for geotextiles to survive the most severe conditions anticipated during construction.

Table 3-31.—Geotextile strength required for survivability during construction physical—property requirements (< 50% geotextile elongation / > 50% geotextile elongation (2,3))

Survivability level ASTM D 4632 (lb)	Grab strength ASTM D 4833 (lb)	Puncture resistance ASTM D 4533 (lb)	Trapezoid tear strength
High	270/180	100/75	100/75
Medium	180/115	70/40	70/40

Additional requirements	Test methods
(1) Apparent Opening Size	ASTM D 4751
• <50% soil passing a no. 200 U.S. sieve, AOS <0.6 mm.	
• >50% soil passing a no. 200 U.S. sieve, AOS <0.3 mm.	
(2) Permeability	ASTM D 4491
• k of the fabric>k of the soil (Permittivity times the nominal geotextile thickness.)	
(3) Ultraviolet degradation	ASTM D 4355
• At 150 hours exposure, 70% strength retained for all cases.	
(4) Geotextile acceptance	ASTM D 4759

[1] Values shown are minimum roll average value—strength values are in the weaker principle direction.
[2] Elongation as determined by ASTM D 4632.
[3] The values of geotextile elongation do not imply the allowable consolidation properties of the subgrade soil. These must be determined by a separate investigation.
* From FHWA publication FHWA-HI-90-00, "*Geotextile Design & Construction Guidelines*", tables 9-1 and 9-2, p. 225.

Note: These survivability requirements were not based on systematic research. These requirements are based on performance of geotextiles as separators on temporary roads and similar applications. Current research is suggesting that elongation may also be a factor. In the absence of other data, these values may be used. Judgment and experience need to be applied to geotextile selection and, for major projects, field tests should be used to verify values.

Chapter 4
Specific Wall Designs

4A Introduction

Six example problems have been developed to demonstrate important design considerations relating to retaining walls. The examples selected have been distributed among the four wall classifications presented in table 2-1. Standard designs (proprietary) and various standard design details are grouped in appendix E, and typical specifications are presented in appendix F.

The authors emphasize that the earth pressures developed in the following sample problems are based on classical earth pressure theory or on the semiempirical methods presented in chapter 3. The user of this guide must realize that the active and passive states of stress discussed in chapter 3 are conditions of failure within the soil mass. The active state of stress corresponds to the minimum or lowest level of stress that the soil can exert on the uphill side of the structure. The passive state of stress corresponds to the maximum or highest level of stress that the soil can develop to resist downslope movement.

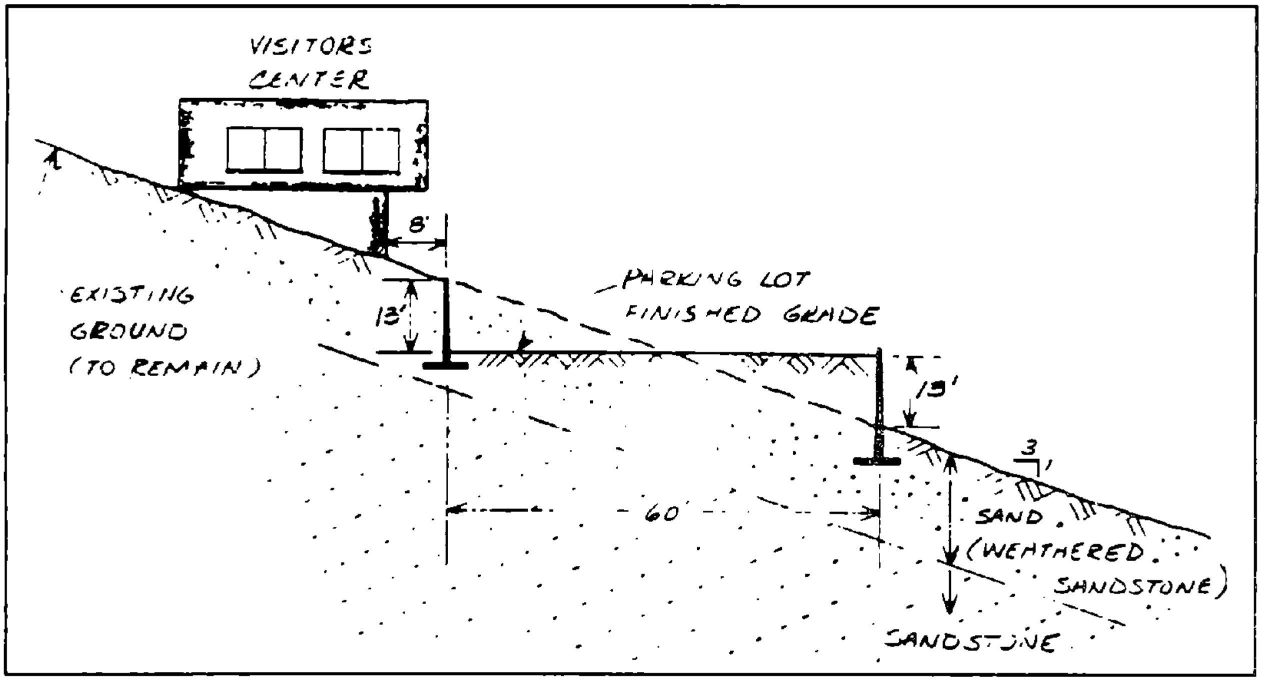
Further, the authors emphasize that construction practices and stratified subsurface conditions can vastly alter the magnitude of stress mobilized by the wall. These stress changes are generally nonconservative and may require analysis by one of the trial-and-error methods (see section 3.D.4).

The remainder of this chapter consists of: (1) a problem statement including soil properties and a cross-section through the proposed wall, (2) a numerical analysis based on design methods presented in chapter 3 of this guide, and (3) a discussion of important aspects and methods of construction.

4B Concrete Cantilever Wall

4B.1 Problem Statement

A visitor's center and viewpoint are proposed to be constructed on a hillside. The soil on the 3H:1V slope is weathered sandstone to a depth of approximately 15 feet below the existing ground surface. The visitor's center will be founded above a parking lot approximately 60 feet wide. To form a suitable grade for the parking lot and to retain natural landforms above and below the lot, a pair of reinforced concrete retaining walls will be utilized, as shown in the figure below. The total



length of the wall will be 178 feet. The walls will be constructed prior to the construction of the visitor center.

Because there is a cafeteria in the visitor center, it is anticipated that heavy trucks will occasionally use the parking area. Hence, the structural design of the concrete walls must allow for such.

The visitor center will be founded 8 feet from the upper wall. The bearing pressures allowed for the structure are 4,000 psf. The structural design of the building will fully utilize this allowed bearing pressure on the 18-inch-wide strip footing at 8 feet horizontal distance from the wall.

Standard penetration test values average 20 blows per foot. From the approximate correlation given in table 3-1, it is estimated that the angle of internal friction is 33°. Unit weight of the material is estimated at 120 pcf in its natural state. The walls will be backfilled with weathered sandstone. The water table will fluctuate between the ground surface and 15 feet below on a seasonal basis.

Each of the walls will have a 13-foot retained face and will be founded 2 feet below the finished ground line. The same wall section is desired for each wall. Therefore, the most conservative loadings should be used in design.

4B.2 Design Calculations

Determine most conservative loading criteria.

From table 3-9, the total lateral load due to a legal .16-1/2 K/axle highway load is .97 kips with the truck parked 2 feet from the top of the wall.

The influence of the strip footing load will be computed as if it were a line load by reference to figures 3-23c and d.

$$M = \frac{X}{H} = \frac{8}{15} = 0.53$$

From figure 3-23c, $P_h/q_L = 0.50$, or $P_h = .50q_L$

where $q_L = 4{,}000\left(\frac{18}{12}\right) = 6{,}000$ plf.

so $P_h = 0.50(6 \text{ klf}) = 3.00$ klf.

The resultant is located at 0.55 m above the base of the wall, i.e. 8.25 feet.

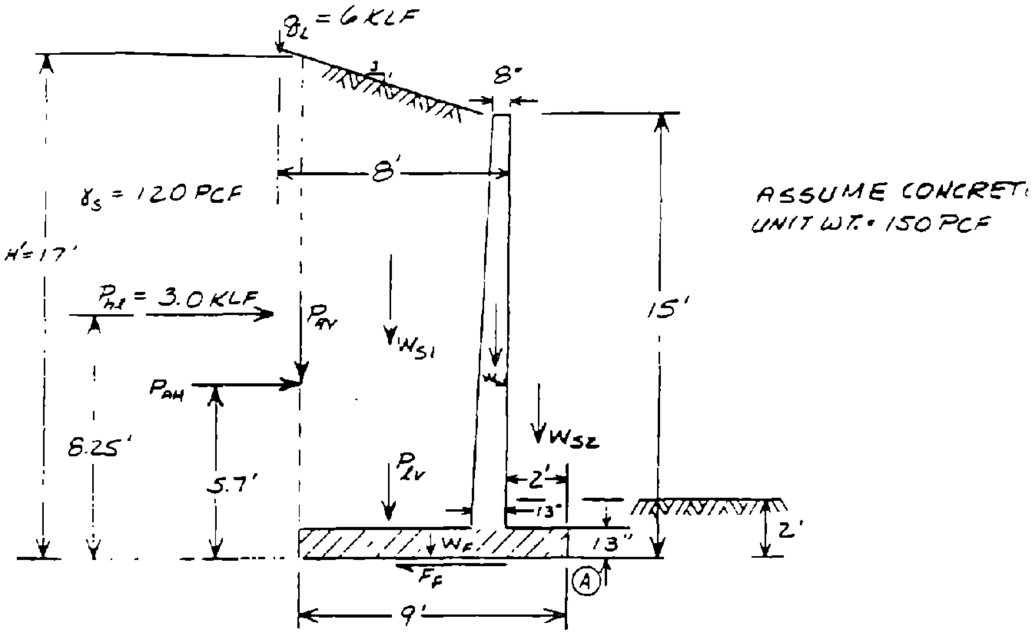
The lateral case will govern design.

The lateral earth pressures, in this case, will be computed by the semi-empirical method in figure 3-4. The sand backfill may be classified as type 3, i.e. residual soil with stones and fines and granular materials. The slope angle $\beta = \tan^{-1}(1/3) = 18.4°$. Referring to figure 3-4, $K_V = 15$ psf/ft and $K_h = 50$ psf/ft are the equivalent fluid pressures for vertical and horizontal earth loads on the retaining wall.

Select wall dimensions for initial trial per the design hints for cantilever walls, table 3-13.

1. Footing width, $3 = 0.60H = 0.6(15) = 9$ feet.
2. Extent of footing beyond face of wall = $0.21B$ or $0.21(9)$ = 1.89 feet, say 2 feet.
3. Wall thickness at top = 8 inches.
4. Wall thickness at base = $0.12B$ = 1.08 feet = 13 inches.
5. Footing thickness = $0.12B$ = 13 inches.

Consider the free body diagram on the next page. (Note that passive earth pressures are ignored here.)



Compute forces per foot of wall:

$$W_W = 1/2\left(\frac{8H3}{12}\right)(13.9')(0.15\ \text{kcf}) = 1.91\ \text{klf}$$

$$W_F = 9(1.1)(0.15\ \text{kcf}) = 1.49\ \text{klf}$$

$$W_{S1} = 6'(14)(0.12\ \text{kcf}) = 10.1\ \text{klf}$$

$$W_{S2} = 1'(2')(0.12\ \text{kcf}) = 0.24\ \text{klf}$$

$$P_{AH} = 1/2\ K_{AH}H^{12} = 1/2(50)(17)^2 = 7.23\ \text{klf}$$

$$P_{AV} = 1/2\ K_{AV}H^{12} = 1/2(15)(17)^2 = 2.17\ \text{klf}$$

The vertical influence of the line load on the footing may be computed as described in figure 3-6b.

$$\overline{ef} = 2\left[\frac{16}{\tan\ 60}\right] = 18.5\ \text{ft}$$

$$\Delta\sigma_V = \frac{q_L}{\overline{ef}} = \frac{6.0}{18.5} = 0.32\ \text{ksf (eq. 3-8)}$$

Considering the portion of $\Delta\sigma_V$ over the heel of the wall gives $P_{lv} = 6\Delta\sigma_V$, or $P_{lv} = 6(0.32) = 1.92$ klf.

The frictional resistance force F_f is then

$F_f = (\Sigma F_v)\tan\delta$

where ΣF_v = sum of vertical loads normal to footing base, and $\delta = 2/3\phi = 22°$, i.e. angle of friction between concrete and sand.

$$= [W_w + W_F + W_{S1} + W_{S2} + P_{AV} + P_{lv}]\tan 22°$$

$$F_f = 7.20 \text{ k/ft}$$

4B.2.1 External Stability

Consider External Stability

+ Overturning—sum moments about point A

$$F.S. = \frac{\Sigma M \text{ resisting}}{\Sigma M \text{ driving}} = \frac{1'W_{S2} + 2.5'W_W + 6'W_{S1} + 4.5'W_F + 6'P_{RV} + 9'P_{AV}}{5.7P_{AH} + 8.25P_{nc}}$$

$$= \frac{103.33}{65.96} = 1.57 > 1.5 \text{ so, OK!}$$

+ Sliding at base—sum horizontal forces

$$F.S. = \frac{\Sigma F \text{ resisting}}{\Sigma F \text{ driving}} = \frac{F_f}{P_{AH} + P_{\ell H}}$$

$$= \frac{7.20}{10.23} = 0.70 < 1.5 \text{ so, no good.}$$

The net horizontal load required for $F.S.$ = 1.5 is 10.23(1.5) – 7.59 = 7.76 klf. A portion of this would be furnished if the passive soil resistance was considered. For a level back-fill—as is the case of the upper wall being considered—K_p is given by equation 3-21.

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = 3.39 \text{ for } \phi = 33°$$

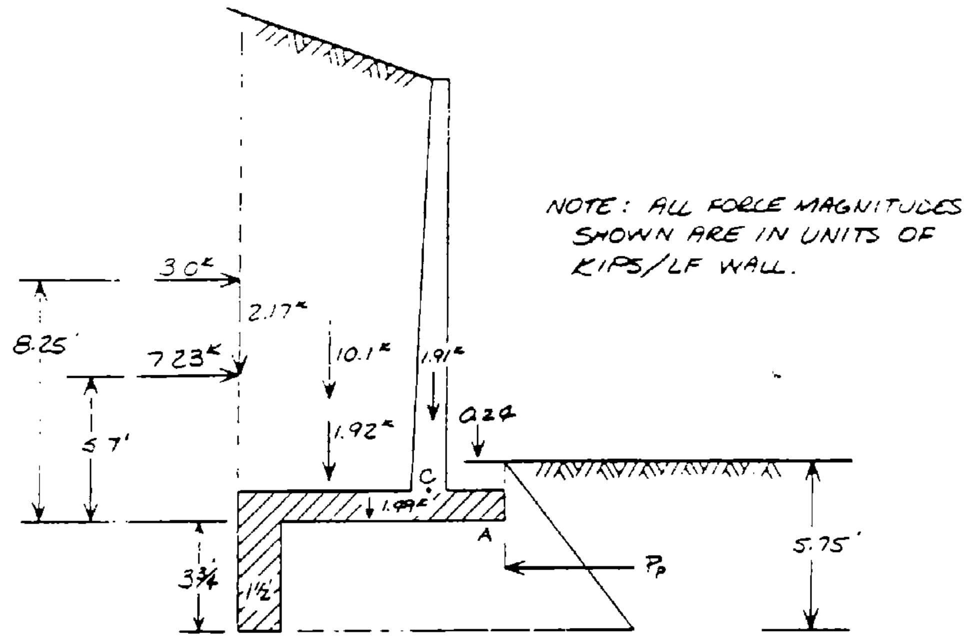
$$\text{Passive resistance} = 1/2\gamma K_p d^2 = 1/2(0.12)(3.39)(2)^2$$

$$= 0.81 \text{ klf}$$

Including passive resistance in computation gives

$$F.S. = \frac{7.20 + 0.81}{10.23} = 0.78 < 1.5, \text{ so, no good.}$$

Structure must be redesigned. Add a key of dimension 1-1/2 feet by 1-3/4 feet as shown on the revised free body diagram below.



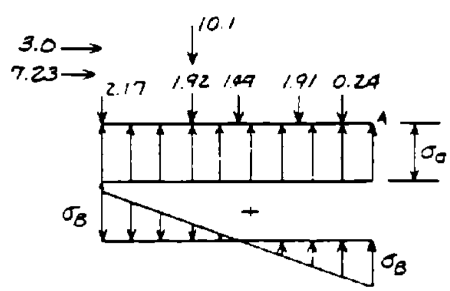
$$P_p = 1/2\gamma K_{pd}{}^{12} = 1/2(0.12)(3.9)(5.75)^2$$

$$= 7.74 \text{ k/lf}$$

$$\text{Now } F.S. = \frac{7.20 + 7.74}{10.23} = 1.46 \approx 1.50 \text{ so, OK.}$$

+ Check bearing pressure at point A

Idealize the bearing pressures as a trapezoidal shape indicated below.



$$\sigma_A = \frac{\Sigma F_V}{B} = \frac{2.17 + 10.1 + 1.92 + 1.49 + 1.91 + 0.24}{9} = \frac{17.83}{9} = 1.98 \text{ ksf}$$

Locate resultant ΣF_V by summation of moments about A.

$$Y_{FV} = \frac{2.17(9) + (10.1 + 1.92)6 + 1.49(4.5) + 1.91(2.5) + 1.0(24) - 5.7(7.23) - 8.25(3.0)}{17.83}$$

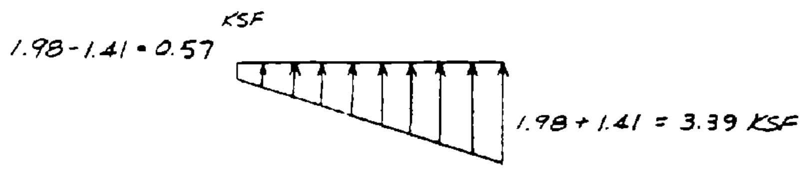
$= 3.43$ feet. (Inside of middle 1/3, OK.)

Moment induced stress σ_B is

$$\sigma_B = \frac{\Sigma F_V\, e}{S} = \frac{17.83^K(4.5 - 3.43)}{9^2/6}$$

$$= 1.41 \text{ ksf/ft}$$

Resultant stress distribution is



Since maximum bearing stress < allowable (i.e. 3.39 < 4.0 ksf) and minimum stress is compressive, the footing is adequate as proportioned.

4B.2.2 Internal Stability

Consider internal stability—follow procedure outlined on pages 155 to 165.

1. Compute moment at the base of wall stem point C

$$M_C = 3(8.25 - 1.1) + 7.23(5.7 - 1.1) = 54.71 \text{ k/ft}$$

2. From equation 3-57

$$d = t - (2 + d_r/2) \text{ say use \#6 rebar, } d_r = 0.75 \text{ inches}$$

$$d = 13 - \left(2 + \frac{0.75}{2}\right) = 10.62 \text{ inches}$$

$$F = \frac{bd^2}{12{,}000} = \frac{12(10.62)^2}{12{,}000} = 0.11$$

Assuming use of grade 60 steel and f_c = 3,500 psi

$$K = 248 \text{ from table 3-14}$$

$$KF = 248(0.11) = 27.28 < M_C = 54.71$$

Therefore, compression steel would be required or else stem of wall could be thickened.

$$\text{Try } t = 17 \text{ inches; } d = 17 - \left(2 + \frac{0.75}{2}\right)$$

$$= 14.63 \text{ inches}$$

$$\therefore F' = \frac{bd^2}{12{,}000} = \frac{12(14.63)^2}{12{,}000} = 0.21$$

$$KF' = 248(0.21) = 53.04 < 54.71$$

So, use t = 17 inches rather than compression steel.

3. From equation 3-59,

$$A_S = \frac{M}{ad} = \frac{54.71}{(1.76)(14.63)} = 2.12 \text{ in}^2\text{/ft}$$

So, five #6 bars/foot could be used, but a spacing of less than 4 inches is undesirable. Switching to three #8 bars/foot would provide A_S = 2.35 in^2/ft wall at a spacing of 4 inches.

Gross cross-sectional area and steel area can be reduced by the same process as above, as moment reduces with height above the base of the wall.

Minimum vertical reinforcement of the front face of the wall may be calculated by use of the coefficients of minimum reinforcement given in table 3-17, but only one-third of that minimum is required on the front face. So,

$$A_{SF} = 1/3(0.0012)A_{concrete}$$

$$= 0.004(17)(12)$$

$$= 0.082 \text{ in}^2\text{/ft}$$

Use one #3 per foot of wall.

The minimum horizontal reinforcement running the length of the wall may be divided equally between the front and back faces. The coefficient of minimum reinforcement from table 3-17 for grade 60 horizontal steel is 0.0020.

$$A_{SH} = A_{con}(0.0020) = 1/2(8 + 17)(15')(12)(0.0020)$$

$$= 4.5 \text{ in}^2$$

Splitting this minimum horizontal steel to front and back, 2.25 in^2 will be required per side. From table 3-5, select twelve #4 bars with spacing = $\frac{15(12)}{12}$ = 15 inches (A_{SH} = 2.36 in^2/12-#4 bars).

Check shear stress at the base of the stem:

$$\Sigma F_H = 3.0 + 7.23 = 10.23 \text{ k/ft}$$

4. From equation 3-61,

$$V_C = \frac{\text{horiz. force/ft}}{\text{stem area/ft}} = \frac{10{,}230 \text{ lb}}{17 \text{ in}(12)}$$

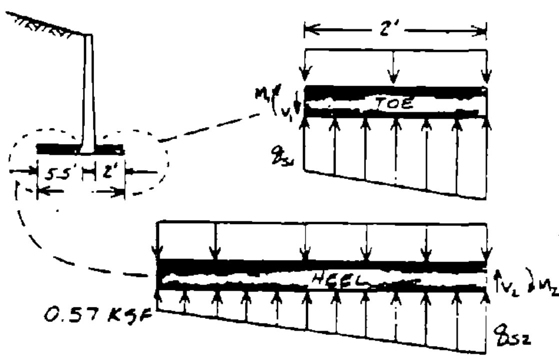
$$= 50.2 \text{ psi}$$

Compare to allowable $1.1\sqrt{f'_c} = 1.1\sqrt{3{,}500}$

$$= 65 \text{ psi} > 50.2 \text{ psi}$$

Therefore, shear is OK.

5. Design steel in wall footing in the same fashion as above. Moments for design may be calculated by considering the free body diagrams of the heel and toe of the footings (see figure 3-47).



$$0.15\left(\frac{13}{12}\right) = 0.16 \text{ ksf (ftg. width)}$$

$$q_{s1} = \frac{9 - 2}{9}(3.39 + 0.57)$$

$$= 3.08 \text{ ksf}$$

3.39 ksf (see ftg. stress dist.)

0.16 + 16(0.12) = 2.08

$$q_{S2} = \frac{5.5}{9}(3.39 + 0.57)$$

$$= 2.42 \text{ ksf}$$

Design moments and shears in the footing are then,

<u>TOE</u> $M_1 = 1/2(3.39 + 3.08)(2)\left[\frac{1}{3}(2)\frac{2(3.39) + 3.08}{3.39 + 3.08}\right] - 0.16(2)^2(1/2)$

$= 6.57 - 0.32 = 6.25$ ft-kips/ft

$V_1\downarrow = 1/2(3.39 + 3.08)(2) - 0.16(2)$

$= 6.79$ kips/ft

<u>HEEL</u> $M_2 = -1/2(0.57 + 2.42)(5.5)\left[\frac{1}{3}(5.5)\frac{2(0.57) + 2.42}{0.57 + 2.42}\right] + 2.08(5.5)^2(1/2)$

$= -17.95 + 31.46 = 13.51$ ft.-kips/ft

$V_2\uparrow = 2.08(5.5) - 1/2(0.57 + 2.42)(5.5)$

$= 3.22$ kips/ft

For a footing of 13-inch thickness, say d = 10 inches, use equation 3-59 to compute steel required.

$$A_S = \frac{M}{ad}$$

where M is computed above

a = 1.76 as before
d = 10 inches

$$A_{S1} = \frac{M_1}{ad} = \frac{6.25}{1.76(10)} = 0.36 \text{ in}^2/\text{ft}$$

So use two #6 bars, A_s = 0.39 in^2 in the bottom of the footing.

spacing = 6 inches

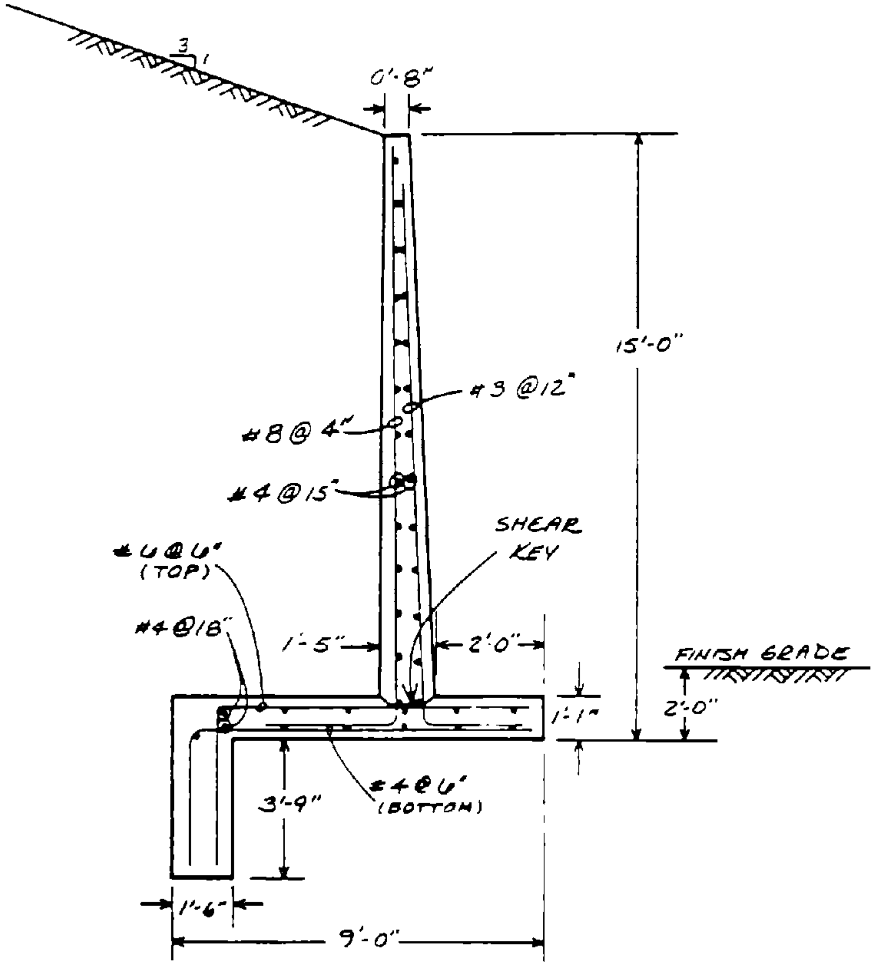
$$A_{S2} = \frac{M_2}{ad} = \frac{13.51}{176(10)} = 0.77 \text{ in}^2/\text{ft}$$

So use two #6 bars, A_S = 0.88 in^2 in the top of the footing.

spacing = 6 inches

<u>Check shear stresses:</u>

$$V_{C1} = \frac{6.79}{13(12)} = 43.5 \text{ psi} < 65 \text{ psi, so OK.}$$



Geologic Section and Cross-Section of
Proposed Gabion Wall

Scale 1"–20'

4B.3 Additional Design Considerations

4B.3.1 Drainage

The seasonally high ground water table requires that engineered drains be provided for both walls. In the case of the upper wall, a vertical drain is recommended. The lower wall should be constructed with adequate drainage as outlined in section 3F.

4B.3.2 Footing Embedment

The footings are embedded 2 feet not only to increase the allowable soil bearing pressure and passive resistance, but to provide adequate frost protection. In some areas an embedment greater than 2 feet may be required. Consult local building codes for information on embedment requirements.

4B.4 Construction Considerations

Typically, cantilever walls are constructed in three phases. Phase one involves excavating for and constructing the footing, including appropriate reinforcing steel to extend into the wall stem. A shear key should be formed in the wet concrete at the location of the stem of the wall. The second phase of construction involves fabricating, reinforcing, forming, and placing concrete in the wall stem. The third phase of construction involves the construction of drains and backfilling.

Assuming that conventional excavation techniques can be employed to dig the footings, concrete walls can be constructed with a minimum of special equipment. In general, special equipment requirements relate to the method of placement of the concrete. Depending on the location of the wall (uphill or downhill of the access), the concrete may be placed into the forms directly from the ready-mix truck or it may be placed with a crane and hopper or a pump truck. Not including the latter, concrete vibrators would also be required.

A tight, enforceable specification on concrete is one of the keys to successfully constructing a concrete structure in a remote area. With minor modifications, most bridge specifications can also be used for retaining wall construction. Particular attention is called to the sections on batching, mixing and delivery, adverse weather concrete, curing of concrete, and opening to traffic.

In addition to close control of the batching, placement, and curing of concrete, considerable attention should be paid to foundation preparation and backfill placement. Backfill should be placed in accordance with the guidelines presented in section 3E.2 of this guide (maximum dry density of 92 to 93 percent, and placement moisture content within –2 percent of optimum as determined by AASHTO T-99). The following guidelines could be used for backfill placement.

4B.5 Backfill Placement

No backfill shall be placed behind concrete retaining walls until the concrete in the wall reaches the specified 28-day strength as determined by compressive strength tests on cylinders poured with the wall.

All backfill shall be placed in horizontal lifts with the following thicknesses:

Material Type	Uncompacted lift thickness (inches)
Clean sands and gravels	12
Silts, clays and silt clay mixtures	9

Subsequently, the loose backfill shall be mechanically compacted to 92 to 93 percent of the maximum dry density obtained by AASHTO T-99. For moisture sensitive soils, the water content shall be controlled in the range of ±2 percent of optimum, as determined by AASHTO T-99.

Compaction within 5 feet of the back of the wall shall be accomplished with lightweight, hand-operated compaction equipment. Until the wall is completely backfilled, no heavy construction equipment shall be operated within 5 feet of the back of the wall.

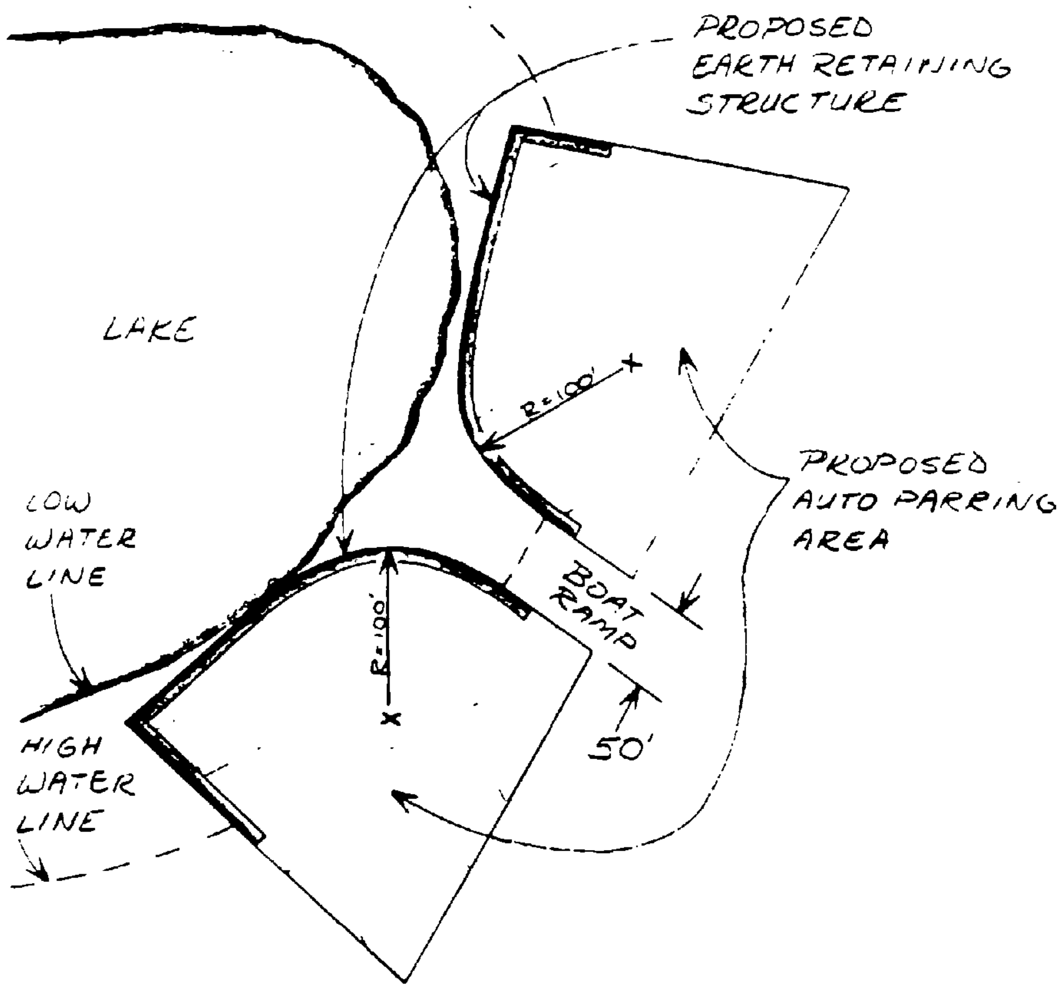
4C Bin Wall

4C.1 Problem Statement

An auto parking area is required adjacent to a boat ramp on the shore of a glacial lake. Due to seasonal water level variations, the lot must be elevated 9 feet above the existing ground level. Planners of the boat ramp project have designed the walls with horizontal curves away from the boat ramp (see plan view of proposed boat ramp).

A program of subsurface exploration revealed that in the vicinity of the retaining structures the foundation soil is a very stiff glacial till (bouldery clay) to a depth of at least 25 feet. The overconsolidation ratio of the till was determined in the laboratory to be 11.5 at a depth of 17.5 feet.

Backfill material will be a local source of uniform, well-rounded, clean outwash gravel with some sand, with cobbles and boulders removed. Previous investigations showed that ϕ = 36° and γ = 135 pcf upon compaction.



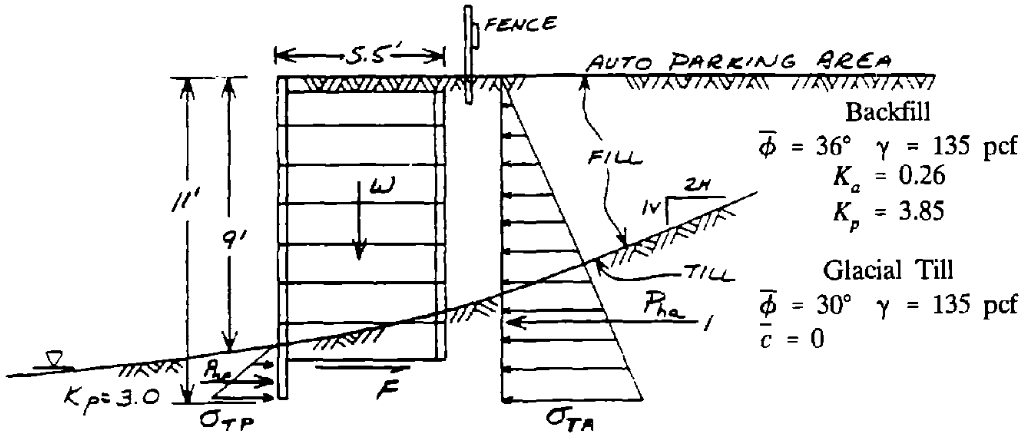
Plan View of Proposed Boat Ramp

4C.2 Design Calculations

The owner desires a galvanized bin-type retaining wall due to its known economy in the project vicinity. The analysis for essentially all types of bins and cribs will be substantially similar.

<u>Check external stability</u>

The wall and forces acting on it are shown in the following figure. Lateral earth pressures will be computed by Rankine earth pressure coefficients.



$$P_{ha} = 1/2\gamma H^2 K_a \quad \text{(equation 3-11)}$$

$$= 1/2(0.135 \text{ kcf})(11 \text{ ft})^2(0.26)$$

$$= 2.12 \text{ k/ft}$$

$$P_{hp} = 1/2\gamma H^2 K_p \quad \text{(equation 3-19)}$$

$$= 1/2(0.135 \text{ kcf})(2 \text{ ft})^2(3.85)$$

$$= 1.04 \text{ kp/ft}$$

$$F_f = W\tan\phi_{till} \quad W = \text{wt. of soil above bottom of stringer}$$

$$= (0.135 \text{ kcf})(9.67 \text{ ft})(5.5 \text{ ft})\tan 30°$$

$$F_f = 4.15 \text{ kip/ft}$$

Lateral loading on gravity structure due to auto parking is negligible.

<u>Sliding at base</u>

$$F.S. = \frac{\Sigma \text{ resisting forces}}{\Sigma \text{ driving forces}} \quad \text{(equation 3-48)}$$

$$F.S._{\min} = \frac{F_f}{P_{ha}} = \frac{4.15}{2.12} = 1.96$$

$$F.S._{\max} = \frac{F_f + P_{hp}}{P_{ha}} = \frac{4.15 + 1.13}{2.12} = 2.49$$

Both factors of safety are acceptable. No reliance on passive earth pressures is required. If such were required, the structural ability of the bin wall members to develop the same F.S. should be double-checked with the manufacturer.

Overturning stability

$$F.S. = \frac{\Sigma\,\text{resisting moments}}{\Sigma\,\text{overturning moments}} \quad \text{(equation 3-46)}$$

Summation of moments above toe of wall. For this analysis, F_f should be assumed to act at the same level as the toe, so that it does not contribute to F.S.

$$F.S._{min} = \frac{w(5.5/2)}{P_{ha}(11/3)} = \frac{7.18(5.5/2)}{2.12(11/3)}$$

$$= 2.54$$

$$F.S._{max} = \frac{w(5.5/2) + 1.13(2/3)}{2.12(11/3)} \quad \text{(with passive resistance)}$$

$$= 2.64$$

So, structure is stable against overturning.

Bearing capacity

For an overconsolidated till, with O.C.R. = 11.5, bearing capacity of a 9-foot gravity structure is not a consideration. For less competent foundation materials, bearing capacity should be considered, including the moment-induced stress as in the cantilever wall example.

4C.3 Additional Design Considerations

4C.3.1 Drainage

The clean outwash gravels proposed for backfill can be considered free-draining and not subject to piping of fines. Although the bouldery clay till is not free-draining, the cohesive nature of the material should prevent piping of fines into the backfill in the bins. Hence, assuming that the facing of the bins will allow seepage, no formal drainage system will be required. For less favorable conditions, however, an engineered drainage system may be required.

4C.3.2 Erosion Protection

The toe of the wall should be protected with riprap to prevent wave-generated erosion of soil away from the structure. Refer to the *USBR Earth Manual* for the design of riprap protection.

4C.4 Construction Considerations

The major advantages of bin and crib wall construction include relative ease of construction and lack of need for special equipment to assemble the units. The major disadvantages are the problems associated with field modification of prefabricated parts to fit changed subsurface conditions. The latter is particularly problematic for steel and concrete structures. With timber structures, although field modification is relatively easy to accomplish, adequate treatment of freshly cut or drilled surfaces to prevent decay is questionable.

For the galvanized metal bin wall system selected, consideration should be given to the type 2 bin walls due to their somewhat easier construction.

An important consideration in the construction of bin walls is the compaction of backfill around (in front of) the embedded portion of the wall facing. The latter is particularly important if the wall design requires mobilization of passive resistance for stability.

4D Gabion Wall

4D.1 Problem Statement

A cut is to be made on a talus-covered bedrock bench. The talus is angular basalt from the cliff above the bench. The width of the roadway is to be 24 feet. Bedrock surface is exposed for the outer 4 feet of the bench. Hence, a 20-foot widening of the existing bench is required. The talus slope varies in slope angle from 2H:1V at the base to a height of 12 feet, then rises at 1-1/4 H:1V for 40 feet.

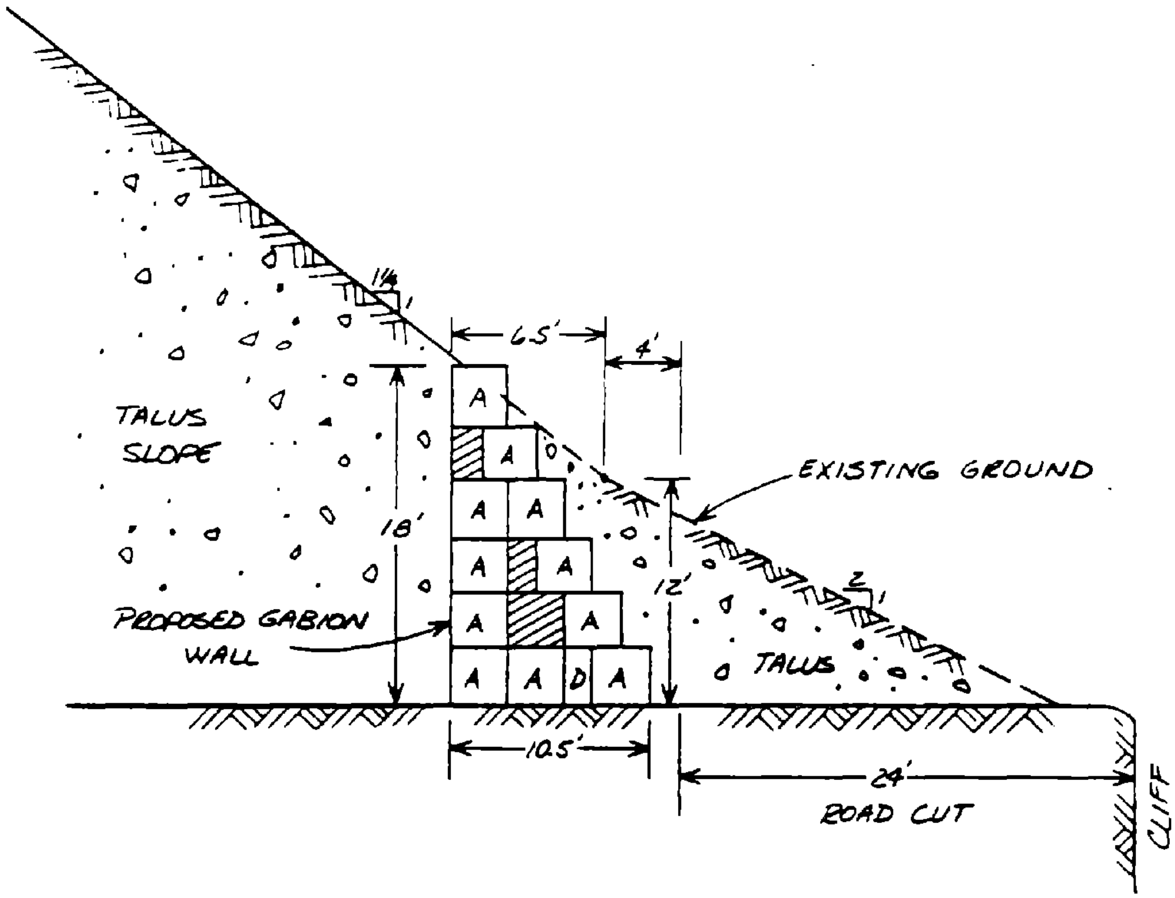
Investigation of talus deposits in the vicinity and at the site reveals that most of the deposits lie at 1-1/4 H:1V slopes. Hence, the angle of repose is determined to be 38°.

Economic considerations and retention of the natural beauty of the cut lead to the use of a gabion wall.

A typical front face-stepped wall is shown on the figure for a surcharge condition.

The gabion fill will be clean angular basalt pieces from excavated talus deposit. The in-place unit weight of the wall will be 130 pcf.

A determination of overturning and sliding stability is required. Since the foundation is bedrock, bearing capacity and long-term slope stability are not concerns.



Geologic Section and Cross Section of
Proposed Gabion Wall

Scale 1'-20'

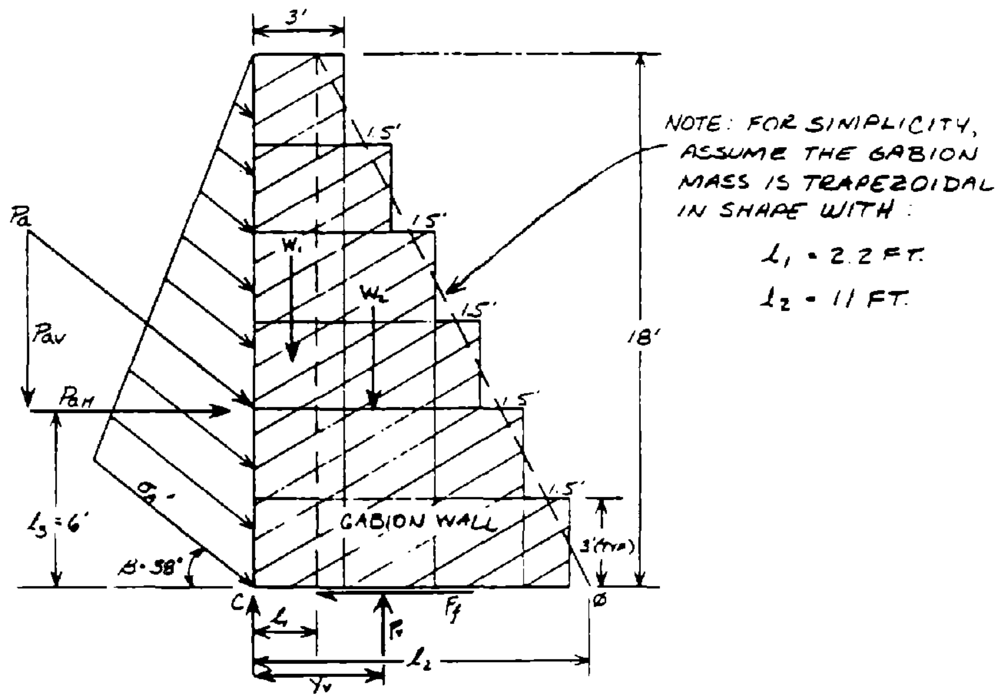
4D.2 Design Calculations

To determine lateral earth pressure on the back side of the wall, a Rankine coefficient of active earth pressure may be used.

From equation 3-12,

$$K_A = \cos\beta\left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]$$

But for $\beta = \phi$ as in this case, $K_A = \cos\beta$, or $K_A = 0.79$. Thus, consider the following free body diagram.



The active earth pressure, P_A, acting at an angle of 38° to the back of the wall, may be computed with equation 3-11, where

$$P_a = 1/2\gamma H^2 K_a$$

$$= 1/2(130\ \text{lbs/ft}^3)(18\ \text{ft})^2(0.79)$$

$$= 16.64\ \text{kips/ft}$$

$$P_{av} = P_a \sin\beta = (16.68\ \text{kips/ft})\sin 38°$$

$$P_{av} = 10.24\ \text{kips/ft}$$

$$P_{ah} = P_a \cos\beta = (16.68\ \text{kips/ft})\cos 36°$$

$$P_{ah} = 13.49\ \text{kips/ft}$$

Compute the location of the vertical resultant acting on the base of the wall by summing the moments about point C and setting equal to zero.

$$\Sigma M_C = 0$$

Hence, from the figure above,

$$\Sigma M_C = P_{AH}\ell_3 + w_1(\ell_1/2) + w_2\left[\ell_1 + \left(\frac{\ell_2 - \ell_1}{3}\right)\right] - (w_1 + w_2 + P_{AV})y_V = 0$$

$$= (13.4 \text{ kips/ft})(6 \text{ ft}) + (2.2 \text{ ft})(18 \text{ ft})(0.13 \text{ kips/ft}^3)(1.1 \text{ ft}) +$$

$$1/2(8.8 \text{ ft})(18 \text{ ft})(0.13 \text{ kips/ft}^3)(2.2 \text{ ft} + 8.8/3 \text{ ft}) - y_V(25.7 \text{ kips})$$

$$= 0$$

$$y_V = \frac{7.87 + 5.7 + 52.85}{25.7} \text{ ft}$$

$y_V = 5.3$ ft $\therefore P_V$ is located within the middle 1/3 of base.

Determine the factor of safety against overturning about point 0 (the toe).

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{driving}}$$

$$= \frac{P_{AV}l_2 + w_1\left[\frac{l_1}{2} + (l_2 - l)\right] + w_2\left[\frac{2(l_2 - l)}{3}\right]}{P_{AH}l_3}$$

$$= \frac{(10.2 \text{ kips})(11.0 \text{ ft}) + (5.2 \text{ kips})(9.9 \text{ ft}) + (10.3 \text{ kips})(5.8 \text{ ft})}{(13.1 \text{ kips})(6.0 \text{ ft})}$$

$FS = 2.8 > 1.5$ $\therefore$ OK (The factor of safety against overturning should be checked at the base of each layer.)

Now determine factor of safety against sliding. First, the frictional resistance of the gabion wall on bedrock must be estimated. Per section 31.2.3, generally friction angles for determination of sliding resistance are taken as $\delta = 2\phi/3$. So, in this case, the friction factor, f, is

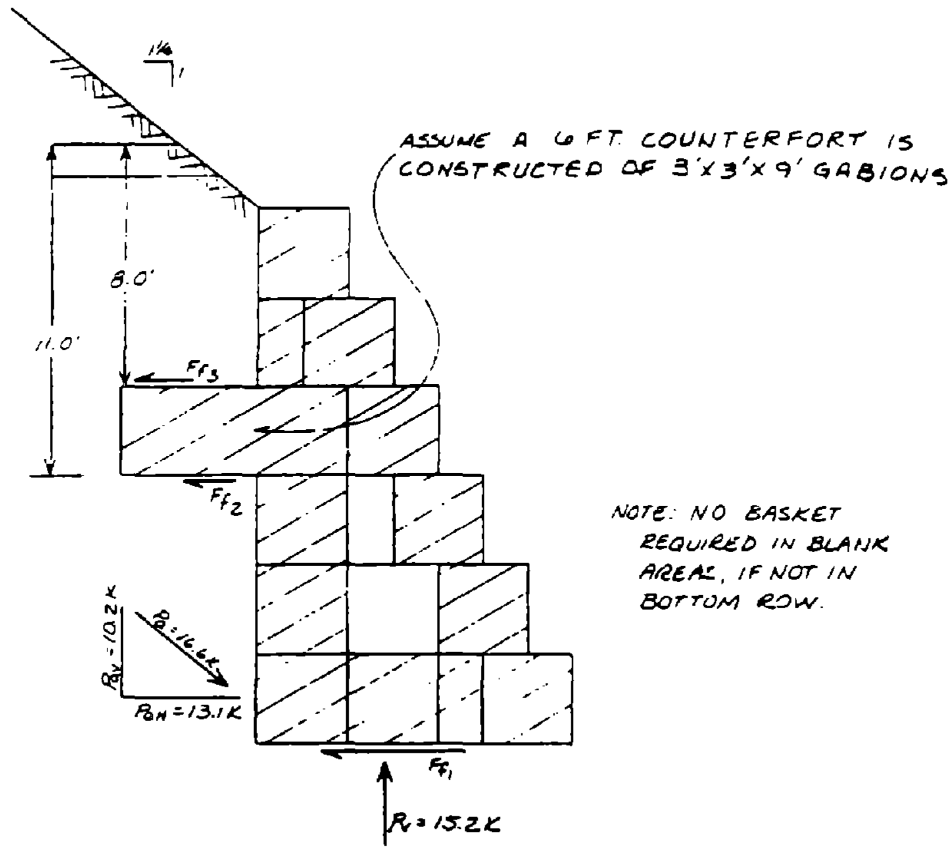
$$f = \tan 2\phi/3 = \tan 2(38)/3 = 0.47$$

$$F.S. = \frac{\Sigma \text{ resisting forces}}{\Sigma \text{ driving forces}} = \frac{f\Sigma \text{ normal forces on wall}}{\Sigma \text{ horizontal earth force}}$$

$$= \frac{f(P_{AV} + P_V)}{P_{AH}} = \frac{(0.47)(10.24 + 13.47)}{13.49}$$

$\therefore F.S. = 0.85 < 1.5$. This factor of safety is unacceptable.

Try to modify the wall by creating a counterfort in the back side, third tier from the top.



Evaluate F_{f_2} and F_{f_3} for a continuous counterfort.

Compute vertical forces on frictional surfaces due to average weight of rock above the respective surface.

$$N_2 = 11.0(0.13)(6) = 8.58 \text{ k/ft}$$

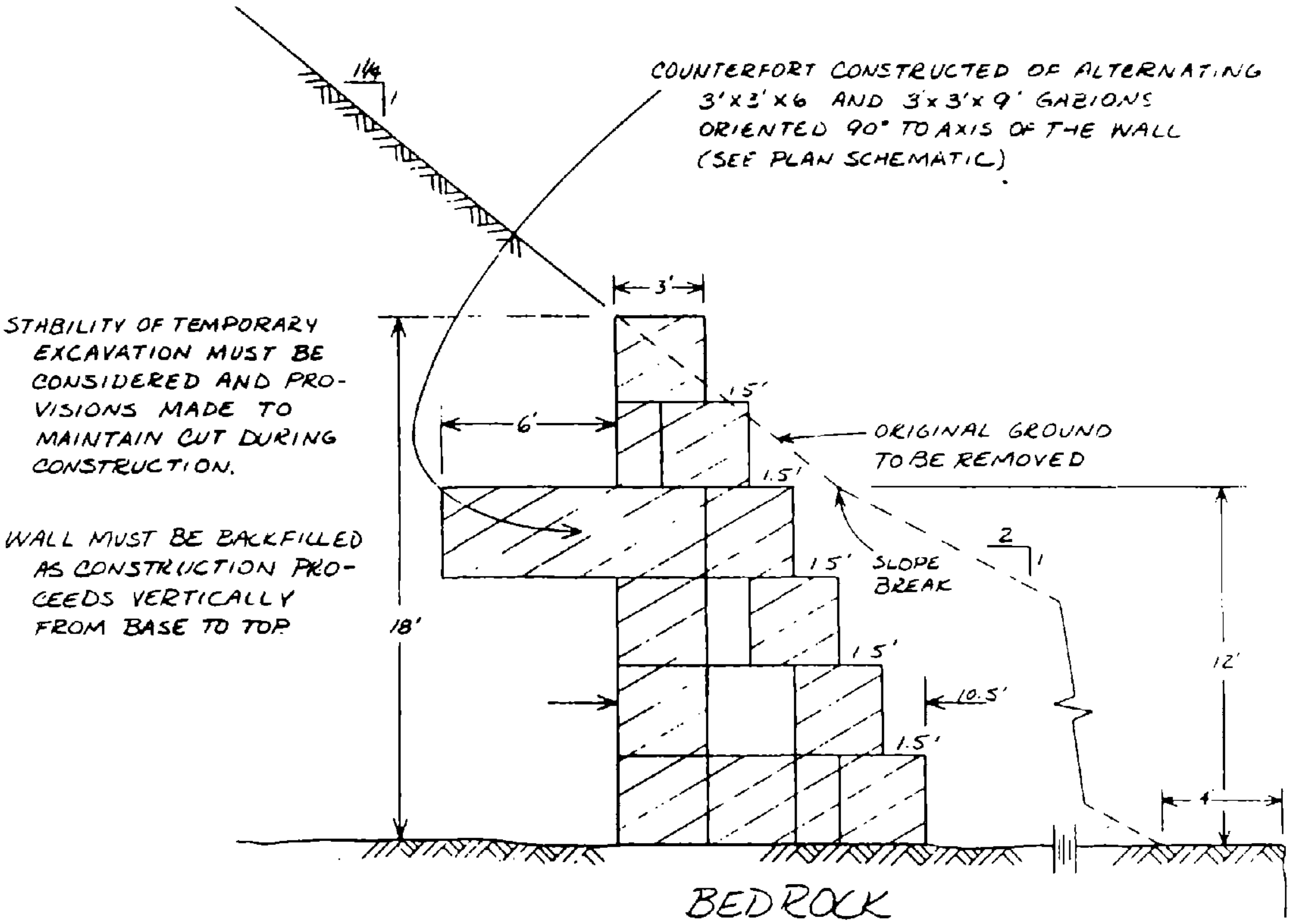
$$N_3 = 8.0(0.13)(6) = 6.24 \text{ k/ft}$$

At counterfort the frictional force that will be mobilized is:

$$\therefore F_{f_2} + F_{f_3} = (N_2 + N_3)\tan\frac{2\phi}{3}$$

$$= (8.58 + 6.24)\tan\frac{2(38°)}{3}$$

$$= 7.02 \text{ kips/ft of wall}$$



Section through Counterforted
Gabion Wall

Recompute factor of safety

$$F.S. = \frac{\tan\phi_b(P_V + P_{av}) + (N_2 + N_3)\tan\phi}{P_{aH}}$$

$$= \frac{(0.47)(10.24 + 15.21) + 7.02}{13.49} = 1.45$$

This factor of safety is within the limits of accuracy of the recommended minimum of 1.50, so it is judged acceptable.

Note: Addition of the counterforts will shift the location of the resultant, P_v, toward the heel of the wall.

As an alternative to using counterforts to improve sliding stability, the gabion wall could be keyed into the rock to develop passive resistance at the toe, thus forcing the critical failure surface through the stabilized gabion mass. Hence, the full frictional resistance of the fill rock would be mobilized.

Selection of one alternative over the other is purely a matter of economics and convenience.

4D.3 Additional Design Considerations

4D.3.1 Slope Stability

It should be anticipated that the talus slope will continue to develop, hence some ravelling of the slope will occur. The majority of this ravelling should be caught by the benched face of the gabion wall; however, provision should be made for cleaning the steps as they become charged with talus rock.

4D.3.2 Drainage

Due to the porous nature of the talus rock, no additional drainage provisions will be required.

4D.3.3 Sliding Friction

In the example problem, the friction angle of the bedrock stabilized gabion mass contact has been taken as 2/3 ϕ (ϕ is the friction angle of fill). For soil foundations, the angular nature of the stabilized rock fill would provide enough interlocking to mobilize all of the frictional resistance of the underlying materials.

4D.4 Construction Considerations

Specifications for the construction of gabion walls are available in most highway department specifications. Emphasis should be placed on site preparation and the placement of gabion fill.

With site preparation, care should be taken to avoid sharp, irregular asperities projecting from the foundation. Gabion fill should be placed with extreme care, particularly in an empty basket. Attention is called to these two points because of the problems associated with damaging or breaking the wires that make up the basket. Broken and damaged wires may seriously affect the overall stability and performance of a gabion wall. The severity of this problem is directly related to the increasing wall height.

A major consideration for the example problem relates to maintenance of the temporary backslope during construction. Past experience shows that short sections of oversteepened talus slopes will stand at slopes in the range of 45 to 50 degrees and steeper if small amounts of matrix are present. It is anticipated that the slope may fail if the entire wall length is cut at once. Hence, the wall should be constructed in short sections and backfilled as work progresses.

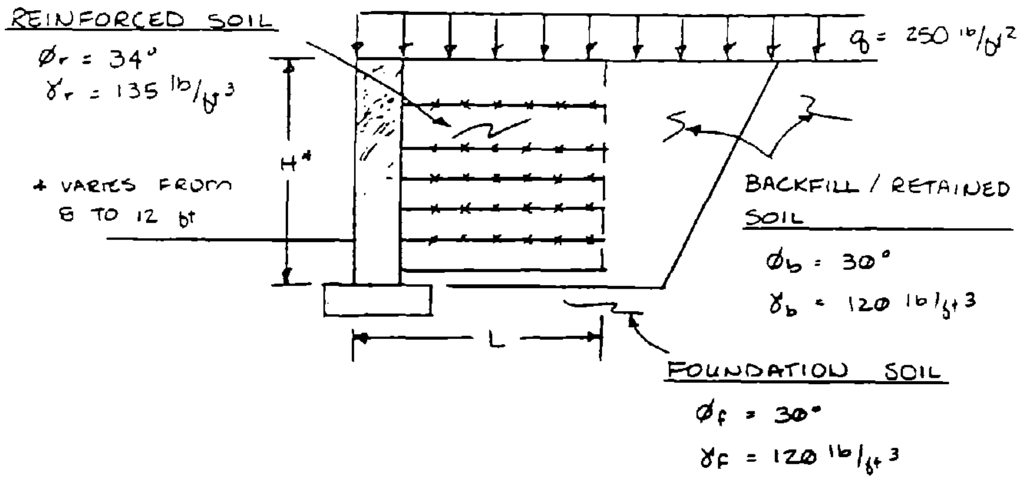
4E Reinforced Soil Retaining Walls

4E.1 Reinforced Soil Retaining Walls

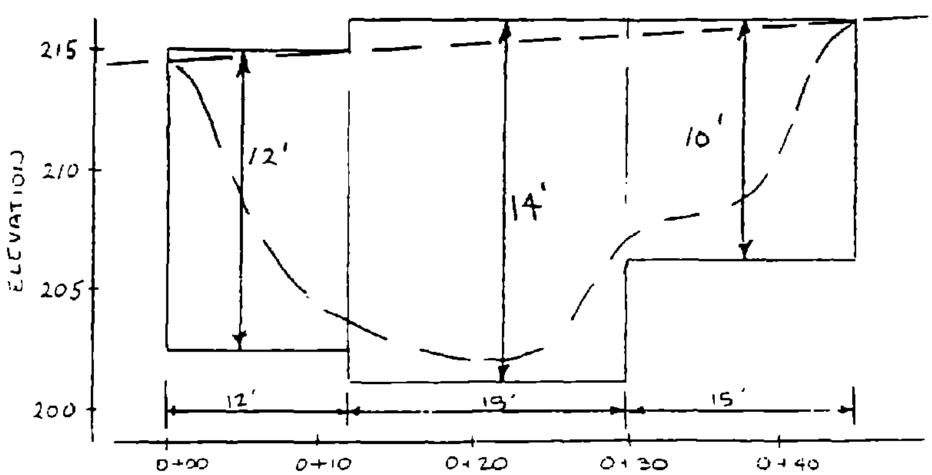
EXTERNAL STABILITY CALCULATIONS

Objective: Design a reinforced soil retaining wall given the typical section and wall profile shown below. Calculations will only be completed for the middle wall panel.

Wall section



Wall profile



4E.2 Design Calculations

EXTERNAL STABILITY CALCULATIONS

Required: to specify reinforcing material length of reinforcements and spacing of reinforcing material.

Solution:

<u>Step 1: Satisfy design assumptions and limitations.</u>

- Design assumptions and limitations appear to be satisfied.

<u>Step 2: Generate wall profile.</u>

- Required wall profile provided on page 318. The wall profile is based on project requirements.

<u>Step 3: Determine wall, loading, and soil parameters.</u>

- For middle wall panel:

$$H_{req} = 12 \text{ ft}$$

$$q = 250 \text{ lb/ft}^2$$

$$D = 2 \text{ ft minimum foundation embedment}$$

$$H = H_{req} + D = 12 \text{ ft} + 2 \text{ ft} = 14 \text{ ft}$$

Soil parameters:

$\phi_r = 34°$	
$\gamma_r = 135 \text{ lb/ft}^3$	$\phi_f = 30°$
	$\gamma_f = 120 \text{ lb/ft}^3$
$\phi_b = 30°$	$c_f = 0 \text{ lb/ft}^2$
$\gamma_r = 120 \text{ lb/ft}^3$	

<u>Step 4: Complete external stability analysis.</u>

- Calculate the Rankine active earth pressure for the backfill soil.

$$K_{A_b} = \tan^2(45° - \phi_b/2) = \tan^2(45° - 30/2) = 0.333$$

$$K_{A_b} = 0.33$$

Step 4.1: Check sliding.

- Determine the required length of reinforcement, L. Use an FS_{SL} of 1.5 in the equation.

$$L = L_{\min} \geq \frac{FS_{SL}(K_{A_b})H[q + 0.5(\gamma_b)H]}{[q + \gamma_r(H)]\tan\phi} \qquad (3\text{-}108)$$

- Use the smallest ϕ value of ϕ_f and ϕ_r.
  Use $\phi_p = 30° < \phi_r$:

- Calculation:

$$L_{\min} \geq \frac{1.5(0.33)14\ \text{ft}[250\ \text{lb/ft}^2 + 0.5(120\ \text{lb/ft}^3)14\ \text{ft}]}{[250\ \text{lb/ft}^2 + 135\ \text{lb/ft}^3(14\ \text{ft})]\tan 30°}$$

$\underline{\underline{L_{\min} \geq 6.18\ \text{ft}}}$ (sliding satisfied)

Step 4.2: Check overturning.

- Determine the required length of reinforcement, L. Use an FS_{OT} of 2.0 in the equation.

$$L \geq \sqrt{\frac{FS_{OT}(K_{A_b})H^2[\gamma_b(H) + 3q]}{3[q + \gamma_r(H)]}} \qquad (3\text{-}110)$$

$$L \geq \sqrt{\frac{2.0(0.33)(14\ \text{ft})^2[120\ \text{lb/ft}^3(14\ \text{ft}) + 3(250\ \text{lb/ft}^2)]}{3[250\ \text{lb/ft}^2 + 135\ \text{lb/ft}^3(14\ \text{ft})]}}$$

$\underline{\underline{L \geq 7.03\ \text{ft}}}$ (overturning satisfied)

Step 4.3: Check bearing capacity.

- Calculate eccentricity, e, of bearing pressure resultant force by summing moments about the center, $L/2$, of the reinforced soil zone considering counterclockwise movement positive (+).

  Use the largest L from steps 4.1 and 4.2:

$$L = 7.1\ \text{ft.}$$

$$e = \frac{K_{A_b}(H)^2[\gamma_b(H) + 3q]}{6L[\gamma_r H + q]} \tag{3-111}$$

$$= \frac{0.33(14\ \text{ft})^2[120\ \text{lb/ft}^3(14\ \text{ft}) + 3(250\ \text{lb/ft}^2)]}{6(7.1\ \text{ft})[135\ \text{lb/ft}^3(14\ \text{ft}) + 250\ \text{lb/ft}^2]}$$

$\underline{\underline{e = 1.74\ \text{ft}}}$

- Check $e < L/6$ to continue:

$$L/6 = 7.1\ \text{ft}/6 = 1.18\ \text{ft}$$

$$e = 1.74 > 1.18\ \text{ft} \Rightarrow \text{increase } L$$

$$\underline{\underline{e = 1.74}}$$

Increase *L* and recalculate *e*.

- Try $L = 10$ ft, recalculate e:

$$e = \frac{0.33(14\ \text{ft})^2[120\ \text{lb/ft}^3(14\ \text{ft} + 3(250\ \text{lb/ft}^2)]}{6(10\ \text{ft})[135\ \text{lb/ft}^3(14\ \text{ft}) + 250\ \text{lb/ft}^2]}$$

$\underline{\underline{e = 1.24\ \text{ft}}}$

- Check $e < L/6$ to continue:

$$L/6 = \frac{10\ \text{ft}}{6} = 1.67\ \text{ft}$$

$$\underline{\underline{e = 1.24\ \text{ft} < 1.67\ \text{ft}}}$$

- After several tries, try $L = 8.7$ ft. Recalculate e:

$$e = \frac{0.33(14\ \text{ft})^2[120\ \text{lb/ft}^3(14\ \text{ft}) + 3(250\ \text{lb/ft}^2)]}{6(87\ \text{ft})[135\ \text{lb/ft}^3(14\ \text{ft}) + 250\ \text{lb/ft}^2]}$$

$\underline{\underline{e = 1.42\ \text{ft}}}$

- Check $e < L/6$ to continue:

$$L/6 = \frac{8.7\ \text{ft}}{6} = 1.45\ \text{ft}$$

$$\underline{\underline{e = 1.42\ \text{ft} < 1.45\ \text{ft}}} \Rightarrow \text{continue}$$

- Calculate the equivalent footing width, B:

Use $L = 8.7$ ft

$$B = L - 2e = 8.7\ \text{ft} - 2(1.42\ \text{ft}) = 5.86\ \text{ft} \tag{3-112}$$

$\underline{\underline{B = 5.86\ \text{ft}}}$

- Calculate the applied bearing pressure, Q_a:

$$Q_a = \frac{\gamma_r(H)L + q(L)}{B} \tag{3-113}$$

$$= \frac{135\ \text{lb/ft}^3(14\ \text{ft})8.7\ \text{ft} + 250\ \text{lb/ft}^2(8.7\ \text{ft.})}{5.86\ \text{ft}}$$

$$\underline{\underline{Q_a = 3177.1\ \text{lb/ft}^2}}$$

- Calculate the ultimate bearing capacity, Q_{ult}, of the foundation:

$$Q_{ult} = C_f N_C + 0.5\gamma_r(B)N\gamma$$

From reference text @ $\phi_b = 30°$

$$N_C = 30.14$$
$$N_\gamma = 22.40$$

$$Q_{ult} = \phi\ \text{lb/ft}^2(30.14) + 0.5(135\ \text{lb/ft}^3)5.86\ \text{ft}(22.40)$$

$$\underline{\underline{Q_{ult} = 8{,}860.3\ \text{lb/ft}^2}}$$

- Calculate the factor of safety against bearing capacity failure, FS_{bc}:

$$FS_{bc} = Q_{ult}/Q_a = 8.860\ \text{lb/ft}^3/3{,}177.1\ \text{lb/ft}^2$$

$$\underline{\underline{FS_{bc} = 2.79 < 3.0}}$$

Bearing capacity not satisfied, increase L and redo step 4.3.

- Try $L = 9$ ft. Recalculate e:

$$e = \frac{0.33(14\ \text{ft})^2[(120\ \text{lb/ft}^3(14\ \text{ft}) + 3(250\ \text{lb/ft}^2)]}{6(9.0\ \text{ft})[135\ \text{lb/ft}^3(14\ \text{ft}) + 250\ \text{lb/ft}^2]}$$

$$\underline{\underline{e = 1.37\ \text{ft}}}$$

- Check $e < L/6$ to continue:

$$L/6 = 9.0/6 = 1.5\ \text{ft}$$

$$\underline{\underline{e = 1.37\ \text{ft} < 1.5\ \text{ft}}} \Rightarrow \text{continue}$$

- Calculate equivalent footing width, B:

$$B = L - 2e = 9 - 2(1.37\ \text{ft}) = 6.26\ \text{ft}$$

$$\underline{\underline{B = 6.26\ \text{ft}}}$$

- Calculate the applied bearing pressure, Q_a:

$$Q_a = \frac{\gamma_r(H)L - q(L)}{B}$$

$$= \frac{135\ \text{lb/ft}^3(14\ \text{ft})9.0\ \text{ft} + 250\ \text{lb/ft}^2(9.0\ \text{ft})}{6.26\ \text{ft}}$$

$$\underline{\underline{Q_a = 3{,}076.7\ \text{lb/ft}^2}}$$

- Calculate the ultimate bearing capacity, Q_{ult}, of the foundation:

$$Q_{ult} = C_f N_C + 0.5\gamma_r(B)N_\gamma$$

$$= \phi(30.14) + 0.5(135\ \text{lb/ft}^3)6.26\ \text{ft}(22.40)$$

$$\underline{\underline{Q_{ult} = 9{,}465.1\ \text{lb/ft}^2}}$$

- Calculate the factor of safety against bearing capacity failure, FS_{bc}:

$$FS_{bc} = Q_{ult}/Q_a = 9{,}465.1\ \text{lb/ft}^2/3{,}076.7\ \text{lb/ft}^2$$

$$\underline{\underline{FS_{bc} = 3.08 > 3.0}}\quad \text{(bearing capacity satisfied)}$$

Conclusions

External Stability Analysis Summary

Condition	L_{min}	FS
Sliding	6.2 ft.	1.5
Overturning	7.0 ft.	2.0
Bearing capacity	9.0 ft.	3.1 ⇒ controls

Use 9.0 ft.

4E.3 Internal Stability

Three types of walls having different reinforcing and facing material will be analyzed for internal stability.

Example #	Wall Type	Reinforcement Type	Facing Type
1	welded wire wall	wire mesh	welded wire mesh
2	geotextile wall	geotextile	shotcrete
3	keystone wall	geogrid	precast concrete blocks

4E.3.1 Welded Wire Wall

Step 5: Select reinforcement.

- Hilfiker welded wire inextensible grid reinforcement will be used.

- Assume a 9-foot minimum L from completed external stability calculations.

Step 6: Select facing.

- Welded wire mesh facing is typically used with welded wire reinforcement.

Step 7: Complete internal stability analysis.

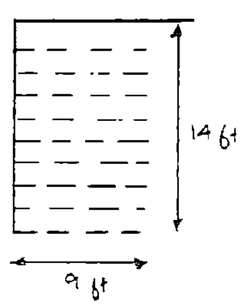
Step 7.1: Select failure surface and earth pressure coefficient.

- Hilfiker wall uses the assumption that the earth pressure coefficient is constant and equal to 0.65 (see figure 3-72). Based on a review of manufacturer's guidelines, the appropriate failure surface is Rankine.

$$K_o = 0.65$$

Step 7.2: Select spacing measurements.

- For welded wire walls, standard vertical spacing, S_z, between reinforcements is 1.5 feet. Two feet of fill will be used above the top reinforcement. This makes the design height of the wall 14 feet.



Layer #	Depth, Z (ft.)
1	2.0
2	3.5
3	5.0
4	6.5
5	8.0
6	9.5
7	11.0
8	12.5
9	14.0

Step 7.3: Check tensile stress in reinforcements and at connections.

- Horizontal earth pressure can be calculated at each layer using the equation below.

- Calculate $f(F_y)$, the allowable steel stress.

FHWA requires that $f(F_y) = 0.48\ F_y$ where F_y is the yield strength of the wire. The yield strength for Hilfiker wire mesh is 65,000 psi.

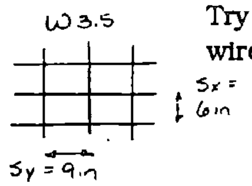
$$f(F_y) = 0.48(F_y) = 0.48(65{,}000\text{ psi}) = 31{,}200\text{ lb/in}^2$$

$$\underline{\underline{f(F_y) = 31{,}200\text{ lb/in}^2}}$$

- Compute the tensile stress, F_T, for each layer:

$$F_T = P_H/A_{CR}$$

where A_{CR} is the area of the mesh.



Try wire mesh w3.5 with A_{CR} = 0.035 in^2 (The standard wire mesh sizes can be obtained from the manufacturer.)

$$\underline{\underline{F_T = P_H/A_{CR}}}$$

Complete calculation for each layer:
(At layer 4,

$$F_T = 549.7\text{ lb}/0.035\text{ in}^2 = 15{,}705\text{ lb/in}^2)$$

- Compute the factor of safety against rupture, FS_R.

$$FS_R = f(F_y)/F_T = 31{,}200\text{ lb/in}^2/F_T$$

$$\underline{\underline{FS_R = 31{,}200\text{ lb/in}^2/F_T}}$$

Complete calculation for each layer.
(At layer 4,

F_{SR} = 31,200 lb/in^2/15,705 lb/in^2 = 2.0)

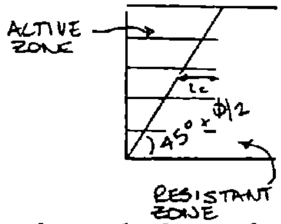
All layers must have $F_{SR} > 1.0$.

- Welded wire walls do not have a reduced reinforcement area at the wall face. Therefore, tensile stress at the facing connection is not a concern.

Step 7.4: Check pullout of reinforcement.

- Compute the number of transverse wires in the resistant zone. At Hilfiker recommendation, the failure plane is assumed to be inclined at 45° + U/2 to the horizontal and passing through the toe of the wall. (This is based on a K equal to 0.65.) See figure 3-72.

The length of reinforcement in the resistant zone, L_e, can be calculated.



$$L_e = L - [(H - z)\tan(45° - \phi/2)]$$

$$= 9.0 \text{ ft} - [(14 - z)\tan(45° - 34°/2)]$$

$$\underline{\underline{L_e = 1.556 \text{ ft} + 0.532(z)}}$$

Complete calculation for each layer.
(At layer 4,

$$L_e = 1.556 \text{ ft} + 0.532(6.5 \text{ ft}) = 5.01 \text{ ft})$$

As stated under step 7, the minimum length of reinforcement in the resistant zone is 3.0 ft.

- Calculate the number of transverse wires in the resistant region, N, for wire mesh w3.5. The spacing between transverse wires, S_y, is 9 inches This information can be obtained from the manufacturer.

$$N = L_e/S_y = L_e/9 \text{ in} = L_e/0.75 \text{ ft}$$

$$\underline{\underline{N = L_e/0.75 \text{ ft}}}$$

- Calculate the number of longitudinal wires per foot, m.

$$m = 1 \text{ ft}/S_x = 1 \text{ ft}/6 \text{ in} = 2$$

$$\underline{\underline{m = 2}}$$

Complete calculation for each layer.
(At layer 4, $m = 2$

$N = 5.01 \text{ ft}/0.75 \text{ ft} = 6.7)$

$$S_x = K_o(\gamma_r z + q) = 0.65[135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2]$$

$$\underline{\underline{S_x = 87.75 \text{ lb/ft}^3(z) + 162.5 \text{ lb/ft}^2}}$$

See table below for results.

- Calculate F_H, the horizontal load, that needs to be resisted by each reinforcement layer. Use 6-inch by 9-inch mat. The longitudinal spacing, S_x, is 6 inches. S_z is 1.5 feet.

$$P_H = \sigma_x(S_x)S_z = \sigma_x(6 \text{ in}/12 \text{ in})1.5 \text{ ft} = (0.75 \text{ ft}^2)\sigma_x$$

$$\underline{\underline{P_H = (0.75 \text{ ft}^2)\sigma_x}}$$

Complete calculation for each layer.
(At layer 4, σ_x = 732.9 lb/ft^2

P_H = 0.75 ft^2(732.9 lb/ft^2) = 549.7 lb

Layer No.	Depth z (ft)	σ_x (lbs/ft^2)	P_H (lbs/wire)	wire size	F_T (lbs/in^2)	L_e (ft)	m	N	P_t (lb/ft)	FS_P	FS_R
1	2.0	338.0	253.5	3.5	7,243	2.6*	2	3.5	1,275	2.5	4.3
2	3.5	469.6	352.2	3.5	10,063	3.4	2	4.6	2,101	3.0	3.1
3	5.0	601.3	450.9	3.5	12,884	4.2	2	5.6	3,187	3.5	2.4
4	6.5	732.9	549.7	3.5	15,705	5.0	2	6.7	4,605	4.2	2.0
5	8.0	864.5	648.4	3.5	18,525	5.8	2	7.7	6,253	4.8	1.7
6	9.5	996.1	747.1	3.5	21,346	6.6	2	8.8	8,258	5.5	1.5
7	11.0	1,128	845.8	3.5	24,166	7.4	2	9.9	10,565	6.2	1.3
8	12.5	1,259	944.5	3.5	26,987	8.2	2	10.9	13,062	6.9	1.2
9	14.0	1,391	1,043	3.5	29,807	9.0	2	12.0	15,957	7.6	1.1
* minimum L_e of 3.0 ft., use 3.0 ft.						>3.0				>1.5	>1.0

- Determine the total pullout resistance, P_t.

  Hilfiker has developed empirical equations for different soil types (Mitchell and Villet). These are listed below.

  d = wire diameter

Silty sand:

If $N(\gamma_r)z(d)$ > 113.6 lb/ft, use

$$P_t = 2{,}143 \text{ lb/ft} + \gamma_r(z)d[0.75(\pi)L_e(m)(2/3\tan\phi_r) - 17.61(N)]$$

If $N(\gamma_r)z(d) \leq$ 113.6 lb/ft, use

$$P_t = \gamma_r(z)d[0.75(\pi)L_e(m)(2/3\tan\phi_r) + 36.47(N)]$$

Washed sand:

$$P_t = 633 \text{ lb/ft} + \gamma_r(z)d[0.75(\pi)L_e(m)(2/3\tan\phi_r) + 36.8(N)]$$

Pea gravel:

$$P_t = 712 \text{ lb/ft} + \gamma_r(z)d[0.75(\pi)L_e(m)(2/3\tan\phi_r) + 38.1(N)]$$

For design example, assume most like washed sand. For w3.5 wire mesh, d = 0.0176 ft.

$$P_t = 633 \text{ lb/ft} + 135 \text{ lb/ft}^3(z)0.0176 \text{ ft}[0.75(\pi)L_e(2)(2/3\tan 34°) + 36.8(N)]$$

$$\underline{\underline{P_t = 633 \text{ lb/ft} + 2.376 \text{ lb/ft}^2(z)[2.119(L_e) + 36.8(N)]}}$$

Calculate P_t for each layer:

(At layer 4, z = 6.5 ft., L_e = 5.01 ft., N = 6.7

$$P_t = 633 \text{ lb/ft} + 2.376 \text{ lb/ft}^2(6.5 \text{ ft})[2.119(5.01 \text{ ft}) + 36.8(6.7)]$$

$$P_t = 4{,}604.8 \text{ lb/ft})$$

- Check the factor of safety against pullout, FS_p.

$$FS_p = \frac{P_t}{m(P_H)} \geq 1.5$$

$$\underline{\underline{FS_p = \frac{P_t}{2(P_H)} \geq 1.5}}$$

Calculate FS_p for each layer. If FS_p < 1.5, increase the length of the reinforcement or decrease the horizontal and vertical spacing, S_x and S_z and recalculate Step 7.4.

(At layer 4, P_t = 4604.8 lb/ft, P_H = 549.7 lb.

$$FS_p = \frac{4604.8 \text{ lb/ft}}{2(549.7 \text{ lb}} = 4.2)$$

<u>Step 7.5: Evaluate corrosion (optional).</u>

- Steel reinforcements should be designed to be corrosion resistant to ensure a minimum design life of 75 years. Guidelines to evaluate degradation of the wire mesh should be obtained from the manufacturer.

- For Hilfiker wire mesh, a 0.685-mil zinc coating is applied to the wires for corrosion protection.

FHWA guidelines state:

> 0.059 mil/yr loss for the first 2 years
> 0.016 mil/yr loss for subsequent years

The number of years required to corrode corrosion protection, y

0.685 mil = 0.059 mil/yr (2 yr) + 0.016 mil/yr (y)

$$\underline{\underline{y = 34.5 \text{ yrs}}}$$

- Calculate the loss of carbon steel at the end of the design life, T_C,

$$T_C = T_n - T_s$$

where T_n is the thickness of the wire at construction, which equals half the wire diameter, $d/2$.

$$T_n = d/2 = \frac{0.0176 \text{ ft}}{2} = \frac{0.211 \text{ in}}{2} = 0.106 \text{ in}$$

$$\underline{\underline{T_n = 0.106 \text{ in}}}$$

$$T_s = 0.059 \text{ mil/yr}[75 \text{ yr} - 35.4 \text{ yr}]$$

$$\underline{\underline{T_s = 2.34 \text{ mils} = 0.00234 \text{ in}}}$$

$$\underline{\underline{T_c = 0.106 \text{ in} - 0.00234 \text{ in} = 0.1037 \text{ in}}}$$

- The area of steel wire remaining after 75 years is A_L.

$$\underline{\underline{A_c = (T_c)^2\pi = (0.1037 \text{ in})^2\pi = 0.0338 \text{ in}^2}}$$

- According to the results shown in the table on page 292, the highest tensile force occurs in layer 9.

$$F_C = P_H/A_C = \frac{1043 \text{ lb/wire}}{0.0338 \text{ in}^2} = 30{,}896.7 \text{ lb/in}^2$$

$$\underline{\underline{F_C = 30{,}896.7 \text{ lb/in}^2}}$$

- Check factor of safety against corrosion, FS_c.

$$FS_c = \frac{F_y}{F_c} = \frac{65{,}000 \text{ psi}}{30{,}896.7 \text{ psi}} = 2.10$$

$$\underline{\underline{FS_c = 2.10 > 1.0}} \quad \text{(satisfy corrosion)}$$

Conclusion:

The following design parameters have been determined:

Length of reinforcement, L	9.0 ft
Vertical spacing, S_z	1.5 ft
Horizontal spacing, S_x	6 in
Transverse spacing, S_y	9 in

Wire mesh w3.5 is used with nine layers.

Factors of safety against pullout (FS_p) ranged from 2.5 to 7.6 and were all above the minimum value of 1.5.

Factors of safety against rupture (FS_R) ranged from 1.1 to 4.3 and were all above the minimum value of 1.5.

Factors of safety against corruption (FS_c) was 2.01 and was above the minimum value of 1.0.

4E.3.2 Geotextile Wall

Step 5: Select reinforcement.

- Geotextile fabric will be used as reinforcements.

Step 6: Select facing.

- Geotextile fabric will be used as the facing and then covered with shotcrete.

Step 7: Complete internal stability analysis.

Step 7.1: Select failure surface and earth pressure coefficient.

- Technical information reviewed, such as NCHRPR 290 and *Reinforced Soil Structure, Volume 1*, recommend a more conservative approach and use K_o rather than K_A.

$$K_o = 1 - \sin\phi_r = 1 - \sin 34° = 0.441$$

$$K_o = 0.441$$

- The failure plane is assumed to be linear, extending from the wall toe to the top of the wall at an angle of 45 + $\phi/2$ from the horizontal. See figure 3-70.

Step 7.2: Select spacing measurement.

- Calculate the spacing requirements.

$$\text{vertical spacing, } S_z = \frac{F_a}{\sigma_x(FS_p)}$$

where F_a = allowable strength
FS_p = factor of safety for pullout
$\sigma_x = K_o(\gamma_r z + q)$

$$F_a = f(T_u)$$

where T_u value is obtained from manufacturer.

f value for different materials is listed in Step 7.5 in the design calculations.

From the 1993 specifier's guide, there are several geotextile fabrics that could be used, including:

Manufacturer	Product	Structure	Material	Strength (lb/in) T_u
Exxon	GTF 1000T	woven	polyester	1000
	GTF 1500T	woven	polyester	1500
Huesker	Comtrac 270.270	woven	polyester	1500
Nicolon	HP 1500	woven	polypropylene/ polyester	1350
	HP1300	woven	polyester	1300
Hoechst	Trevira 1135	nonwoven	polyester	152.9
	Trevira 1145	nonwoven	polyester	183.5
	Trevira 1155	nonwoven	polyester	205.5

From "Geotechnical Fabrics Report," Industrial Fabrics Association International 1993 *Specifier's Guide.*

From table in section 3.0, step 7.5, reduction factors (f) for polyester and polypropylene are 0.13 and 0.08, respectively.

Calculate F_a for a couple of different fabrics.

For GTF 1000T:

$$F_a = f(T_u) = 0.13(1000 \text{ lb/in})(12 \text{ in/ft}) = 1560 \text{ lb/ft}$$

For HP 1500:

$$F_a = 0.08(1350 \text{ lb/in})(12 \text{ in/ft}) = 1296 \text{ lb/ft}$$

For HP 1300:

$$F_a = 0.13(1300 \text{ lb/in})(12 \text{ in/ft}) = 2028 \text{ lb/ft}$$

For Trevira 1155:

$$F_a = 0.13(205.5 \text{ lb/in})(12 \text{ in/ft}) = 320.58 \text{ lb/ft}$$

For vertical spacing calculations (S_z):

$$S_z = \frac{F_a}{S_x(FS_p)} = \frac{F_a}{0.441\,(135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2)(1.5)}$$

For GTF 1000T:

$$S_z = \frac{1560 \text{ lb/ft}}{0.441\,(135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2)(1.5)}$$

For HP 1500:

$$S_z = \frac{1296 \text{ lb/ft}}{0.441\,(135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2)\,1.5}$$

For HP 1300:

$$S_z = \frac{2028 \text{ lb/ft}}{0.441\,(135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2)\,1.5}$$

For Trevira 1155:

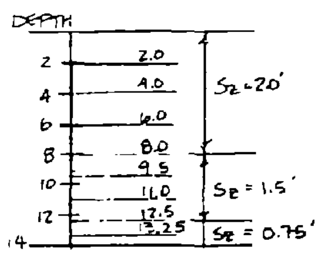
$$S_z = \frac{320.58 \text{ lb/ft}}{0.441\,(135 \text{ lb/ft}^3(z) + 250 \text{ lb/ft}^2)\,1.5}$$

See table below for results of S_z calculations.

Depth z (ft)	S_z (ft) GTF 1000T	S_z (ft) HP 1500	S_z (ft) HP 1300	S_z (ft) Trevira 1155	Design using HP 1300
2	4.5	3.8	5.9	0.93	
4	3.0	2.5	3.9	0.61	Sz = 2.0 ft.
6	2.2	1.8	2.9	0.46	MAX SPACING
8	1.8	1.5	2.3	0.36	@ 8.0 ft.
10	1.5	1.2	1.9	0.30	Sz = 1.5 ft.
12	1.3	1.0	1.6	0.26	@ 12.5 ft
14	1.1	0.9	1.4	0.23	Sz = 0.75 ft

The spacing requirements for the Trevira are prohibitive because they are so small. GTF 1500T and HP 1300 provide similar S_z results. HP 1500 requires smaller spacing than GTF 1000T and HP 1300, but larger than Trevira 1155. The remaining design calculations will be completed on HP 1300.

The selected vertical spacing is shown below for HP 1300.



Maximum spacing allowed is 2.0 feet. The vertical spacing selected must be less than the S_z calculated.

Step 7.3: Calculate the tensile stress in reinforcement.

- Calculate the tensile stress felt by the reinforcement.

$$\sigma_x = K_o(\gamma_{rz} + q) = 0.441(135\ \text{lb/ft}^3(z) + 250\ \text{lb/ft}^2)$$

$$\underline{\underline{\sigma_x = 0.441(135\ \text{lb/ft}^3(z) + 250\ \text{lb/ft}^2)}}$$

Calculate σ_x for each layer. See results in attached table on page 301.
(For layer 4, z = 8 ft.

$\sigma_x = 0.441(135\ \text{lb/ft}^3(8\ \text{ft}) + 250\ \text{lb/ft}^2) = 586.5\ \text{lb/ft}^2)$

Step 7.4: Check pullout of reinforcement

- Compute pullout resistance, P:

$$P = 2\tan\phi_{sr}(L_e)\sigma_z$$

where $\phi_{sr} = 2/3\phi_r$ for geotextile. (Based on recommendation provided in section 3.0, step 7.4.)

$$L_e = L - [\tan(45 - \phi_r/2)](H - z)$$

$$\sigma_z = \gamma_r z + q$$

Calculate L_e and σ_z for each layer:

$$L_e = L - (\tan 45° - \phi_r/2)(H - z)$$

$$= 9.0 - (\tan 45° - 34/2)(14 - z)$$

$$\underline{\underline{L_e = 9.0\text{ ft} - 0.5317(14\text{ ft} - z)}}$$

(For layer 4, z = 8 ft

L_e = 9.0 ft – 0.5317(14 – 8 ft) = 5.81 ft)

$$\sigma_z = \gamma_r(z) + q = 135\text{ lb/ft}^3(z) + 250\text{ lb/ft}^2$$

$$\underline{\underline{\sigma_z = 135\text{ lb/ft}^3(z) + 250\text{ lb/ft}^2}}$$

(For layer 4, z = 8 ft

σ_z = 135 lb/ft^3(8 ft) + 250 lb/ft^2 = 1330 lb/ft^2)

$$P = 2\tan 2/3(34°)L_e(\sigma_z)$$

$$\underline{\underline{P = 0.8353L_e(\sigma_z)}}$$

(For layer 4, L_e = 5.81 ft, σ_z = 1330 lb/ft^2

P = 0.8353(5.81 ft)(1330 lb/ft^2) = 6454 lb/ft)

See calculated results in attached table on page 301.

- Calculate tensile force, P_H, for each layer:

$$P_H = K_o(S_z)\sigma_z = K_o(S_z)(\gamma_r z_m + q)$$

where z_m = the depth to the middle of the layer

$$\underline{\underline{P_H = 0.441(S_z)(135 \text{ lb/ft}^3(z_m) + 250 \text{ lb/ft}^2)}}$$

Calculate P_H for each layer. See results in attached table on page 301.

(For layer 4, S_z = 2.0 ft, z_m = 7.0 ft

P_H = 0.441(2.0 ft)(135 lb/ft^3(7.0 ft + 250 lb/ft^2))

P_H = 1054 lb/ft)

- Calculate the factor of safety against pullout, FS_p, for each layer. FS_p should be greater than 1.5.

$$\underline{\underline{FS_p = P/P_H \geq 1.5}}$$

(For layer 4, P = 6454 lb/ft, P_H = 1054 lb/ft

FS_p = 6454 lb/ft/1054 lb/ft = 6.1)

See table for FS_p results. <u>All layers have FS_p > 1.5.</u>

<u>Step 7.5: Check rupture of reinforcement.</u>

- Calculate factor of safety against rupture, FS_R, for each layer.

$$FS_R = F_a/P_H \geq 1.0$$

$$\underline{\underline{FS_R = 2028 \text{ lb/ft}/P_H}}$$

See attached table on page 301 for results.

(For layer 4, P_H = 1054 lb/ft

FS_R = 2028 lb/ft/1054 lb/ft = 1.9 > 1.0)

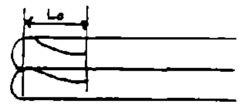
<u>All layers have $FS_R \geq 1.0$</u>

<u>Step 7.6: Evaluate corrosion rates for reinforcement.</u>

- For normal granular backfill soils, chemical degradation will not be a problem as long as the geotextile has been protected from UV radiation. The reduction factor used to calculate the allowable strength of the fabric accounts for degradation under normal conditions.

Step 7.7: Calculate wrap-around length.

- Calculate wrap-around length, L_o, for each layer.



$$L_o = \frac{\sigma_{xa} S_z (FS_w)}{2(\tan 2\phi/3) z_t \gamma_r} \geq 3.0 \text{ ft}$$

where FS_w = factor of safety = 1.5
Z_t = depth to top layer
σ_{xa} = average composite horizontal stress for the layer. Use the calculate stress at the middle of the layer at 2 m.
= $K_o(\gamma_r(z_m) + q)$

Layer	z (ft)	z_m (ft)	z_t (ft)	σ_x (lb/ft²)	σ_z (lb/ft²)	σ_{xa} (lb/ft²)	L_e (ft)	P (lb/ft)	P_H (lb/ft)	FS_p	FS_R	L_o (ft)
1	2.0	1.0	0.0	229.2	520	169.7	2.62	1138	339	3.4	6.0	∞→ 3.8
2	4.0	3.0	2.0	348.2	790	288.7	3.68	2430	577	4.2	3.5	3.8
3	6.0	5.0	4.0	467.3	1060	407.7	4.75	4202	815	5.2	2.5	2.7→ 3.0
4	8.0	7.0	6.0	586.5	1330	526.8	5.81	6454	1054	6.1	1.9	2.3→ 3.0
5	9.5	8.75	8.0	675.5	1532.5	630.9	6.61	8458	946	8.9	2.1	1.6→ 3.0
6	11.0	10.25	9.5	764.8	1735	720.2	7.40	10731	1080	9.9	1.9	1.5→ 3.0
7	12.5	11.75	11.0	854.1	1937.5	809.4	8.20	13274	1214	10.9	1.7	1.5→ 3.0
8	13.25	12.88	12.5	898.7	2038.8	876.7	8.60	14647	658	22.3	3.1	0.7→ 3.0
9	14.0	13.63	13.75	943.3	2140	921.3	9.00	16087	691	23.3	2.9	0.7→ 3.0
										≥ 1.5	≥ 1.0	≥ 3.0

4E3.3 Keystone Wall

Step 5: Select reinforcement.

- Use geogrid, TENSAR UX1500, or UX1600 depending on results of calculations.

- Start with 9-foot minimum length based on external stability calculations.

Step 6: Select facing.

- Use keystone standard blocks. Dimensions are 8 inches high by 18 inches wide at facing. Block is 22 inches deep.

Step 7: Complete internal stability analysis.

Step 7.1: Determine failure plane and earth pressure coefficient.

- Based on table 3-22, assume Rankine failure plane for geogrids.

- Based on table 3-22, use active case for earth pressure coefficient.

$$K_{Ar} = \tan^2(45 - \phi_r/2)$$

$$= \tan^2(45 - 34/2)$$

$$= 0.283$$

Step 7.2: Determine spacing of geogrids.

- Calculate maximum geogrid spacing. This is accomplished by calculating the maximum unreinforced height of wall allowable.

- Maximum unreinforced height will be governed by construction conditions where traffic will surcharge backfill.

- Treat unreinforced wall as a gravity wall. Check external failure modes of sliding and overturning. Bearing capacity should not be a concern for short wall heights.

- Try H = 2.67 ft (four blocks).
  Assume unit weight of filled block γ_k = 140 lb/ft^3.

- Check overturning.

$$FS_{OT} = \frac{3(L^2)(\gamma_k)(H)}{K)ar(H^2)[\gamma_r(H) + 3(q)]}$$

L = block width = 1.83ft
γ_r = 135lb/ft^3
q = 250lb/ft^2

$$FS_{OT} = \frac{(3)(1.83)^2(140)(2.67)}{(0.283)(2.67)^2[(135)(2.67) + (3)(250)]}$$

FS_{OT} = 1.67 < 2.00. Reduce H and recalculate FS_{OT}

EXAMPLE 3: KEYSTONE/TENSAR® WALL

- Try H = 20 ft (three blocks)

$$FS_{OT} = \frac{(3)(1.83)^2(140)(2.0)}{(0.283)(2.0)^2[(135)(2.0)+(3)(250)]}$$

$$FS_{OT} = 2.43 > 2.00 \qquad \text{OK}$$

- Check sliding:

$$FS_{SL} = \frac{(L)(\gamma_K)(H)(\tan\phi_r)}{(K_{ar})(H)[(q)+(0.5)(H)(\gamma_r)]}$$

$$= \frac{(1.83)(140)(2.0)(\tan 34°)}{(0.283)(2.0)[(250)+(0.5)(2.0)(135)]}$$

$$FS_{SL} = 1.59 > 1.5 \qquad \text{OK}$$

Use maximum geogrid spacing—2 feet (three blocks).

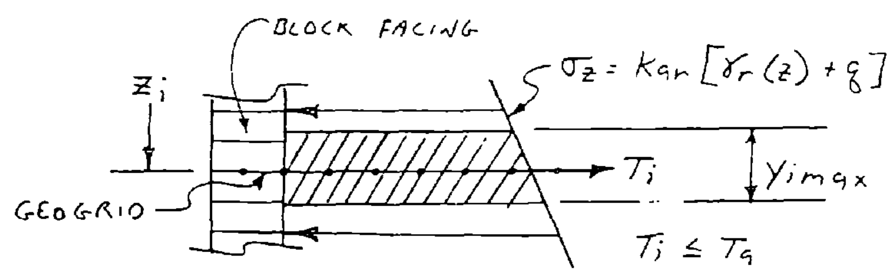
- Calculate vertical distance, y_{max}, the geogrids can support at depth z.

$$y_{\max} = \frac{T_a(P_c)}{K_{ar}[\gamma_r(z)+q]}$$

where T_a = allowable design strength of geogrid = $f(T_u)$

P_c = percent coverage of geogrid (= 1.0 for a layer that is continuous along the wall face)

- Sketch of y_{max}:



- T_u values obtained from *GFR Specifiers Guide* (also can be obtained from manufacturer). The Tensar® UX1600 and UX1500 geogrids are made from polyethylene, therefore f = 0.10 (see page 227).

Geogrid Type	T_u (lb/ft)	$T_a = 0.10\ T_u$ (lb/ft)
UX1500	5,892	589
UX1600	8,004	800

For UX1500:

$$Y_{imax} = \frac{(589\ \text{lb/ft})(1.0)}{0.283[(135\ \text{lb/ft}^3)(z) + 250\ \text{lb/ft}^2]}$$

For UX1600:

$$Y_{imax} = \frac{(800\ \text{lb/ft})(1.0)}{0.283[(135\ \text{lb/ft}^3)(z) + 250\ \text{lb/ft}^2]}$$

Block height is 8 inches. Calculate y_{imax} in feet and block increments.

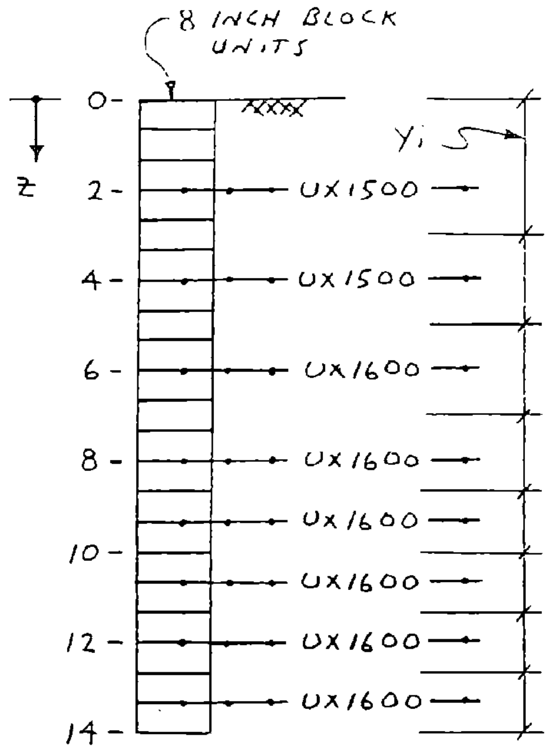
	UX1500		UX1600	
z (ft)	y_{imax} (ft)	No. Blocks	y_{imax} (ft)	No. Blocks
2	4.00	3*	5.44	3*
4	2.63	3	3.58	3*
6	1.96	2	2.67	3*
8	1.56	2	2.13	3
10	1.30	1	1.77	2
12	1.11	1	1.51	2
14	0.97	1	1.32	1

* Maximum spacing is three blocks previous calculations

- Place geogrids in wall working from the bottom of the wall. Use y_{imax} block spacings to place geogrids. First geogrid should be placed at a spacing of $y_{imax}/2$ or less. Other layers should be spaced at y_{imax} or less based on the above table.

Step 7.3: Calculate tensile stress in the geogrids

- Calculate geogrid tension in each layer based on actual elevation of geogrid. Set up figure shown on next page showing proposed geogrid elevations using above table.



y_i (ft)	z (ft)	T_i (lb/ft)	T_a (lb/ft)	
3.00	2.00	441	589	> T_i OK
2.00	4.00	447	589	OK
2.00	6.00	600	800	OK
1.67	8.00	628	800	OK
1.33	9.33	568	800	OK
1.33	10.67	636	800	OK
1.33	12.00	704	800	OK
1.33	13.33	771	800	OK

- Calculate T_i values based on actual y_i and z values.

$$T_i = K_{ar}[(\gamma_r)(z_i) + q]y_i$$

values shown in above table

(for layer 1 at z = 13.67 ft

$$T_i = (0.283)[(135 \text{ lb/ft}^2)(13.33 \text{ ft}) + (250 \text{ lb/ft}^2)](1.33\text{ft})$$

$$= 771\text{lb/ft}^2$$

$$T_i \le T_a \qquad 771 \le 800 \qquad \text{OK)}$$

- Check connection strength. For Tensar® geogrid with keystone block use pullout analysis at facing. Pullout strength, P_{Fi}, must be greater than or equal to T_i.

$$P_{Fi} = (2)(L)(\mu)(\gamma_K)(z_i)(\tan\phi_r)$$

$$L = 1.83 \text{ ft (width of block)}$$

$$\mu = 0.8 \text{ (friction factor assumed for geogrid/block interface)}$$

$$P_{Fi} = (2)(1.83)(0.8)(140)(\tan 34°)(z_i)$$

$$= 277(z_i) \text{ lb/ft}$$

Most critical location is at z = 2 feet

$$P_{Fi} = (277)(z) = 554 \text{ lb/ft}$$

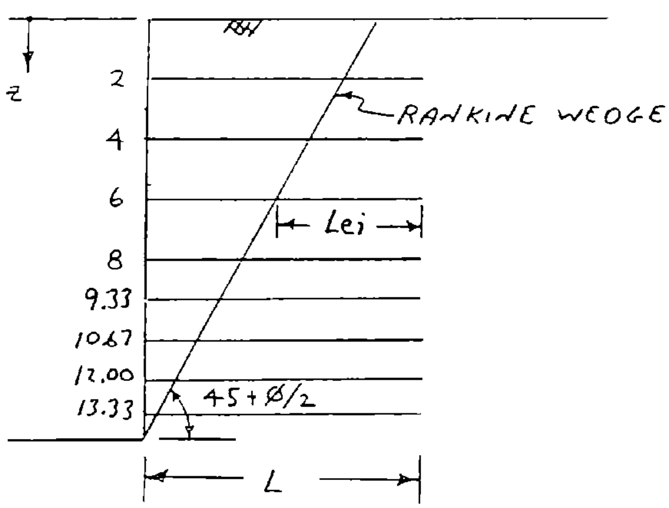
$$T_i = 441 \text{ lb/ft (from previous table)}$$

Facing Pullout Strength (P_{Fi})			
z (ft)	T_i (lb/ft)	P_{Fi} (lb/ft)	
2	441	554	> T_i OK
4	447	1108	OK
6	600	1662	OK
8	628	2216	OK
9.33	568	2584	OK
10.67	636	2955	OK
12.00	704	3324	OK
13.67	771	3787	OK

Step 7.4: Check pullout of geogrid.

- Calculate pullout capacity of length of geogrid behind assumed failure wedge. Pullout capacity should be greater than 1.5 T_i. If not, increase geogrid length. Minimum pullout length behind wedge is 3.0 feet.

- Pullout sketch:



L min = 9 FT FROM EXTERNAL CALCS.

- By inspection, uppermost geogrid will control 3-foot minimum L_e.

$$L_e = L - [(H - z)\tan(45 - \tfrac{\phi}{2})]$$

for z = 2 ft and L = 9 ft

$$L_e = 9 - [(14-2)\tan(45-34/2)] = 2.61 < 3.00$$

increase L

Increase L by 0.5 feet to 9.5 feet and recalculate L_e:

$$L_e = 9.5 - [(14-2)\tan(45-34/2)] = 3.12$$

$$3.12 > 3 \quad OK$$

- Calculate FS_{po} for all layers; use new L of 9.5 feet:

$$FS_{po} = \frac{(2)(L_{ei})(\gamma_r)(z_i)(\tan\phi_{sr})}{T_i}$$

where L_{ei} = 9.5 - [(14 - z_i)tan(45 - 34/2)]

$$\phi_{sr} = 2/3\phi_r = 22.7°$$

$$FS_{po} = \frac{(2)(L_{ei})(\gamma_r)(z_i)(\tan 22.7°)}{T_i}$$

$$= \frac{(L_{ei})(z_i)(113)}{T_i}$$

z (ft)	L_{ei} (ft)	T_i (lb/ft)	FS_{po}	
2	3.12	441	1.60	>1.5 OK
4	4.18	447	4.23	OK
6	5.24	600	5.92	OK
8	6.31	628	9.08	OK
9.33	7.02	568	13.03	OK
10.67	7.73	636	14.65	OK
12.00	8.44	704	16.26	OK
13.67	9.32	771	18.67	OK

Step 7.5: Check overstressing of geogrids.

- T_i must be less than T_a for all layers. This was confirmed in step 7.3.

Step 7.6: Evaluate corrosion rates.

- Corrosion rates not required for polymeric materials. Other environmental factors such as durability and construction damage under normal conditions have been accounted for in the reduction factor, f.

Conclusion:

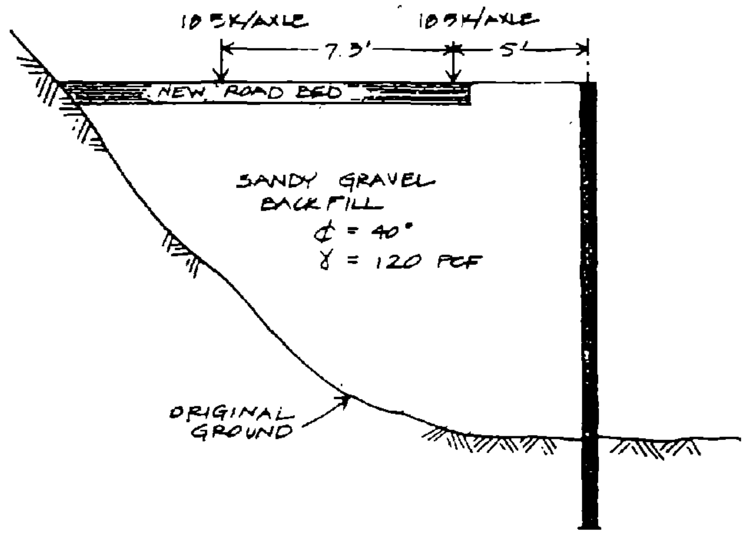
- 8 layers of geogrid
- Length = 9.5 feet
- Geogrid layout

Type	Depth (ft)	Length (ft)
UX1500	2.00	9.5
UX1500	4.00	9.5
UX1600	6.00	9.5
UX1600	8.00	9.5
UX1600	9.33	9.5
UX1600	10.67	9.5
UX1600	12.00	9.5
UX1600	13.33	9.5

4F Cantilever Sheet Pile Wall

4F.1 Problem Statement

A low volume road is to be designed for a 160 k off-highway load. A 12-foot-high wall will be required. At a minimum, trucks will maintain a distance 5 feet from top of the wall backfill. Existing ground consists of clean, well-graded angular sandy fine gravel (phi = 40 degrees, weight = 140 pcf). No water table is present. The wall used is to be driven cantilever sheet pile.



4F.2 Design Calculations

Use log spiral earth pressure coefficients, assuming wall friction angle $\delta = \phi/2 = -20°$ (log – spiral sign convention).

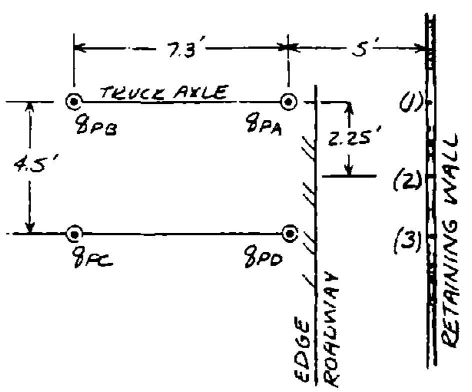
For steel, or see table 3-4:
From figure 3-17, for $\beta/\phi = 0$, $K_A = 0.22$, and for $\delta/\phi = -0.5$, $K_p = 18(0.592) = 10.6$.

Say, for stability concerns, use $F.S. = 1.5$ on K_p; hence use $K_p = 10.6/1.5 = 7.1$. Alternatively, K_p could be left unreduced and the calculated embedment increased by 20 percent to 40 percent.

Earth Pressure Diagram:

Compute horizontal components of traffic loads and location of their resultants (refer to chapter 3D.5.1, figures 3-24.b, c, d). Consider each dual set as a point load of intensity 18.5 kips. See plan view below.

$$q_{PA} = q_{PB} = q_{PC} = q_{PD} = 18.5 \text{ kips}$$



Design wall to withstand the highest of horizontal loads at points (1), (2), and (3).

From symmetry,

$$P_{h_1} = P_{h_3}.$$

This neglects axle groups ahead of or behind the one considered.

For computation points (1) and (3),

$$M_A = M_D = \frac{x}{H} = \frac{5}{12} = 0.42$$

$$M_C = M_B = \frac{x}{H} = \frac{12.73}{12} = 1.02$$

From figure 3-24(b),

$$P_{hA,D}\left(\frac{H}{q_P}\right) = 0.73 \text{ at } y = 0 \text{ feet}$$

$$P_{hB,C}\left(\frac{H}{q_P}\right) = 0.18 \text{ at } y = 0 \text{ feet}$$

Force reduction factors for q_{PC} and q_{PD} at (1) are from figure 3-24(c):

$$P_C = \frac{y}{x} = \frac{4.5}{5 + 7.3} = 0.37 \rightarrow I_{\phi C} = 0.86$$

$$P_D = \frac{y}{x} = \frac{4.5}{5.0} = 0.90 \rightarrow I_{\phi D} = 0.50$$

Hence, horizontal forces at (1) and (3) are:

$$P_{hA_{1,3}} = 0.73\left(\frac{q_P}{H}\right) = 0.73\left(\frac{18.5}{12}\right) = 1.13 \text{ kips}$$

$$P_{hB_{1,3}} = 0.18\left(\frac{q_P}{H}\right) = 0.18\left(\frac{18.5}{12}\right) = 0.28 \text{ kips}$$

$$P_{hC_{1,3}} = 0.18\left(\frac{q_P}{H}\right)I_{\phi C} = 0.18\left(\frac{18.5}{12}\right)(0.86) = 0.24 \text{ kips}$$

$$P_{hD} = 0.73\left(\frac{q_P}{H}\right)I_{\phi D} = 0.73\left(\frac{18.5}{12}\right)(0.50) = 0.56 \text{ kips}$$

Location of resultants of force components and of total force.

$$R_{Al} = R_{Dl} = 0.57H = 0.57(12) = 6.8 \text{ ft} \quad \text{for } M_A = M_D \text{ (figure 3-24.c)}$$

$$R_{Bl} = R_{Cl} = 0.38H = 0.38(12) = 4.6 \text{ ft} \quad \text{for } M_B = M_C \text{ (figure 3-24.c)}$$

$$R_1 = \frac{P_{hA}R_A + P_{hB}R_B + P_{hC}R_C + P_{hD}R_D}{P_{hA} + P_{hB} + P_{hC} + P_{hD}}$$

$$= \frac{1.13(6.8) + 0.28(4.6) + 0.24(4.6) + 0.56(6.8)}{1.13 + 0.28 + 0.24 + 0.56}$$

R_1 = 6.3 feet above ground line

where $P_{1,3} = P_{hA} + P_{hB} + P_{hC} + P_{hD} = 2.21$ kips

For computation point (2), only I_ϕ values change since y changes.

$$P_{A,D} = \frac{y}{x} = \frac{2.25}{5} = 0.45 \rightarrow I_{\phi A} = 0.77$$

$$P_{C,B} = \frac{y}{x} = \frac{2.25}{12.3} = 0.18 \rightarrow I_{\phi A} = 0.97$$

$$P_{hA_2} = 0.73\left(\frac{18.5}{12}\right)(0.77) = 0.87 \text{ kips}$$

$$P_{hB_2} = 0.18\left(\frac{18.5}{12}\right)(0.97) = 0.27 \text{ kips}$$

$$P_{hC_2} = 0.18\left(\frac{18.5}{12}\right)(0.97) = 0.27 \text{ kips}$$

$$P_{hD_2} = 0.73\left(\frac{18.5}{12}\right)(0.77) = 0.82 \text{ kips}$$

Resultant location:

$$R_2 = \frac{0.87(6.8) + 0.27(4.6) + 0.27(4.6) + 0.87(6.8)}{0.87 + 0.27 + 0.27 + 0.87}$$

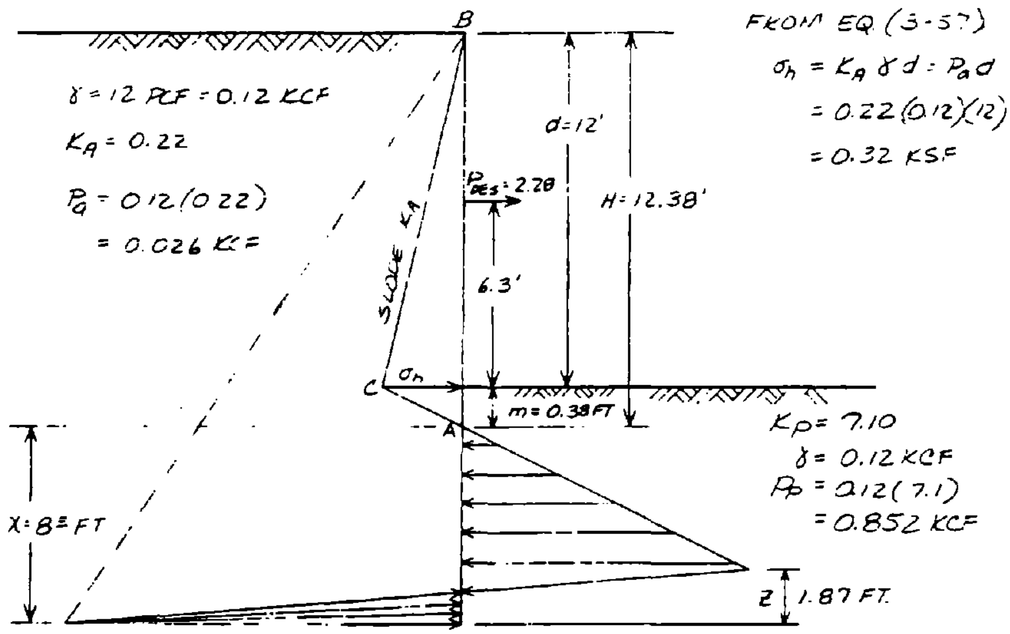
$$= 6.3 \text{ ft}$$

where P_2 = 2.28 kips.

Design traffic surcharge load (lateral component):

P_{DB3} = 2.28 kips at 6.3' above

Note: In lieu of calculating the above, use table 3-9.



From equation 3-65,

$$\frac{m+d}{m} = \frac{P_p}{P_a} = \frac{K_p\gamma}{K_a\gamma} = \frac{K_p}{K_a}$$

$$= \frac{7.1}{0.22} = 32.3$$

$$\text{or } d = 31.3 \text{ m} = 12 \text{ ft}$$

$$\text{and } m = \frac{12}{31.3} = 0.38 \text{ ft}$$

From equation 3-66, resultant earth pressure above A is:

$$P_{h_E} = \frac{P_q d}{2}(d+m) = \frac{0.22(0.12)(12)}{2}(12+0.38)$$

$$= 1.96 \text{ kips.}$$

Total resultant horizontal force includes:

$$P_h = P_{h_E} + P_{DES} = 1.96 + 2.28 = 4.24 \text{ kips}$$

and is located above point A at a distance h

$$h = \frac{P_{nE}h_E + P_{nT}h_T}{P_h}$$

Using the third point rule for $P_{nE}h_E$,

$$h = \frac{[1/2(12)(12/3 + 0.38) + 1/2(0.38)^2(2/3)](0.32) + 2.28(6.3 + 0.38)}{4.24}$$

$$h = 5.58 \text{ ft}$$

Compute depth of penetration, x, by equation 3-67:

$$6P_h(h+x) - x^3(P_p - P_a) + \frac{[(P_p - P_a)x^2 - 2P_h]^2}{2(P_p - P_a)x + H_{P_p} - MP_a} = 0$$

$$6(4.24)(5.58 + x) - x^3(0.852 - 0.026) + \frac{[(0.852 - 0.026)x^2 - 2(4.24)]^2}{2(0.852 - 0.026)x + 12.38(0.852) - 0.38(0.026)} = 0$$

$$25.44x \neq 141.95 - 0.826x^3 + \frac{[0.826x^2 - 8.48]^2}{1.652x + 10.538} = 0$$

Solution by trial and error gives $x = 8.0$ ft.

Complete loading diagram—compute z with equation 3-68:

$$z = \frac{(P_p - P_a)x^2 - 2P_h}{2(P_p - P_a)x + HP_p - MP_a}$$

$$= \frac{(0.852 - 0.026)8^2 - 2(4.24)}{2(0.852 - 0.026)(8) + 12.38(0.852) - 0.38(0.026)}$$

$$= 1.87 \text{ ft}$$

Compute point of zero shear, b, by equation 3-69:

$$b = \sqrt{\frac{2P_h}{P_p - P_a}} = \sqrt{\frac{2(4.24)}{0.826}}$$

$$= 3.20 \text{ ft} \quad (\text{below } A)$$

Compute maximum moment per equation 3-70:

$$M_{max} = P_h(h+b) - \frac{b^3}{6}(P_p - P_a)$$

$$= 4.24(5.58+3.20) - \frac{3.20^3}{6}(0.852-0.026)$$

$$= 32.72 \text{ kip-feet/ft}$$

Minimum section modulus by equation 3-71:

$$S_{min} = \frac{32.72(12)}{0.6(36)} = 18.18 \text{ in}^3\text{/ft}$$

$$\therefore \text{ use PZ27, } S = 30.2 \text{ in}^3\text{/ft}$$

4F.3 Additional Design Considerations

4F.3.1 Pile Length

Sheet piles are supplied by most mills in lengths that range from 30 to 65 feet. Orders for shorter or longer lengths may require a long lead time. In addition, requirements for most low volume road walls will not meet the minimum required roll lot quantity of 300 to 400 tons. Hence, for the design pile length of $\pm$ 20 feet, the engineer must do one of the following:

(1) Locate the proper length (either new or used).

(2) Drive some additional length of embedment.

(3) Cut over-length piles and splice leftover pieces.

The economics of the latter two alternatives must be carefully evaluated.

In alternative 3, where splicing is necessary, it should be accomplished by butt-welding or by fishplating (i.e., welding plates to the pile webs at a joint) the two pieces. Interlocks cannot be readily welded without interfering with threading operations; hence, if the full strength of the shape is required, fishplating is recommended over butt-welding. When using spliced sections, it is recommended that the splices be staggered in elevation. In addition, it is recommended (if possible) to alternate spliced pieces and whole pieces or stagger splices on alternate pieces. Further, it is desirable to have the splice as far from the point of maximum bending moment as possible.

4F.3.2 Pile Driving

Damage to the sheet pile due to overdriving generally occurs as a plastic deformation of the top of the pile or by the tearing of the interlock at the base of the pile and subsequent bending of the pile tip. Both of these problems can be eliminated or grossly reduced by proper selection of the pile hammer and cushioning. In addition to proper hammer selection, the tips of the piles can be fitted with protective driving shoes to reduce tip damage and driving friction. Detailed discussions of the above are included in most textbooks on pile foundations and in the pile-tips how-to book, *Design and Installation of Pile Foundations* by Hal Hunt.

A final comment regarding pile driving: if excessive hard driving is anticipated, it might be prudent to allow some additional pile length to permit damaged pile tops to be removed, thus improving the appearance of the completed project.

4F.3.3 Drainage

For the conditions presented, drainage should not be a problem since the backfill can drain freely in a vertical direction. However, if the foundation soils are impermeable and the sheet piles form a complete enclosure, then there should be provisions for drainage of the backfill.

4F.4 Construction Considerations

The major construction considerations involve the availability of the design piles and the required equipment, and the need for an experienced contractor to successfully install the sheet piles. On a job where difficult driving is anticipated, there is no substitute for the proper equipment and an experienced contractor.

Inspection of the pile-driving operation is mandatory in order to verify the penetration depth of the pile and the proper alignment of the driven pile. Piles driven in excess of 1/4H to 12V out of alignment should be rejected. Pulling piles into alignment generally is not an acceptable practice due to the resulting overstressing of the steel. Under no circumstances should piles be heated to aid in straightening a bent pile or one that is out of alignment.

Special equipment requirements for driving sheet piling include a crane, the recommended pile driver, and the proper driving cap for the design sheets. Although not completely necessary, a driving template is also recommended to provide support for the pile while it is being driven. The latter is particularly important if the piles are battered into the hill to compensate for deflections after backfilling.

Backfilling of sheet pile walls may be initiated as soon as a significant length of wall has been constructed. Generally, all backfill should be clean and free-draining, unless it is approved by the design engineer.

All backfill should be placed in horizontal lifts with the following thicknesses:

Material type	Uncompacted lift thickness (inches)
Clean sands and gravels	12
Silts, clays, and silt clay mixtures	9

Subsequently, the loose backfill should be mechanically compacted to 92 to 93 percent of the maximum dry density obtained by AASHTO T-99. For moisture sensitive soils, the water content should be controlled in the range of ±2 percent of optimum as determined by AASHTO T-99.

Compaction within 5 feet of the back of the wall shall be accomplished with lightweight, hand-operated compaction equipment.

4G Anchored Soldier Pile Wall—Timber Lagging

4G.1 Problem Statement

A road is being constructed to a logging area on a mountaintop. A portion of the proposed road traverses a steep (1.25H:1V) talus slope with a proposed grade raise of 10 feet. The bedrock face at the location of the proposed roadway is sound, uniform basalt (q_u = 10,000 psi). The face is located only 30 feet from the outer edge of the proposed roadway.

Observations of the slope lead to the conclusion that it is currently lying at its angle of repose. Test pits show that the material is relatively free-draining, angular basalt with some sand-size particles.

A retaining structure is required for the grade raise. Economic studies and previous experiences show that an anchored H-pile wall with timber lagging will be desirable from the owner's point of view.

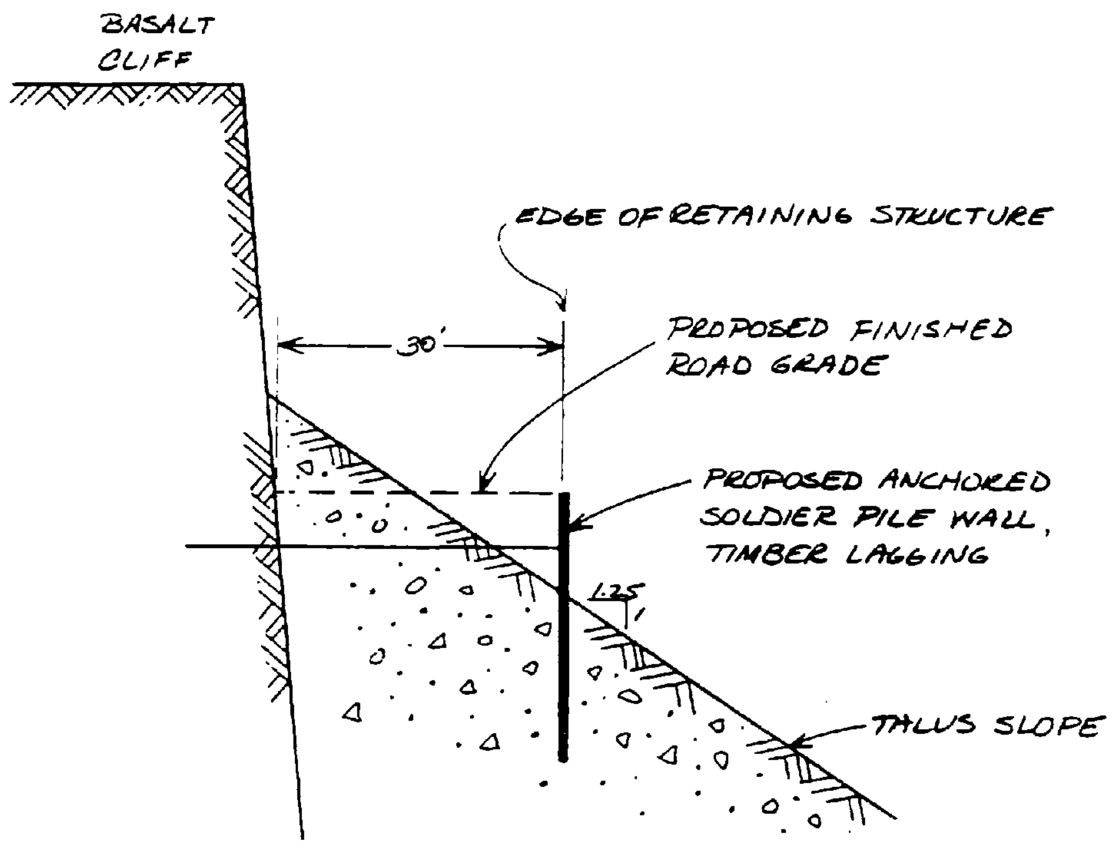
See the cross-section of the proposed wall and geologic configuration.

Off-highway logging trucks will be using the new roadway. They will be kept at a minimum of 2 feet from the retaining structure by the guard-rail to be installed in the project.

Backfill will be the material excavated from the talus slope above the proposed finished grade.

A geological study of the joint system in the basalt face leads to the recommendation that if rock anchors are to be used, they should be drilled into the face at an elevation 6 feet below the finished grade.

The equations and stress diagrams relating to anchored walls presented in chapter 3 pertain to the special case where the anchor is located at, or very near, the top of the pile. For the stated problem, the anchor will be located some 6 feet from the top of the wall, hence a general solution is required. The generalized equations and free body diagrams are presented before each case that was evaluated (the minimum embedment, case 1, and embedment to some greater depth, case 2).



Geologic section and cross-section of proposed retaining structure

Scale 1"–20'

4G.2 Design Calculations

The angle of internal friction of the talus material may be taken to equal the angle of repose of the talus material, i.e., ϕ = 38.6° .

Use of the log spiral earth pressure coefficients will be made from the curves of Caquot and Kerisel, figure 3-17:

$$K_A\text{:}\ \ \beta = 0°,\ \ \beta/\phi = 0°,\ \ \delta = 0°,\ \ \phi = 38.6°$$

$$K_A = 0.23$$

$$K_P\text{:}\ \ \beta = 38.6°,\ \ \beta/\phi = 1.0°,\ \ \delta/\phi = -0.5°,\ \ \phi = 38.6°$$

$$K_P = 0.79(0.617) = 0.49$$

An estimate of the unit weight of the material may be made by reference to figure 3-1. Select γ = 130 pcf for material between gp and gw. Moisture content will be included therein.

Computation of the horizontal load by the 160 K gross (37 K dual-axle load) off-highway log truck may be estimated by reference to table 3-9, by ratio of the axle load of the off-highway truck to the legal truck. An estimate of the total wall height must be made for the calculation:

Estimate H = 20 ft x = set back = 2 ft

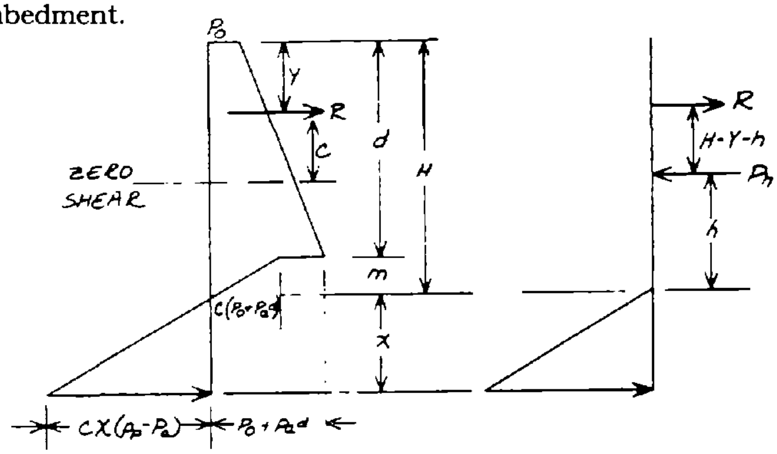
$$P_{h_T} = 0.84\left(\frac{37}{16.5}\right) = 1.88 \text{ kips}$$

where 0.84 is from table 3-9 for x = 5 ft

The discussion of section 3D.5.1 indicates the resultant should be located at about 0.58H, or 11.6 feet above the tip of the wall, or 8.4 feet from top.

Assume that the spacing of soldier piles is 8 feet and that they will be H14x?

General case 1: Anchored soldier. Pile wall—minimum pile embedment.



$$\Sigma M_R = 0]\ 0 = P_h(H-Y-h) - 1/2Cx^2(P_p - P_a)[Hx^2 - Yx^2 + 2/3x^3] = 0$$

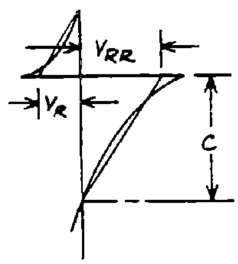
$$\text{or } P_h(H-Y-h) - 1/2C(P_p - P_a)[Hx^2 - Yx^2 + 2/3x^3] = 0$$

$$\text{or } 6P_h(H-Y-h) - C(P_p - P_a)[3Hx^2 - 3Yx^2 + 2x^3] = 0$$

So, general equation for x—solution by trial:

$$6P_h(H-Y-h) - C(P_p - P_a)[2x^3 + 3Hx^2 - 3Yx^2] = 0$$

Locate point of zero shear below anchor level:



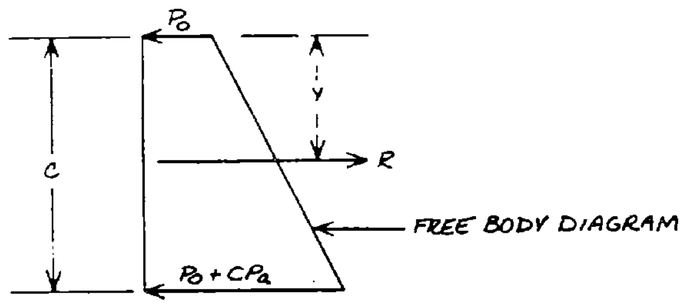
$$V_R = P_oY^2 + 1/2P_aY = (P_oY + 1/2P_a)Y$$

$$V_{RR} = R - V_R = R - (P_oY + 1/2P_a)Y$$

$$0 = V_{RR} - 1/2C^2P_a - CP_o$$

$$= R - (P_o + 1/2P_a)Y - 1/2C^2P_a - CP_o$$

General equation for point of zero shear below top of wall



$$\text{For zero shear } \Sigma F = 0]\ P_oC + 1/2C^2P_a - R = 0$$

$$P_aC^2 + 2Pf - 2R = 0$$

$$C = \frac{-2P_o \pm \sqrt{(2P_o)^2 + 4P_a(2R)}}{2P_a}$$

$$C = \frac{-2P_o \pm \sqrt{4P_o^2 + 8R_{P_a}}}{2P_a}$$

$$\text{for } C \le d$$

Calculate the earth pressure diagram—Case 1 (Minimum embedment case)

$P_a = 0.23(0.130) = 0.030$ KCF

$P_P = 0.49(0.130) = 0.064$ KCF

Need to resolve P_o to horizontal (see figure 3-17)

$$P_{P_h} = P_P \cos\delta = P_P \cos\frac{38.6°}{2}$$

$$= 0.064 \cos 19.3°$$

$$= 0.60 \text{ KCF}$$

$\sigma_{h_1} = P_{ad} = 0.030(10) = 0.30$ KSF

$\sigma_{h_2} = C\sigma_{hi}$ where

C = ratio of effective pile width to ℄ spacing of soldier piles.

Refer to table 3-20.

Select $W_e = 4B$

$$= 4\left(\frac{14}{12}\right) = 4.67$$

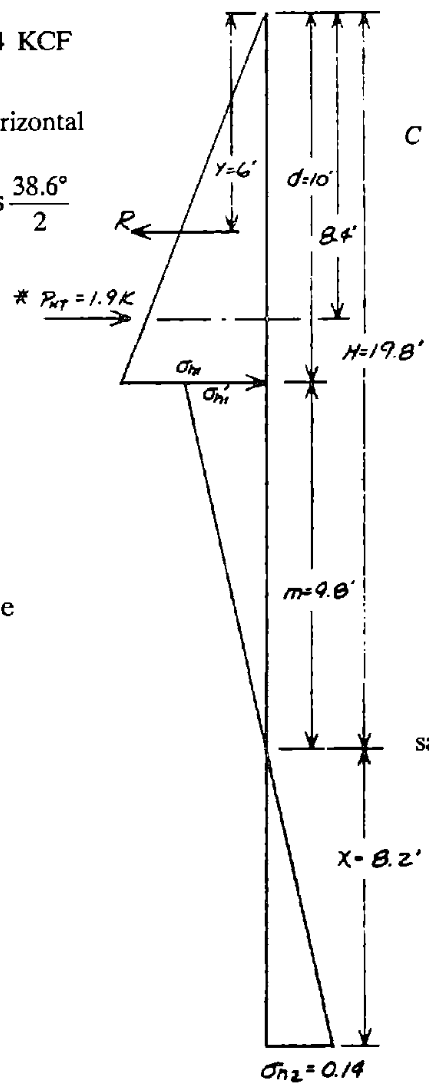
$$\therefore C = \frac{4.67}{8.0} = 0.58$$

$\sigma_{h_1'} = 0.58(0.30) = 0.17$ KSF

$$m = \frac{\sigma_{h_1'}}{C(P_{P_h} - P_a)} = \frac{0.17}{(0.58)(0.06-0.03)}$$

$$= 9.77 \text{ ft}$$

say $m = 9.8$ ft



* for conservatism, assume P_{hT} acts uniformly over 1 foot length of wall, i.e., P_{HT} = 1.9 k/ft

Calculate minimum x assuming the anchor level to be at y = 6 feet below top of the wall.

First find total horizontal resultant on the active side of the retaining wall, and dimension h (refer to figure 3-54).

$$P_h = P_{HT} + 1/2\sigma_{hl}d + 1/2\sigma'_{hl}m$$

$$= 1.9 + 1/2(0.30)(10) + 1/2(0.17)(9.8)$$

$$= 1.9 + 1.5 + 0.8$$

$$= 4.2 \text{ k/ft}$$

$$\text{and } h = \frac{P_{HT}[H - Y] + 1/2\sigma_{hl}d[m + d/3] + 1/2\sigma'_{hl}m[2/3m]}{P_h}$$

$$h = \frac{1.9[19.8 - 8.4] + 1.5[9.8 + 10/3] + 0.8[2/3(9.8)]}{4.2}$$

$$h = 11.1 \text{ ft}$$

Now solve equation 3-99 by trial and error for x (summation of moments about R).

$$6P_h(H - Y - h) - C(P_P - P_a)(3x^2H + 2x^3 - 3Yx^2) = 0$$

Substituting values gives:

$$6(4.2)(19.8 - 6 - 11.1) - 0.58(0.06 - 0.03)[3(19.8)x^2 + 2x^3 - 3(6)x^2] = 0$$

Simplifying,

$$68.04 - 1.03x^2 - 0.035x^3 + 0.31x^2 = 0$$

$$68.04 - 0.72x^2 - 0.035x^3 = 0$$

Solution by trial gives $x = 8.2$ feet.

Completing case 1 earth pressure diagram

$$\sigma_{h2} = 8.2(P_P - P_a)C$$

$$= 8.2(0.030)(0.58)$$

$$= 0.14 \text{ KSF}$$

Compute the tension in the anchor by summation of horizontal forces. See equation 3-100.

$$R = P_h - 1/2C(P_p - P_a)x^2$$

$$= 4.2 - 1/2(0.58)(0.030)(8.2)^2$$

$$= 3.63 \text{ kips/ft}$$

The point of zero shear is found from equation 3-101. For C above bottom exposed wall height, d.

$$C = \sqrt{\frac{2R}{P_a}}$$

$$= \sqrt{\frac{2(3.6)}{0.03}} = 15.5 \text{ ft} > d = 10 \text{ ft}$$

Since equation 3-101 is based on $0 < C \leq d$, the result is not valid. The point of zero shear is below depth d due to the shape of the earth pressure diagram. Therefore, the special case must be reanalyzed by computation of the shear at depth d and then computing the depth C' below that point, at which shear goes to zero.

Shear at depth d is:

$$Y_d = 1/2\sigma P_{hl}d + P_{HT} - R$$

$$= 1/2(0.30)(10) + 1.9 - 3.62$$

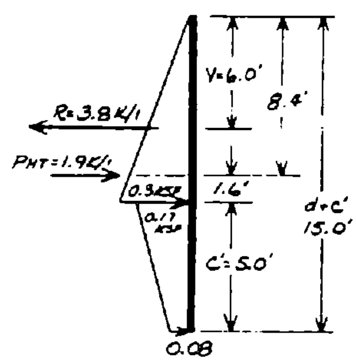
$$= -0.22 \text{ k/ft}$$

Now depth to zero shear C' below depth d:

$$C' = \sqrt{\frac{2|V_d|}{C(P_p - B)}} = \sqrt{\frac{2(0.22)}{(0.03)(0.58)}} = 5.03 \text{ ft, say } 5.0 \text{ ft}$$

Compute moment at depth $d + C' = 10 + 5.0 = 15.0$ feet.

The free body diagram for moment computation is as shown below:



where:

$$\sigma_C = 0.17 - 5.0(0.06 - 0.03)(0.58)$$

$$= 0.08 \text{ KSF}$$

$$\overset{\curvearrowleft +}{M_C} = 3.8[15-6] - 1.9[15-8.4] - 1/2(0.3)(10)[5+10/3]1/2(5)(0.17+0.08)\left[5/3\left(\frac{2(0.17)+0.08}{0.17+0.08}\right)\right]$$

$$= 3.6[9] - 1.9[6.6] - 1.5[8.33] - 0.63[2.80]$$

$$M_C = 5.6 \text{ kip} \cdot \text{feet/ft}$$

Check moment at the level of the anchor

$$\overset{\curvearrowright +}{M_R} = 1/2 P_a Y^2 [Y/3]$$

$$= 1/2(0.03)(6)^3(1/3)$$

$$= 1.08 \text{ kip} \cdot \text{feet/ft}$$

Minimum section modulus required for case 1 design may be computed by equation 3-82 where $M_{max} = M_C = 7.4$ k-ft/ft.

$$S_{min} = \frac{M_{max}}{F_b} S = \frac{5.6(12)}{0.6(36)} 8$$

$$\therefore S_{min} = 24.9 \text{ in}^3$$

For $S_{min} = 24.9$ in^3 an H10 x 42 would be adequate, but for an 8-foot spacing, the ratio of effective width to spacing would be reduced from the originally assumed $C = 0.58$ to a value of

$$C = \frac{4(10/12)}{8} = 0.417 \qquad \text{say } 0.42$$

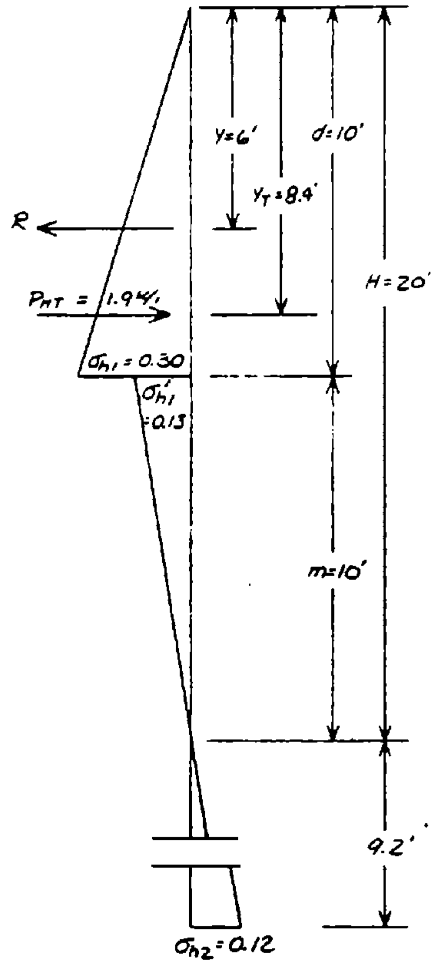
This 30 percent reduction is too great to ignore, hence another iteration must be done to properly evaluate the problem.

A decrease in the value of C will cause a greater embedment requirement, and will also alter the design moment. As the next approximation, an H10 x ? will be tried. So,

$$C = \frac{4(10/12)}{8} = 0.42$$

See next page for revised earth pressure diagram.

Revised Earth Pressure Diagram:



$$\sigma_{hl} = 0.30 \text{ KSF}$$

$$\sigma'_{hl} = C\sigma_{hl} = 0.42(0.3) = 0.13 \text{ KSF}$$

$$m = \frac{\sigma'_{hl}}{C(P_p - P_a)} = \frac{0.13}{0.42(0.03)} = 10.0 \text{ ft}$$

Resultant P_h:

$$P_h = P_{HT} + 1/2\sigma_{hl}d + 1/2\sigma'_{hl}m$$

$$= 1.9 + 1/2(0.3)(10) = 1/2 + 1/2(0.13)(10)$$

$$= 1.9 + 1.5 + 0.7$$

$$= 4.1 \text{ kips}$$

$$h = \frac{P_{HT}[H - Y_T] + 1/2\sigma_{hl}d[m + d/3] + 1/2\sigma'_{hl}m[2m/3]}{P_h}$$

$$= \frac{1.9[20 - 8.4] + 1.5[10 + 10/3] + 0.7[2(10)/3]}{4.1}$$

$$= \frac{22.0 + 20.0 + 4.67}{4.1}$$

$$h = 11.4 \text{ ft}$$

Minimum x by equation 3-99:

$$6(4.1)(20 - 6 - 11.4) - 0.42(0.03)[3(20)x^2 + 2x^3 - 3(6)x^2] = 0$$

$$63.96 - 0.76x^2 - 0.025x^3 + 0.23x^2 = 0$$

$$63.96 - 0.53x^2 - 0.025x^3 = 0$$

Solution by trial gives $x = 9.2$ feet.

$$\sigma_{h2} = xC(P_p - P_a) = 9.2(0.42)(0.03) = 0.12$$

Compute R by equation 3-100:

$$R = 4.1 - 1/2(0.42)(0.03)(9.2)^2$$

$$= 3.57 \text{ kips/ft, say } 3.64 \text{ k/ft}$$

Locate zero shear—below depth d:

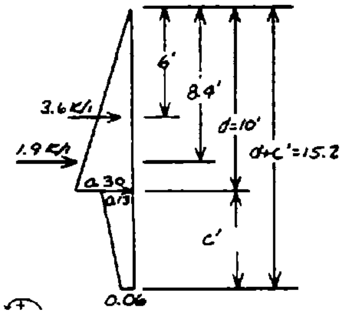
$$V_d = 1/2\sigma_{hl}d^2 + P_{HT} - R$$

$$= 1.5 + 1.9 - 3.57$$

$$= 0.17 \text{ k/ft}$$

$$C' = \sqrt{\frac{2(0.17)}{0.03(0.42)}} = 5.19 \text{ ft , say } 5.2 \text{ ft}$$

Moment at $d + C' = 10 + 5.2 = 15.2$.



$$\sigma_C = 0.13 - 5.2(0.03)(0.42)$$

$$= 0.06 \text{ KSF}$$

$$M_C = 3.6[15.2-6] - 1.9[15.2-8.4] - 1/2(0.3)(10)[5.2+10/3]$$
$$- 1/2(0.13+0.06)(5.2)\left[\frac{5.2}{3}\left(\frac{2(0.13)+0.06}{0.13+0.06}\right)\right]$$

$$= 3.6[9.2] - 1.9[6.8] - 1.5[8.5] - 0.49[2.92]$$

$$= 33.1 - 12.9 - 12.8 - 1.4$$

$$= 6.0 \text{ kip-feet/ft}$$

Section required

$$S_{\min} = \frac{M_{\max}}{F_b} S$$

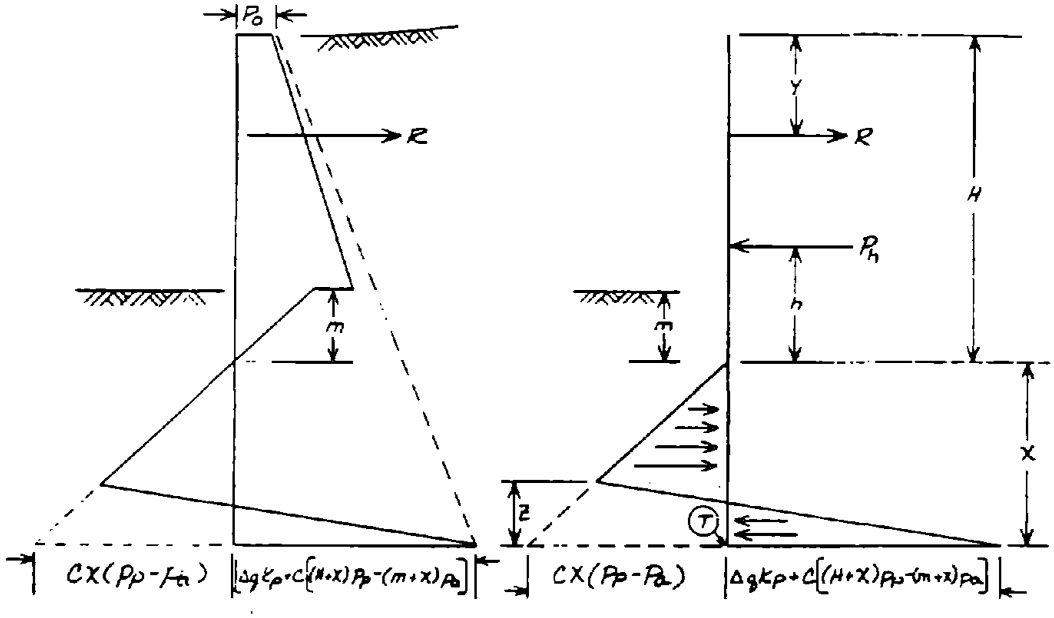
$$= \frac{6.0(12)(8)}{0.6(36)}$$

$$= 26.7 \text{ in}^3$$

$\therefore$ use H10 x 42 if case 2 checks out OK.

$S_{xx} = 43.4 \text{ in}^3$ for H10 x 42.

b. General Case 2: Anchored Soldier Pile Wall



Solve for z by summation of forces:

$$\Sigma \vec{F}_H^{-} = 0]\ 2R - 2P_n + x[Cx(P_p - P_a)]$$

$$- z[Cx(P_p - P_a) + P_o + C((H+x)P_p - (m+x)P_a)] = 0$$

$$\therefore z = \frac{2R - 2P_n + Cx^2(P_p - P_a)}{\Delta q K_p + 2Cx(P_p - P_a) + CHP_p - CmP_a}$$

Solve for x by summation of moments about tip of wall, point T:

$$\Sigma M_T = 0]\ P_n(x+n) - R(H+x-y) - 1/2 \cdot 1/3 x^2 Cx(P_p - P_a)$$

$$+ 1/2 \cdot 1/3 z^2 [Cx(P_p - P_a) + \Delta q Kp + C[(H+x)P_p - (m+x)P_a]] = 0$$

$$0 = P_n(x+h) - 6R(H+x-y) - x^3 C(P_p - P_a) + z^2[\Delta q Kp + 2Cx(P_p - P_a) + CH_{P_p} - m_{P_a} C]$$

Substitute for z:

$$6P_n(x+h) - 6R(H+x-y) - x^3 C(P_p - P_a) + \frac{[2R - 2P_n + Cx^2(P_p - P_a)]^2}{[\Delta q K_p + 2Cx(P_p - P_a) + CH_{P_p} - Cm_{P_a}]} = 0$$

For factor of safety, add 30 percent to the computed values of total embedment and design anchor force calculated in case 1:

$$\therefore \text{embedment} = (10.0+9.2) + 30\%$$

$$= 24.96 \text{ ft, say } 25.0 \text{ ft}$$

$$\therefore R_{DES} = 3.6 + 30\%$$

$$= 4.68 \text{ kips/ft, say } 4.7 \text{ kips/ft}$$

Now a check on moments developed for additional embedment is required. Use equation 3-103, assuming $x = 25.0$ ft – 10.0 ft = 15.0 ft, then solve for R.

$$P_p - P_a = 0.03 \text{ KCF}, \; C = 0.42$$

$$6P_n(h+x) - 6R(H+x-y) - x^3(P_p-P_a)C + \frac{[C(P_p-P_a)x^2-2P_h+2R]^2}{C[2(P_p-P_a)x+H_{P_p}-m_{P_a}]} = 0$$

Substituting values:

$$6(4.1)(11.4+15) - 6R(20+15-6) - (15)^3(0.03)(0.42) + \frac{[0.42(0.03)(15)^2-2(4.1)+2R]^2}{0.42[2(0.03)(15)+20(0.06)-10(0.03)]} = 0$$

$$649.44 - 174R - 42.53 + \frac{[2R-5.37]^2}{0.756} = 0$$

$$490.98 - 131.54R - 32.15 + [4R^2-21.48R+28.84] = 0$$

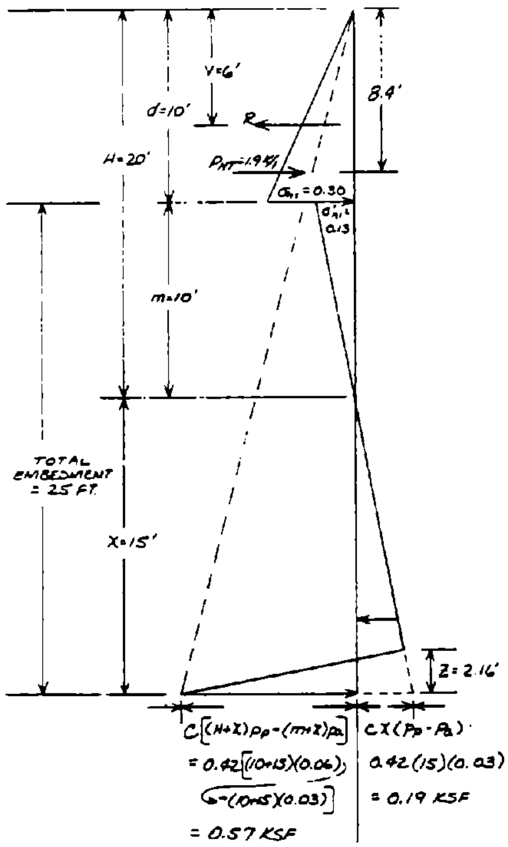
$$4R^2 - 153.02 + 487.67 = 0$$

$$0R, \; R^2 - 38.26R + 121.92 = 0$$

Solve for R by the quadratic equation.

$$R = \frac{\sqrt{(38.26)^2-4(121.92)}}{2} = 3.51 \text{ k/ft}$$

Case 2: Earth Pressure Diagram



From case 1

$$\sigma_{hl} = 0.30 \text{ KSF}$$

$$\sigma'_{hl} = 0.13 \text{ KSF}$$

$$m = 10.0 \text{ ft}$$

$$H = 20.0 \text{ ft}$$

The dimension z may be computed by summing horizontal forces for static equilibrium

$$z = \frac{2R - 2P_n + Cx^2(P_p - P_a)}{C[2x(P_p - P_a) + h_{P_p} - m_{P_a}]}$$

where R and x are case 2 values

$$\therefore z = \frac{2(3.5) - 2(4.1) + 0.42(15)^2(0.03)}{0.42[2(15)(0.03) + 20(0.06) - 10(0.03)]}$$

$$z = 2.16 \text{ ft}$$

case 2, $R = 3.5$ k/ft
$x = 15.0$ ft

Since K is almost the same as calculated in case 1, the moment at a depth $C' + d$ will not change significantly. Moment at b below depth H must be checked, however, from equation 3-104.

$$b = \sqrt{\frac{2(P_h - R)}{(P_p - P_a)C}}$$

$$= \sqrt{\frac{2(4.1 - 3.5)}{(0.03)(0.42)}} = 9.75 \text{ ft} < x = 15.0 \text{ ft}$$

so, b is valid.

From equation 3-105, M_b is:

$$M_b = P_n(h + b) - R(b + m + d - y) - b^3/6(P_p - P_a)C$$

$$= 4.1(11.4 + 9.75) - 3.5(9.75 + 10 + 10 - 6) - (9.75)^3/6(0.03)(0.42)$$

$$= -8.08 \text{ kip-feet/ft}$$

For $M_b = M_{max}$, the steel H-pile section required

$$S_{\min} = \frac{M_{\max}}{Fb}S = \frac{8.08(12)}{0.6(36)}(8)$$

$$= 35.9 \text{ in}^3 < 43.4 \text{ (i.e., } S_{xx} \text{ for H10 x 42)}$$

So, H10 x 42 is good enough for this job.

c. Anchorage System Calculations

Waler

The waler is designed by use of equations 3-126 and 3-127. Say use one tie rod per H-pile.

$$M_{\max} = \frac{A_p l}{8} \quad \text{and} \quad S_{RQD} = \frac{M_{\max}}{F_b}$$

$$\text{or } S_{RQD} = \frac{A_p l}{8F_b} \quad \text{where} \quad A_p = SR_{DES} = 8(4.7)$$

$$= 37.6 \text{ k/anchor}$$

$$\ell = 8 \text{ ft}$$

$$F_b = 0.66(36) = 24 \text{ ksi}$$

$$\therefore S_{RQD} = \frac{37.6(8)(12)}{8(24)}$$

$$= 18.8 \text{ in}^3$$

Assume a double channel waler of

$$2C \;\; 9 \times 13.4 \text{ which has } S = 2(10.6)$$

$$= 21.2 \text{ in}^3 > 18.8 \;\therefore\; \text{OK}$$

Check waler for web crippling by reference to the *AISC Steel Manual*, section 5, specifications. It is found that, for interior loads,

$$\frac{P}{t(N+2K)} \leq 0.75F_y = 27 \text{ ksi for ASTM A-36 steel.}$$

where $P = Ap/2$ = tie rod force, kips
t = web thickness
N = length of bearing plate
K = channel fillet dimension

Try $N = 7.5$ in, $K = 15/16" = 0.94$ in, $t = 0.233$ in.
$Ap = 8(4.7) = 37.6$ kips

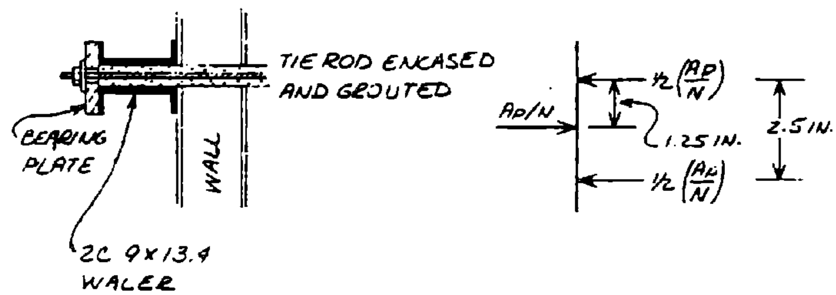
Substituting values and solving:

$$\frac{(37.6)/2}{0.233(7.5+2(0.74))} = 8.6 \text{ ksi} < 27 \text{ ksi} \quad \text{so, OK}$$

Bearing Plate

Assumed allowance of 2.5 inches spacing between channels for installation of tie rod and corrosion protection. Spaces should be installed on both sides of the tie rod between channels to maintain proper separation.

Bearing plate length N = 7.5 inches—try square line. Assume that each channel carries $Ap/2$ equally over the length of bearing.



Compute bending moments and section modulus required for bearing plate, referring to free body diagram above.

$$M_{\max} = 1/2Ap/N(1.25) = 1/2(37.6/7.5)(1.25)$$

$$= 3.13 \text{ in-kips}$$

$$S_{RQD} = \frac{M_{\max}}{F_b} = \frac{3.13}{0.6(36)} = 0.145 \text{ in}^3$$

The section modulus of the plate is $\frac{Nt_p^2}{6}$, so, plate thickness requirement is:

$$tp = \sqrt{\frac{6}{N}S_{RQD}} = \sqrt{\frac{6}{7.5}(0.145)}$$

$$\therefore tp = 0.34 \text{ in}$$

Say use 3/8" plate –7-1/2" x 7-1/2" x 3/8" PL for tie rod bearing.

Anchor Rods

Assume DYWIDAG high strength rock and soil anchors for the design tie rod force of 37.6 kips. The rod size required would be:

$$A_{ROD} = \frac{Ap}{F_a} \quad \text{where} \quad F_a = 0.6\ Fy$$

$$= 0.6\ (120\ \text{ksi})$$

$$= 72\ \text{ksi}$$

$$\therefore A_{ROD} = \frac{37.6}{72} = 0.522\ \text{in}^2$$

$$\phi_{ROD} = \sqrt{\frac{4A_{ROD}}{\pi}} = \frac{\sqrt{4(0.522)}}{\pi}$$

$$\therefore \phi_{ROD} = 0.82\text{in}$$

Assume 7/8" rod diameters.

As the wall is within close range of the sound basalt face, anchorage would be best constructed into rock.

Referring to figure 3-93, since the rock has q_u - 10 ksi, the ultimate anchor capacity would be approximately 61 kips/feet.

$$\therefore L_{ANCHOR} = \frac{Ap}{61}(\text{F.S.}) \quad \text{say} \quad \text{F.S.} = 5$$

$$L_{ANCHOR} = \frac{37.6}{61}(5) = 3.1\ \text{ft (grouted length)}$$

Check equation 3-133, ℓg = 60(7/8)/12 = 4.4 feet.

So, for conservatism, use grouted anchor length = ℓg = 4.4 feet.

Lagging

For noncohesive backfill, use equation 3-85 to design timber lagging, maintain same lagging for full height. Need to add component of moment due to P_{AT}.

$$M = 1/12 P_a zs^2 + 1/4 P_{HT}\ S = 1/12(0.03)(10)(8)^2 + 1/4(1.9)(8)$$

$$= 1.6 + 3.8 = 5.2\ \text{ft-kips}$$

The section required may be computed from equation (3-82), where F_b is the allowable bending stress. Assuming Douglas fir lagging,

F_b = 1,300 psi (No. 1, 5" and thicker, width more than 2" greater than thickness).

Assuming width = 12"

$$S_{\min} = \frac{7.0(12)}{1.300} = 64.6 \text{ in}^3$$

$$\text{By equation 3-86:} \quad S = \frac{bt^2}{6} \quad \text{or} \quad t = \sqrt{\frac{6S_{\min}}{b}} \qquad b = 12 \text{ in}$$

$$t = \sqrt{\frac{6(64.6)}{12}} = 5.7 \text{ in, rounded to 6 in}$$

So, use 6 inches by 12 feet by 8 feet No. 1 Douglas fir lagging.

4G.3 Additional Design Considerations

4G.3.1 Drainage

As with all retaining structures, drainage is a major consideration. However, for the case shown, the talus rock should be free-draining.

4G.3.2 Anchor Location

Generally, the anchor location depends on the site geometry. However, note that deepening the anchor depth reduces the required penetration and increases the anchor force and the maximum bending moment. Hence, a careful evaluation of economics is required in order to select the most economical anchor location and H-pile selections.

The orientation of the anchor hole will be determined by the joint patterns in the rock and must be determined on a case-by-case basis. If threaded bars are utilized for tie rods, there are some obvious advantages to having a horizontal orientation to the hole.

4G.3.3 Anchor Tie

Wire rope should not be used as an anchor tie on permanent projects. On temporary projects, however, wire rope should only be used with the proper thimbles, clamps, and blocking to prevent the cable from damage at sharp transitions of a wrap.

Positive permanent corrosion protection must be provided for all embedded anchorages.

4G.3.4 Tie Rod Orientation

In the design problem the orientation of the anchor tie rod is horizontal; hence, there is no vertical component of the anchor force. If the anchor tie rod were angled at some orientation other than horizontal, then the soldier piles would be subjected to a vertical force. This force must be considered when designing the soldier pile. If the vertical component of the anchor load is sufficiently large to cause the pile to move either up or down, a significant drop in anchor load may result with an attendant deflection of the wall.

4G.3.5 Lagging

Lagging is commonly attached to H-pile cantilever walls by two methods. The most common method entails cutting the lagging to length and slipping it into the channels of the H-pile. The second method requires welding threaded studs to the face of the H-pile and bolting the lagging to the H-pile.

4G.4 Construction Considerations

4G.4.1 Equipment

Construction of an anchored H-pile wall that has driven piles and a grouted anchorage requires the following special equipment: a crane, a pile driver, and a drill capable of boring the anchor holes. The remainder of the equipment would be required to construct essentially all other wall types.

4G.4.2 Materials

Steel H-piles and timber lagging should conform to AASHTO specifications for permanent installations. In lieu of AASHTO specifications, the design engineer may opt to accept an alternate or lower specification, particularly if used materials are involved or for a temporary wall.

Special care should be taken to protect the anchor hardware against corrosion and all timbers that are cut or drilled in the field should be field treated with wood preservative in accordance with AWPA standard M-4.

4G.4.3 Pile Driving

Any pile driver capable of driving the H-piles is suitable, provided that the pile is not damaged during driving. The piles should be driven to specified alignment and penetration at the designated location under the observation of a qualified inspector. The vertical alignment should be controlled to $\pm$ 1/4 inch horizontal and 12 inches vertical, and the horizontal location should be held to $\pm$ 3 inches of the design. Soldier piles may be pulled into position, provided that the moment induced in the pile by pulling combined with the design loads does not exceed the design characteristics of the piling. However, the piles should not be heated to achieve design alignment.

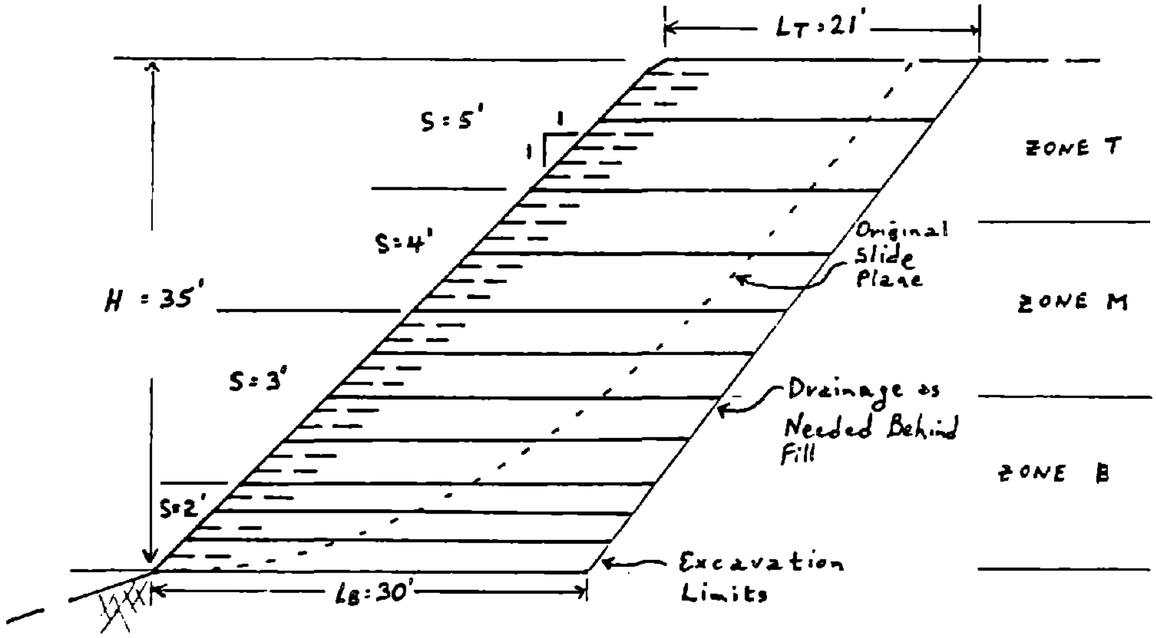
4G.4.4 Anchor Testing

Pulling tests should be performed on the grouted anchorages prior to backfilling to verify the design strength of the anchorage. Typically, the anchors are tested to 150 percent of the design load. After backfilling, the anchor load should be verified with lift-off tests.

4G.4.5 Backfilling

Backfilling of a tied back wall is similar to a cantilever wall, except that the contractor must be aware of wall deflections, particularly prior to the time of the tieback rod installation. Regardless of when the tieback is installed and tensioned, the backfill behind the wall must be allowed to relax into the active state of stress so that the anchors will not be overstressed. As mentioned previously, lift-off tests should be performed to verify the anchor loads.

Sample Problem



Determine a conventional reinforced embankment design for the 35-foot high slide area shown above.

Assumptions

- Repaired slope face will be 1:1 (45°)
- Factor of safety (overall) = 1.3
- The slope is "drained" ($r_u = 0$) (no ground water)

Given

- Backfill material $\phi = 30°$, $c = 0$, $\gamma_m = 120$ pcf (non-plastic silty sand)
- No surcharge on the top of the slope
- Foundation material is bedrock (strength exceeds backfill properties)

1. Effective fill height (H')

$$H' = H + q\ (\text{surcharge})/\ {}_m$$
$$H' = 35\text{ ft} + 0/120 \cdot 35\text{ ft}$$

2. Factored friction angle (for stability, $F.S. = 1.3$)

$$\phi_f' = \tan^{-1}(\tan\phi'/FS)$$

$$\phi_f' = \tan^{-1}(\tan 30°/1.3) = \tan^{-1}(.577/1.3)$$

$$\phi_f' = \tan^{-1}(.444) = 23.9° \qquad \text{use } 24°$$

3. Force coefficient

From figure 5A, for $\beta = 45°$, $\phi_f' = 24°$,
Force coefficient $K = 0.17$

4. Total horizontal force (T_{tot}) (to be resisted by geogrids)

$$T_{tot} = 1/2 \cdot K \cdot \gamma_m \cdot H'^2$$

$$T_{tot} = .5 \cdot .17 \cdot 120 \cdot (35)^2$$

T_{tot} = 12,495 pounds (per foot of fill)

Note that total horizontal force determined from stability analysis, using XSTABL, = 11,800 pounds per foot.

5. Minimum number of geogrid layers required

$$N_{min} = T_{tot}/T'_{allow}$$

Assume using geogrid with manufacturer's
T_{allow} = 2000 pounds/foot.

$T'_{allow} = T_{allow} \div F.S.$ (overall) = 2000 ÷ 1.3 = 1538 pounds/foot

N_{min} = 12,495/1,538 = 8.12 ⇒ use <u>9</u> layers

If a lighter geogrid (T_{allow} = 1000 lb) were used,
N_{min} = 12,495/1000 ÷ 1.3 ⇒ 17 layers

6. Distribution of reinforcing layers

Divide the 35-foot embankment into three ± 11° thick zones

<u>Bottom zone</u> (B) receives one-half the reinforcing and force

12,495 lb ÷ 2 = 6,247 lb; 6,247 lb ÷ 1,538 lb(T'_{allow}) = 4.06 ⇒ use 5

<u>Middle zone</u> (M) receives one-third the reinforcing

12,495 lb ÷ 3 = 4,165 lb; 4,165 lb ÷ 1,538 lb = 4.06 ⇒ use 3

Top zone (T) receives one-sixth the reinforcing

12,495 lb ÷ 6 = 2,082 lb; 2,082 lb ÷ 1,538 lb = 1.35 ⇒ use 2

Total of 5 + 3 + 2 = 10 layers > 9 layer minimum—OK

For intermediate reinforcement, use a low strength geogrid on a 1-foot spacing (every two 6-inch lifts), 4.5 feet long (one-half of a 9-foot long roll).

7. Vertical spacing:

Bottom zone	12 feet ÷ 5 layers = 2.4 feet
Middle zone	12 feet ÷ 3 layers = 4 feet
Top zone	11 feet ÷ 2 layers = 5.5 feet

8. Check maximum allowable vertical spacing:

$S_{max} = T'_{allow}/K \cdot \gamma_m \cdot z$ (z = depth)

At bottom of fill (depth z = 35 feet)

S_{max} = 1,538 lb/.17 · 120 · 35 = 2.15' < 2.4' Use 2-foot spacing

At bottom of mid zone (depth z = 24 feet)

S_{max} = 1,538 lb/.17 · 120 · 24 = 3.14' < 4' Use 3-foot spacing

At depth z = 18', S_{max} = 4' Use 4-foot spacing

At depth z = 12' (top zone), S_{max} = 6.2' > 5.5' Use 5-foot spacing

9. Required embedment length:

Determine length at top (L_T) and length at bottom (L_B) using figure 5B (or 5C when appropriate)

For $\beta = 45°$ and $\phi_f' = 24°$, $L_T/H' = 0.6$

$L_T = 0.6 \cdot 35' = 21$ feet

$L_B/H' = 0.85$

$L_B = 0.85 \cdot 35' = 30$ feet

Interpolate lengths between these values midslope.
Check with stability analysis and pullout formulas.

10. Final reinforcement design is as shown on figure 5 above.

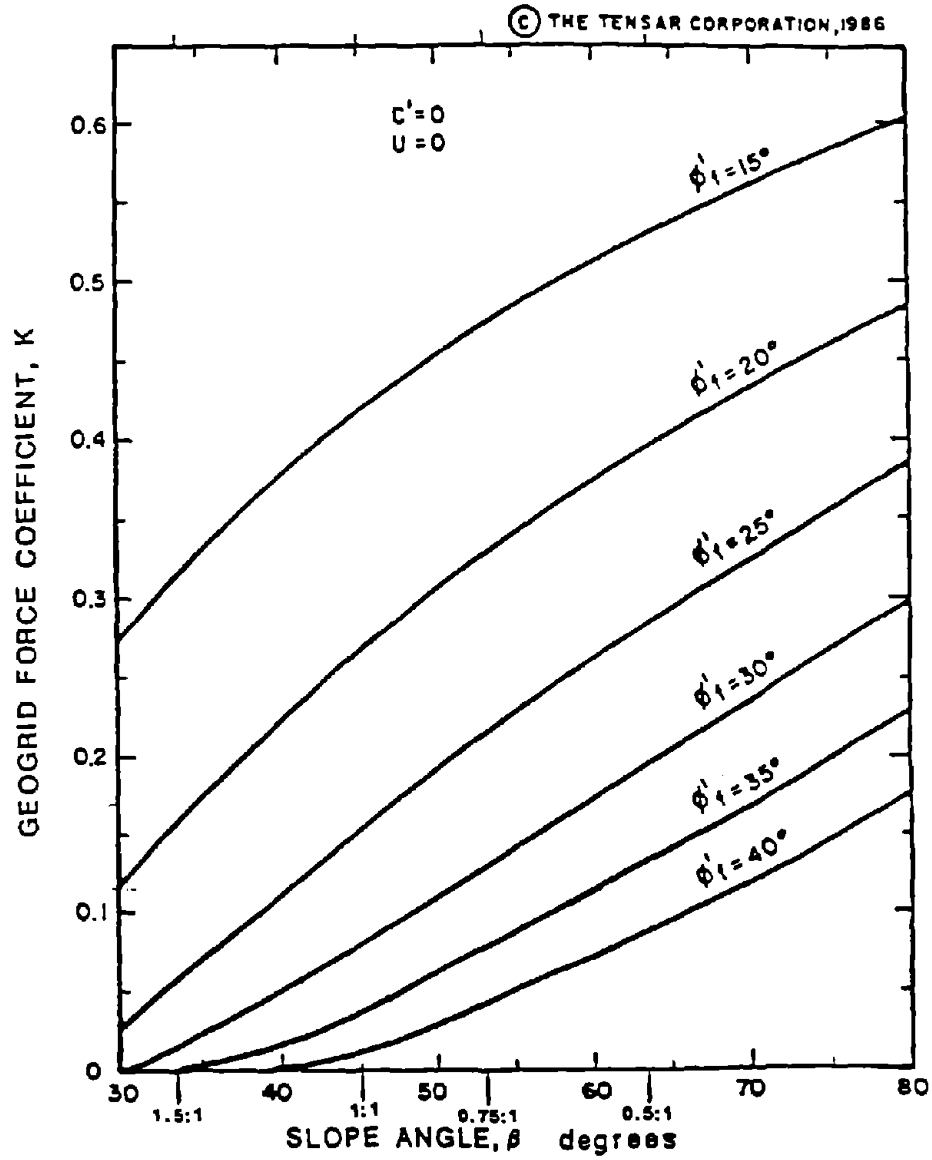


Figure 5A.—Geogrid force coefficient chart.

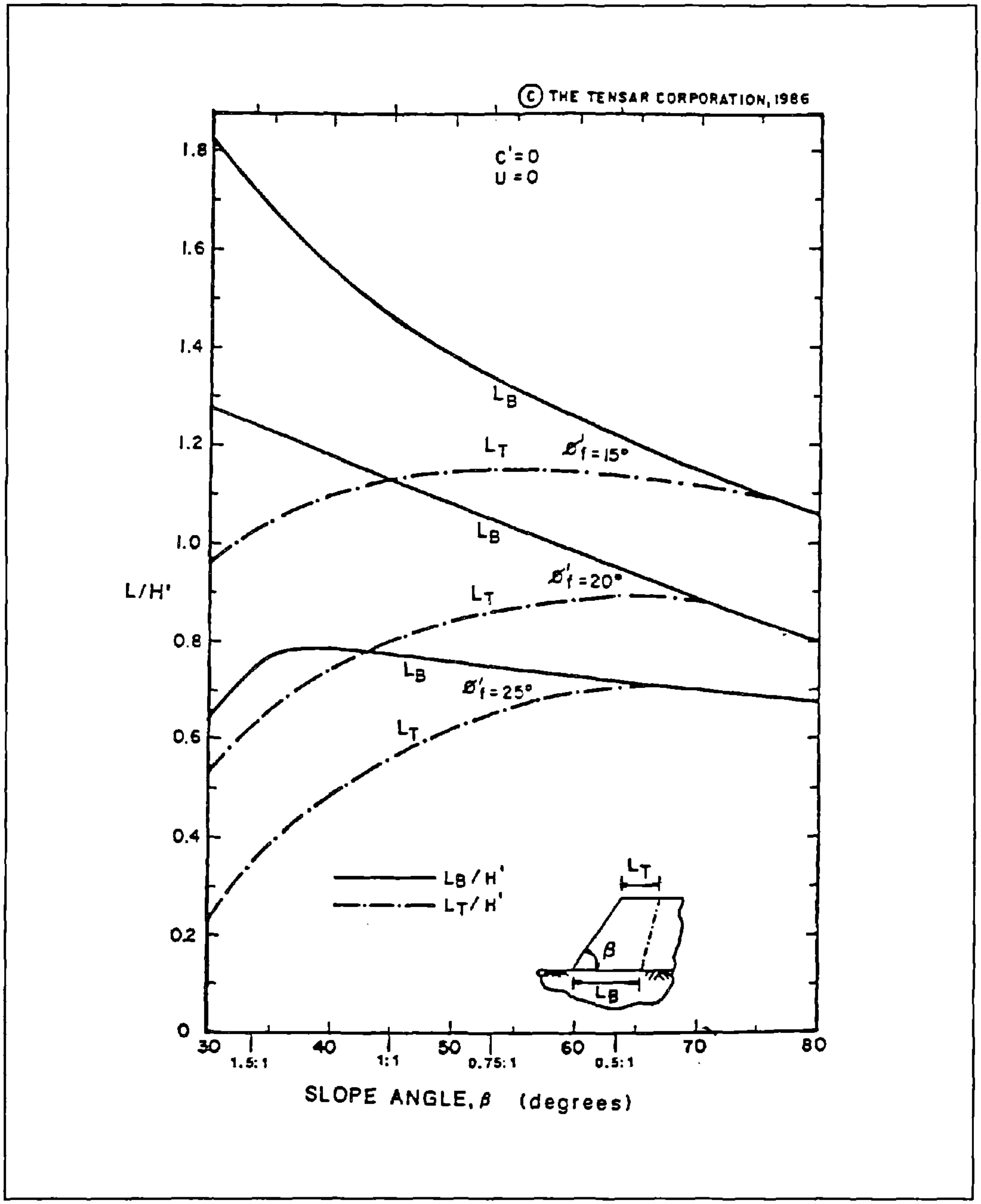


Figure 5B.—Geogrid length chart. $\phi_f' = 15°$ to $\phi_f' = 25°$.

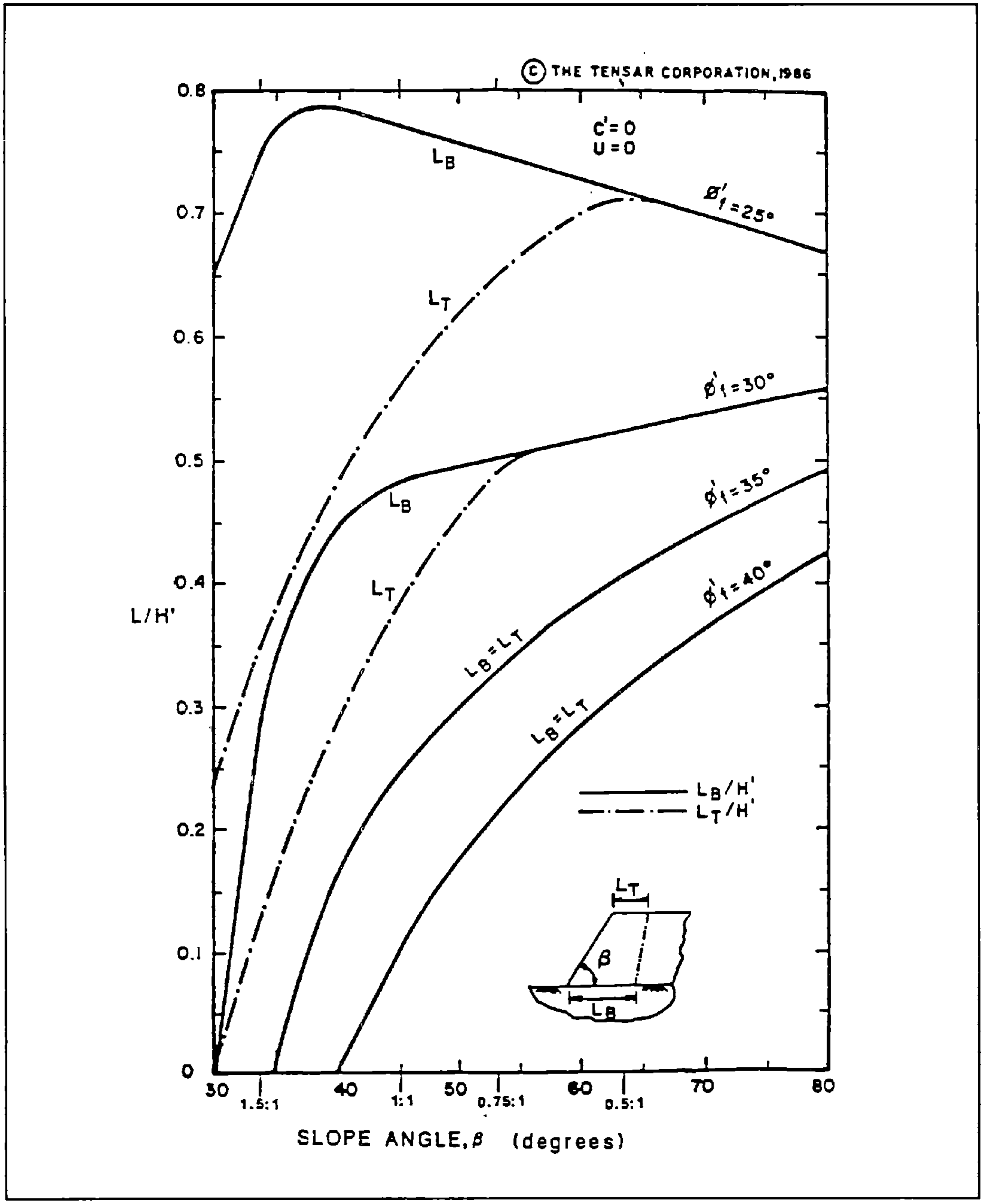


Figure 5C.—Geogrid length chart. $\phi_f' = 25°$ *to* $\phi_f' = 40°$.

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Appendix A
Geotechnical Considerations

For each project, the early phases of the investigation should involve gathering data related to the site, soils, rock, and possible existing structures. The intensity of this effort depends on the size and scope of the project, and on the soil and anticipated foundation problems. For major projects, there may be several phases to the investigation, from preliminary site visits and explorations to final detailed investigations.

The first step is to collect and evaluate available information on soil and rock conditions and ground water conditions from the following sources:

(1) Geologic maps (for example, the U.S.G.S. quadrangle maps of surficial and bedrock geology)

(2) Existing test boring logs and records of other subsurface explorations

(3) Topographic maps

(4) Air photos

(5) Soil resource inventory maps

Always visit the site. More often than not, you will observe something which will have a bearing on the design and construction of the structure. A geological reconnaissance by a qualified engineering geologist or a geotechnical engineer should provide useful information concerning soil and rock conditions likely to be encountered at the site. The Unified Soil Classification System from the *U.S. Bureau of Reclamation Earth Manual*, should be used to describe soils, and the Unified Rock Classification System developed by Doug Williamson should be used to describe rock for engineering purposes.

When visiting the site, take the following materials and tools:

(1) The best available topographic map of the site

(2) Air photos (if available)

(3) Stereo glasses

(4) Shovel, grub hoe, G-pick, bags, and jars

(5) Drive probe

(6) Camera and color film

(7) 100-foot tape

(8) Brunton compass

(9) Geologic maps

(10) Existing subsurface information (if available)

Other materials may be added to suit the probable site conditions and preliminary field explorations which may be required.

The attached form, "Preliminary Site Investigation-Field Summary," has been tailored to the problem of retaining structures, and it is recommended as a checklist of information to gather at the site.

Preliminary
Site Investigation-Field Summary

File no. ____________
Date of inspection____________
By____________
Accompanied by____________

General

Agency____________
Name of project____________
Location of site____________
U.S.G.S. Quadrangle____________Size of site____________
Description of proposed development____________
____
____

Available site plans and other drawings (DWG. no., title, author, date). If no plans are available, prepare a sketch and attach to the report.____________
____
____

Available photographs____________
Available bedrock or surficial geology maps____________

Site Access

Sketch location of roads and trails not shown on site plan and note type and condition. Describe type of exploration equipment which can be moved into site, e.g. small or large backhoe either rubber tired or tracked, truck mounted drill rigs, small portable rigs, etc.
____
____
____
____

Topography (Level: rolling, hilly, mountainous, etc.)

Describe____________
____
Approximate degree of slopes____________

Page 1 of____

Site Investigation-Field Summary File no.____________

Maximum elevation______________________ Minimum elevation_________________________

Reference datum (USGS MSL, etc.) ____

Geomorphology (tidal marsh, bog, landslide, flood plain, valley floor, etc.)

Describe____

____

____

Is the site virgin land? ____

Character of erosion gullies (steep, moderate, or gently inclined)

____

____

Surface vegetation (wooded, brush, grass, cleared, filled, etc.)

Describe____

____

____

Size, type, and condition of trees ____

____

Drainage (locate features on sketch.)

Natural streams and springs (estimate gradient and discharge)

____

Man-made drainage ditches and culverts (describe type and estimate size, lined or unlined, head walls, presence of water)____

____

____

Probable ground water elevation (from observations in pits, wells, bogs, ponds, streams, etc.)

____

____

Evidence of flooding history____

_____________________________________Maximum flood elevation________________

Local landsliding (locate features on sketch or map-scarps, ground cracks, sag ponds, grabens, springs, drainages, outcrops, etc.)_____________________________________

____

Consistency of slide surface (soft, wet, hummocky, etc.)___________________________

____

Describe landslide (size, shape, probable depth to failure plane, etc.) ________________

____

____

Page 2 of____

Site Investigation-Field Summary File no.____________

Material types (soils, rocks)________________________________

Describe probable mechanism of failure (support with rough cross-section, and show scarps and assumed failure plane)________________________________

Existing structures on site or nearby (describe and locate on sketch: type (such as buildings, retaining walls, bridges, fills, roads, size, owner, foundation available borings, performance)________________________________

Soil and rock features (show applicable information on sketch)

Bedrock: type of rock (outcrops, cuts)________________________________
________________________Relative hardness________________________
Orientation of discontinuities (jointing, bedding planes, fissility)________________

Surface boulders (size, number, type of rock) ________________________

Soil Exposures: (describe types of materials and vertical sequence noted in each exposure, e.g., in open pits, roadway cuts, trenches, erosion gullies, etc.)________________

Man-placed fill (granular, sawdust, mine tailings, etc.) ________________________

Topsoil thickness and type ________________________

Organic materials (peat depths, limits, etc.)________________________

Probable subsoil conditions: (describe as completely as possible)________________

Previous subsurface explorations: ________________________

Sources of borrow: (where applicable, give name and location of pit, name, address, and telephone no. of owner, distance from site, type of soil and/or rock sample, estimated quality, price, estimated resources, etc.) ________________________

Page 3 of___

Site Investigation-Field Summary File no.____________

Preliminary notes, comments, and recommendations: (suitable wall types, additional investigation, borings, mapping, pits, field instrumentation, etc.)

(1)

(2)

Page 5 of____

Geotechnical Exploration by the Drive Probe Method

Douglas A. Williamson (USFS ret.), Engineering Geologist

Introduction

The relative density probe is a simple exploratory device that is used to determine the distribution and estimated strength of the subsurface soil units and decomposed rock units. The drive probe also can be used to determine the presence of water and subsurface water levels. The drive probe does require some equipment and moderate work effort, but it does define strength parameters of the soil units. Once set up, it is much quicker than hand-auguring, but it does not provide a sample to determine plasticity. One or more auger holes can be used to classify the subsurface materials, and the drive probe can be used to define the subsurface for the rest of the project area.

The equipment is relatively inexpensive (less than $500) and can be either specified and ordered or fabricated in-house. The drive probe is most effective when used with conventional geotechnical soil exploration to inexpensively extend the known conditions revealed by conventional drilling. The drive probe can also be used alone and has been found to be accurate and discrete in defining subsurface conditions for difficult access projects. It has been proven through geotechnical work over a period of more than 4 years.

"Relative density" in this document is defined as the estimated strength of subsurface materials determined by the resistance to penetration. The resistance to penetration is measured in blows per foot of an 11-pound, circular hammer freely falling 39 inches, striking a coupling, driving a 1-inch-diameter solid end area. Note that the 39 inches is the distance between couplings on a 4-foot pipe length minus the length of the hammer. The "driving-end" of the pipe is a coupling with a pipe plug in it.

Equipment

The equipment for the drive probe consists of segments of 1/2-inch galvanized pipe (3/4-inch-diameter o.d.), cut into 4- and 5-foot lengths and threaded on each end. Commercially, 1/2-inch galvanized pipe comes in 21-foot lengths, so the pipe is cut into one 5-foot length and four 4-foot lengths to eliminate waste. Smooth-walled pipe couplings with full threads (1-inch-diameter o.d.) are used to join the pipes together as the hole is advanced. The 5-foot length is used for the initial drive. The starting 5-foot length is randomly drilled with 3/16-diameter holes to allow water to enter the pipe as it is driven into the ground. It is critical to the operation that the threads on the ends of the pipe be no longer than that which will meet in the center of the full thread couplings when two pipes are joined. This allows the driving force to be transmitted through the pipe ends and not through the threads. The pipe lengths can be made up in advance, with a coupling on one end and ready for adding to the drive string.

A circular, mild steel hammer of 11 pounds is used. The total hammer length is 7-3/4 inches long, including the guards. The driving section is

7 inches long (minus the guards), and 2-1/2 inches in diameter. The driving section has a hole in the center that is 7/8 inches in diameter through which the pipe will pass freely. Welded on each end of the driving section are round handguards. The handguards are 4 inches in diameter and are made from 3/8-inch-thick, mild steel plate.

Additional equipment needed are: two pipe wrenches (preferably Rigid brand) of the magnesium-aluminum variety for convenient weight; a plastic, 5-gallon bucket to carry tools and equipment, and to add water to the hole, or to stand on, if necessary; an electric water level measuring device (M-scope); a funnel; a 5-foot measuring stick; yellow keel-type marking pencil; a field notebook and pencils; and a folding shovel. Several spare couplings are also convenient.

A 1/2-inch pipe die and die stock, a pipe cutter, a pipe reamer, a pipe vise, and a 1/2-inch "Easyout" to remove broken pipe threads from couplings are convenient and economic in-house equipment. A supply of smooth-walled, full thread pipe couplings should be kept on hand.

A drive probe operation is designed for a one-or-two person operation for either technical or nontechnical personnel members. Depths up to 30 feet have been achieved without unusual effort. When "refusal" is reached with the "free-falling method" the hole can be continued by "hand-driving" methods to "absolute refusal."

A small portable tripod is useful if extensive drive probe drilling is done. The tripod can be made from three 10-foot lengths of 1-inch-diameter electrical conduit joined at one end by a 1/2-inch-diameter bolt. A convenient sheave suitable for use with a 3/4-inch diameter rope is attached to the bolt at the top of the tripod. The hammer can then be raised and lowered by attaching the rope to the hammer and pulling on the rope. A loop can be tied in the rope and a foot can be used to raise and lower the hammer. This method is especially useful when "pulling a string." Gloves are a must when either driving by hand or using a rope.

Procedure

The site selected for the probe is cleared of the surface duff or organic material to expose the upper surface of the soil deposit. The starting 5-foot length is measured and marked in 1/2-foot intervals with the keep or marking pencil. The 4-foot length with the hammer on it is screwed into the top of the 5-foot length. The hammer is then raised up to the coupling on the upper end of the 4-foot length and allowed to fall freely and strike the coupling on the upper end of the 5-foot length. A coupling should always be screwed into the top of the 4-foot-length to prevent the hammer from flying off the top of the pipe when raising the hammer.

As each segment is driven, the number of blows required to advance the hole for each one-half foot (6 inches) is recorded. After driving 5 feet, the amount of open hole is measured and the hole checked for the presence of water. The 4-foot pipe segment is unscrewed and the hammer is moved to the uppermost pipe as each additional 4-foot segment is added to the string. The bottom pipe segment is marked each time. The hole is then advanced another 4 feet. The open hole and water level are again checked, and so on, until refused.

The friction of the pipe in the hole is primarily between the couplings and the sides of the hole, not between all of the pipe in the hole. This causes easier advance of the hole than if all of the pipe were in contact. It is recognized that the friction increases slightly with each additional coupling in the hole, but with this type of operation it is negligible. In granular soil materials, all of the pipe is probably in contact with the sides of the hole.

The pipe lengths can be removed from the hole for the inspection of thread conditions, for cleaning and removal of soil material in the pipe, or for recovery of the pipe for further use. The hammer is driven upwards, striking the coupling on the top of the uppermost pipe. Repeated blows remove whatever length is desired. It may be convenient to have a 2-foot segment for pipe removal for a more convenient driving length. Shorter people may have to stand on the bucket when starting the hole or when removing pipe from the hole. The pipes can either be retrieved or left in the hole for later water measurements.

Field Notebook

The headings for the information to be recorded in the field notebook are:

Project Date Hole Location and Elevation Crew

Page headings are to include:

Depth, From/To, Internal, Blows, Blows Per Foot, Open Hole, Depth to Water

"Write-in-the-Rain" level books are recommended.

Calculations

When doing drive probe exploration, the blows needed to advance the hole for the 1/2-foot (6 inches) interval are doubled to get "Blows per foot" that represents the strength of the material for that interval. Six inches are driven in order to be sure of detecting a layer 1-foot or more in thickness. The longer the interval drive, the less likely that thin layers will be detected. Any depth interval can be driven and converted to blows per foot for that interval. In all cases, whatever interval is advanced, the blows per interval are converted to blow per foot, which is the standard of comparison for that given material. The blows per foot has been found to be a discrete and reliable means of correlating between drive probe holes.

When a hole is drilled into a soil deposit by geotechnical driving methods, there is a "typical anticipated result." This anticipated result is that the number of blows necessary to advance the hole will increase with depth to a point which the hole cannot be advanced. In other words, soil deposits normally increase in strength and density with depth. When doing drive probe drilling, the result that is of most importance to a subsurface strength assessment is the detection of a relatively weaker layer revealed by a "blow-count-reversal," which occurs when the blows per foot decrease with the depth rather than increase. This condition indicates the strength and density of the soil material in that interval has decreased. It is this blow-count-reversal that is of prime significance in assessing foundation conditions or slope stability.

Editor's Note: ***The blows per foot determined by this method is not equal to the blows per foot in the Standard Penetration Test where a 140-pound weight is dropped 30 inches to drive a standard 2-inch outside diameter split spoon ampler. Although the blows per foot are not equal for both methods, the relative change in blow counts at various depths will provide an indication of changes in relative density.***

Water Measurement

Drive probe drilling allows static water levels to be measured and simple permeability tests to be done. When water levels that are measured in the pipe during and after driving appear inconsistent with previous measurements, water can be added to the pipe to clean out the holes in the tip. The depth-to-water level can be checked in the pipe after a period of time to see if there is a change. Water can be added to the holes already in the ground, at any time, to check on the reliability of the previously measured static water levels.

There appears to be a "natural sealing" between the pipe and the plastic soil materials which prevents water movement up along the pipe. Metered water flows can be added to the pipe to do maintained head tests. Falling head tests can be done by timing the drop in water level. From these data the permeability can be calculated.

Water can be directly added to the pipe with the 5-gallon bucket or by means of a backpack tank. The hydraulic head of the water in the pipe will usually clear the holes in the tip. When adding water with a bucket, a "screw-on"-type funnel is convenient, but any funnel will do. If a backpack tank is used, a 1/4-inch plastic tube attached to the tank is inserted in the pipe to the bottom of the pipe. The tube is then moved up and down to flush out any soil material filling the holes in the tip.

Log Form For Drive Probe Exploration

A sheet of 8 1/2- by 11-inch graph paper is used for a log form. The depth in feet is recorded along the left side of the sheet and the blows per foot are shown along the top. A convenient scale is used, usually two small squares equal 1-foot on a 10th grid. The blows per foot interval are shown as a bar graph from left to right which indicates "relative strength" according to length. The usual interval is 1/2-foot (6 inches) although any interval can be shown. Elevations are also shown along the sides of the sheet. Interpretative lines are drawn showing correlations of hole segments having similar blows per foot. These correlations indicate changes in material and strength.

Appendix B
Condition Survey—Field Summary

File no. ____________________
Date of inspection__________
By__________________________
Accompanied by____________

General

Agency____
Name of project____
Location of site____
U.S.G.S. Quadrangle____
Description of proposed development___________________________________
____
____

Available site plans and other drawings (Dwg. N., title, author, date). If no plans are available, prepare sketch and attach to report.________________________________
____
____

Available photographs____

(1) Describe the general condition of the structure and surroundings: Indicate evidence of tilting, differential settlement, sunken spots in backfill, sloughing or sliding, cracks in road prisms or surrounding ground, erosion, and scour. Indicate schematically on topography and as-builts.

(2) Describe any specific areas of deterioration (rust, corrosion, rot, degradation of fabric, cut fabric, broken gabion baskets, loss of soil from crib walls, cracks in structure, overstressing of structural elements due to settlement crushing and cracking). Augment verbal description with color photos.

Condition Surveys-
Field Summary (continued)

No.______________

(3) Describe condition and tightness of all exposed hardware.

(4) Perform lift-off tests on the tieback anchors (if possible). Append all test data and summary report.

(5) Check internal and external drainage for blockage and describe conditions.

(6) Monitor all field instrumentation and evaluate data. Append all data and a summary report.

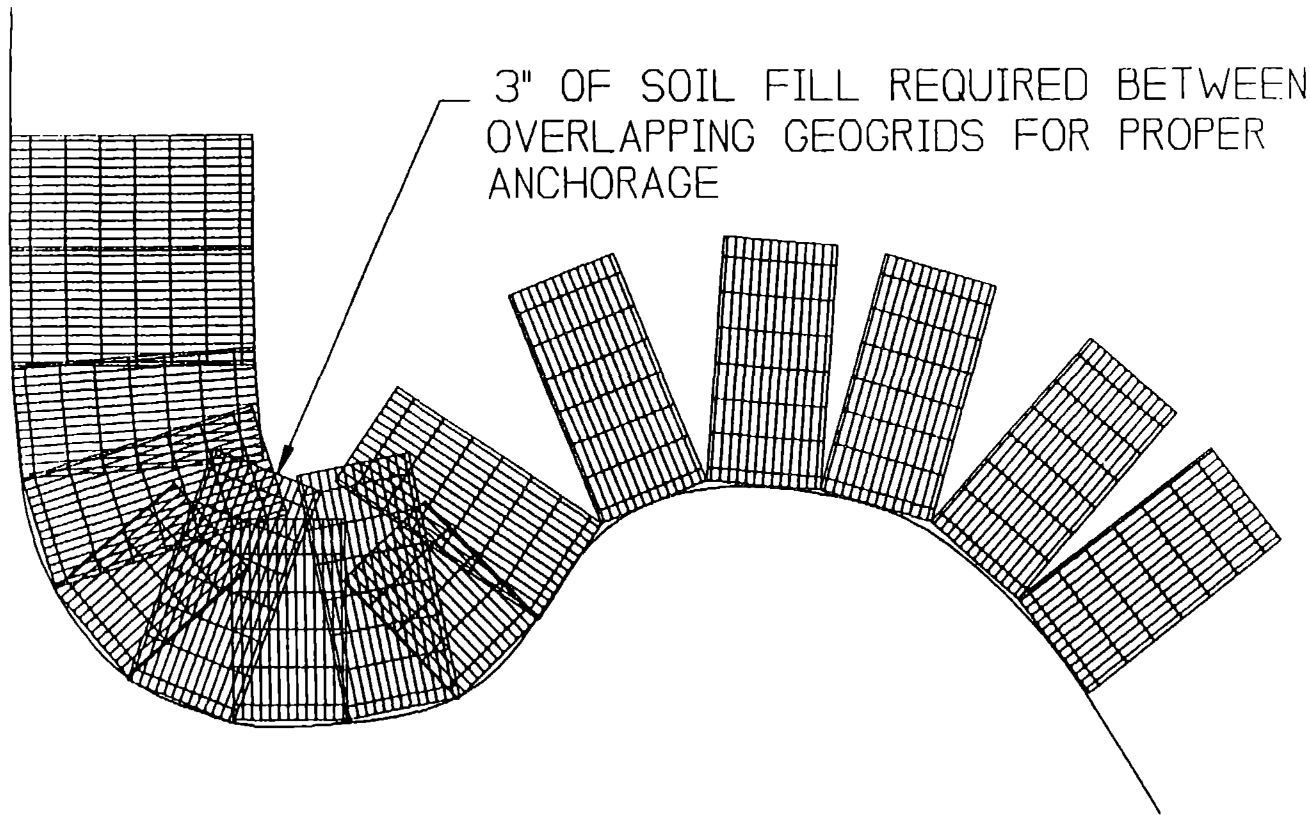
(7) Make specific recommendations for repair or modification, if required.

General overall rating of structure

Excellent		Fair		Poor
5	4	3	2	1

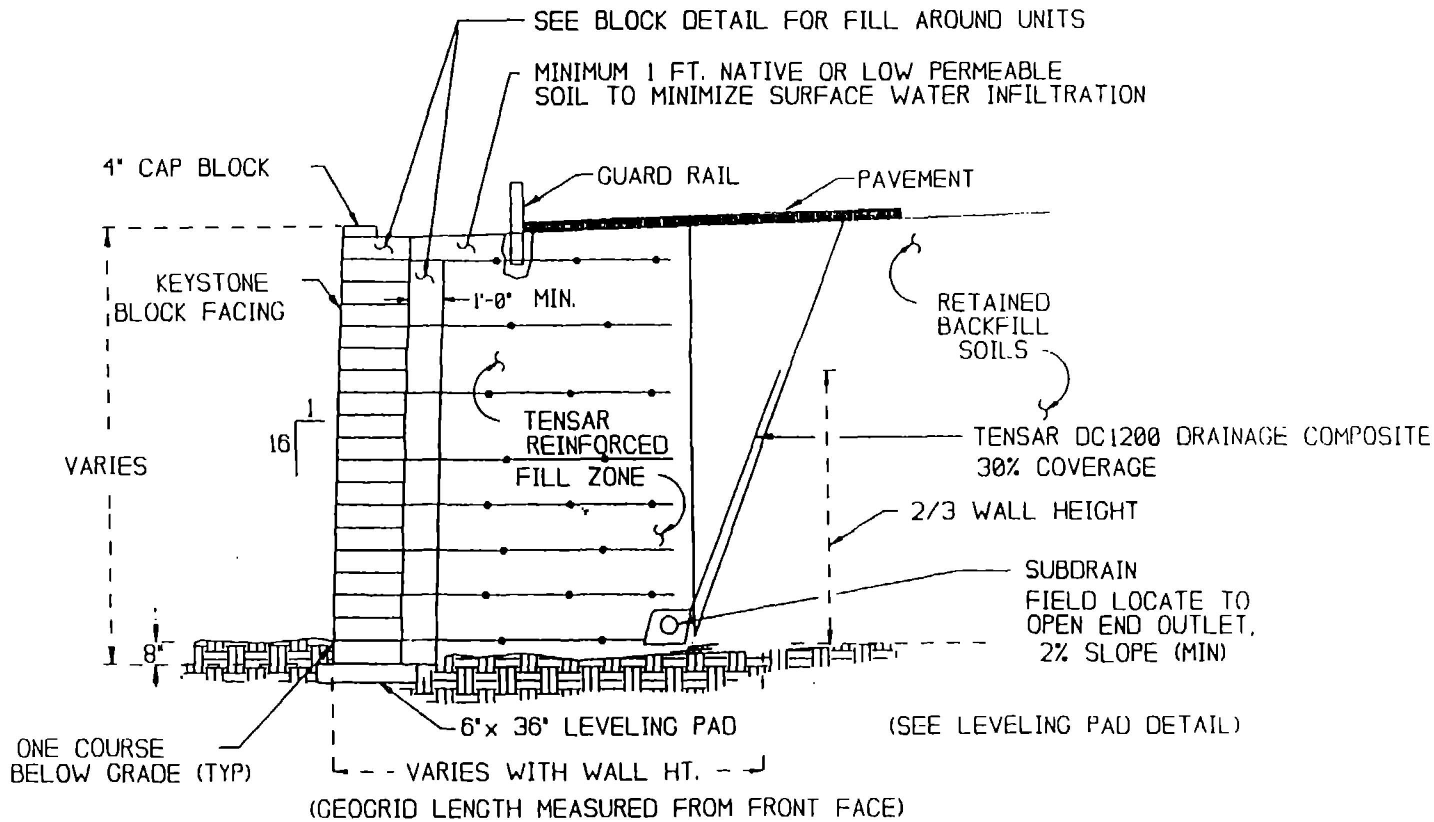
____
Signature

Appendix C
Construction Details



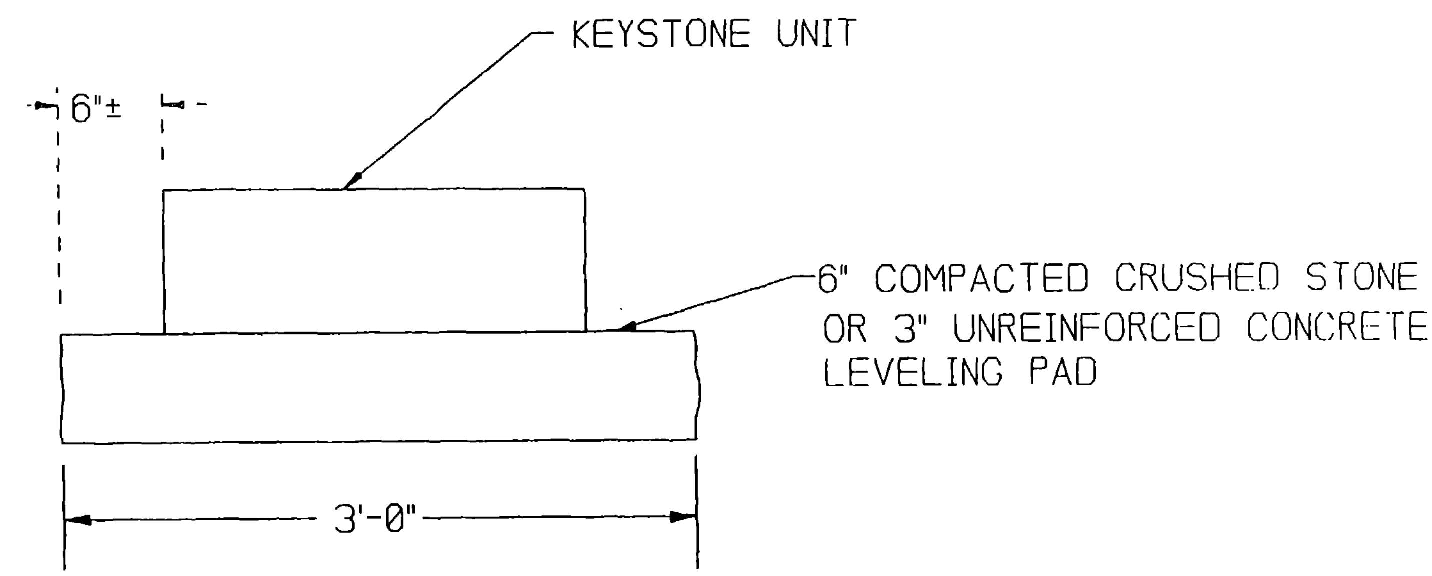
GEOGRID PLACEMENT ON CURVES

NOT TO SCALE



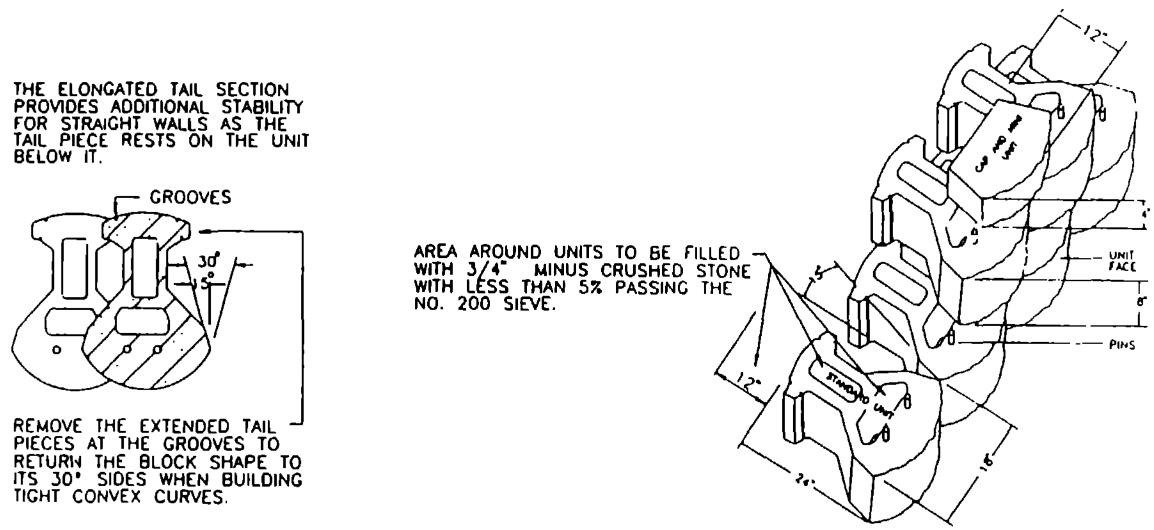
TYPICAL CROSS SECTION

NOT TO SCALE

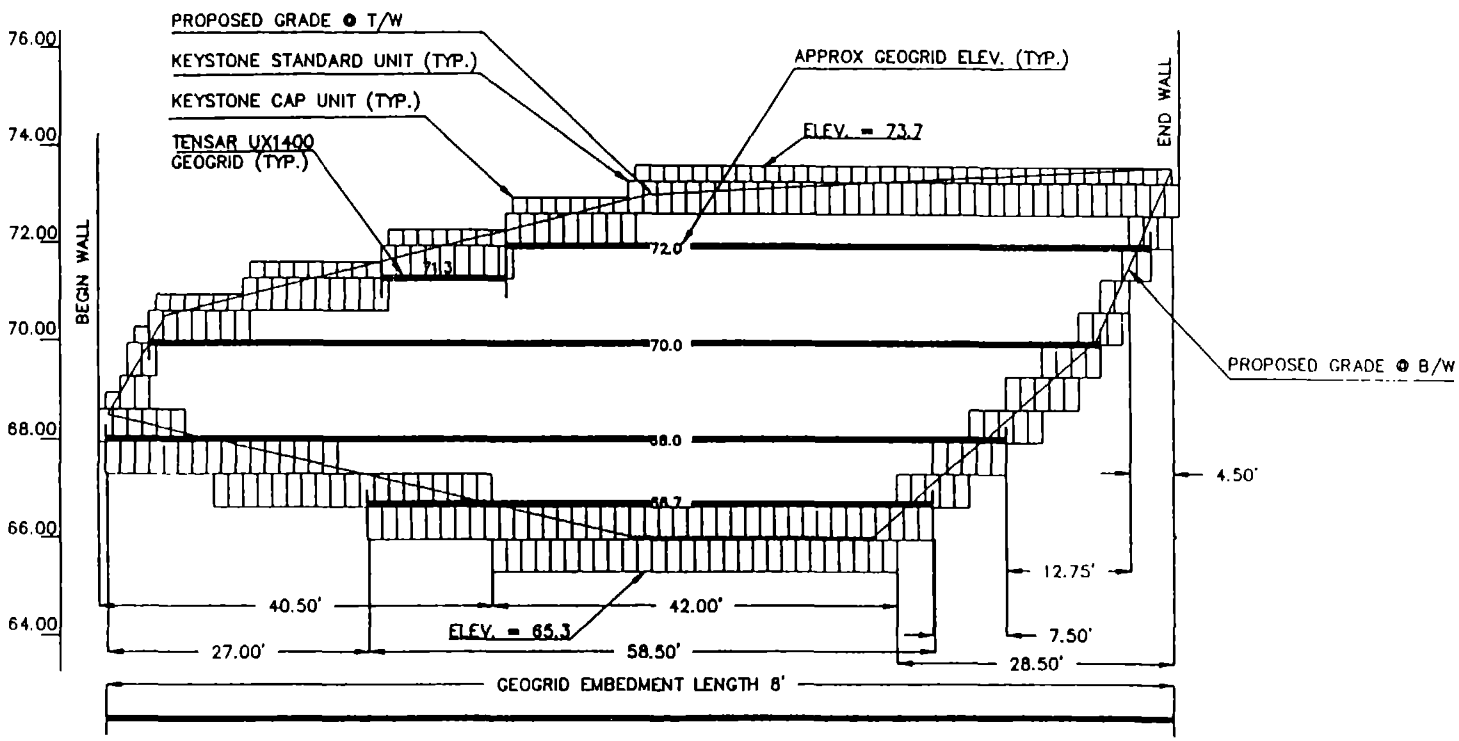


LEVELING PAD DETAIL

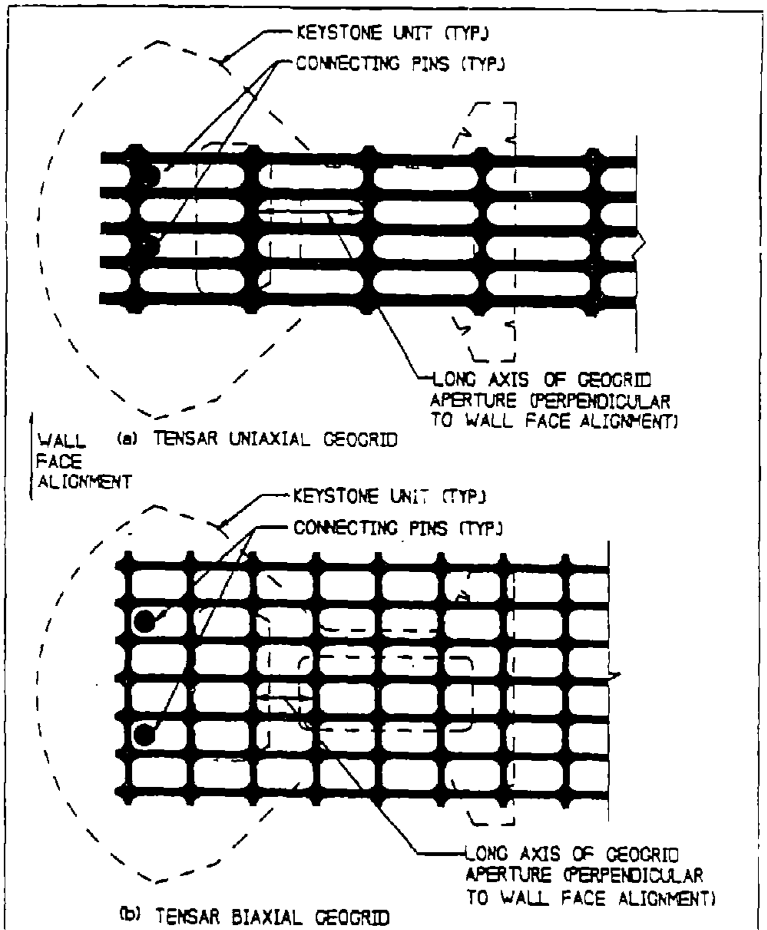
N.T.S.



STANDARD KEYSTONE UNIT DETAIL
NOT TO SCALE

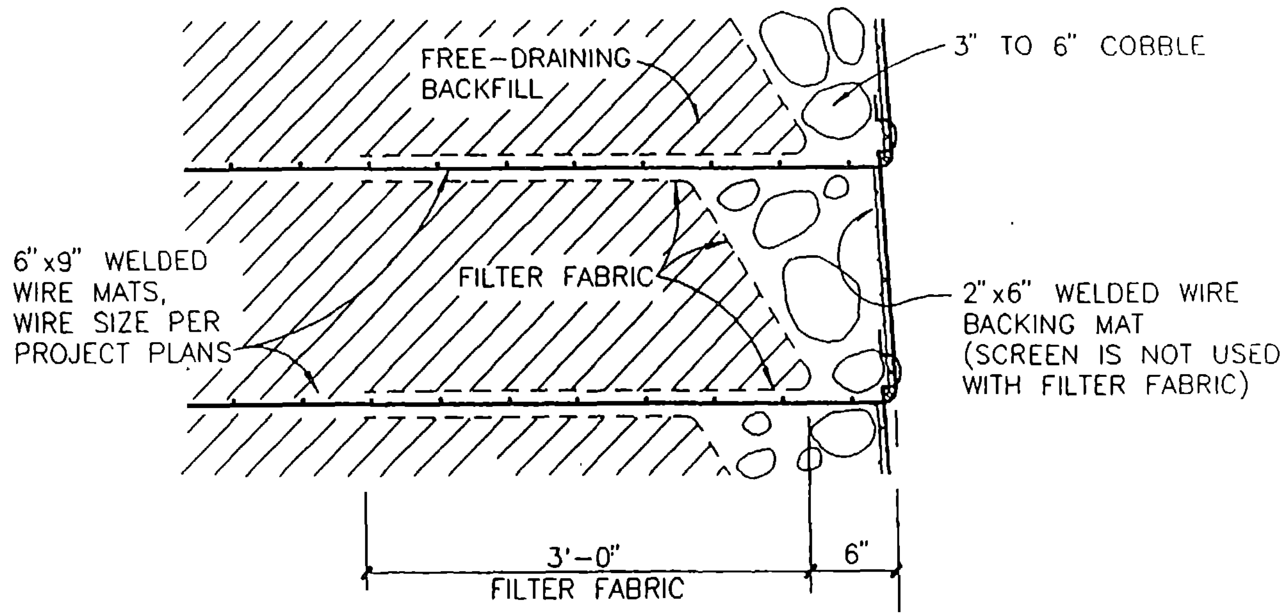


FRONT FACE ELEVATION VIEW WALL " A"

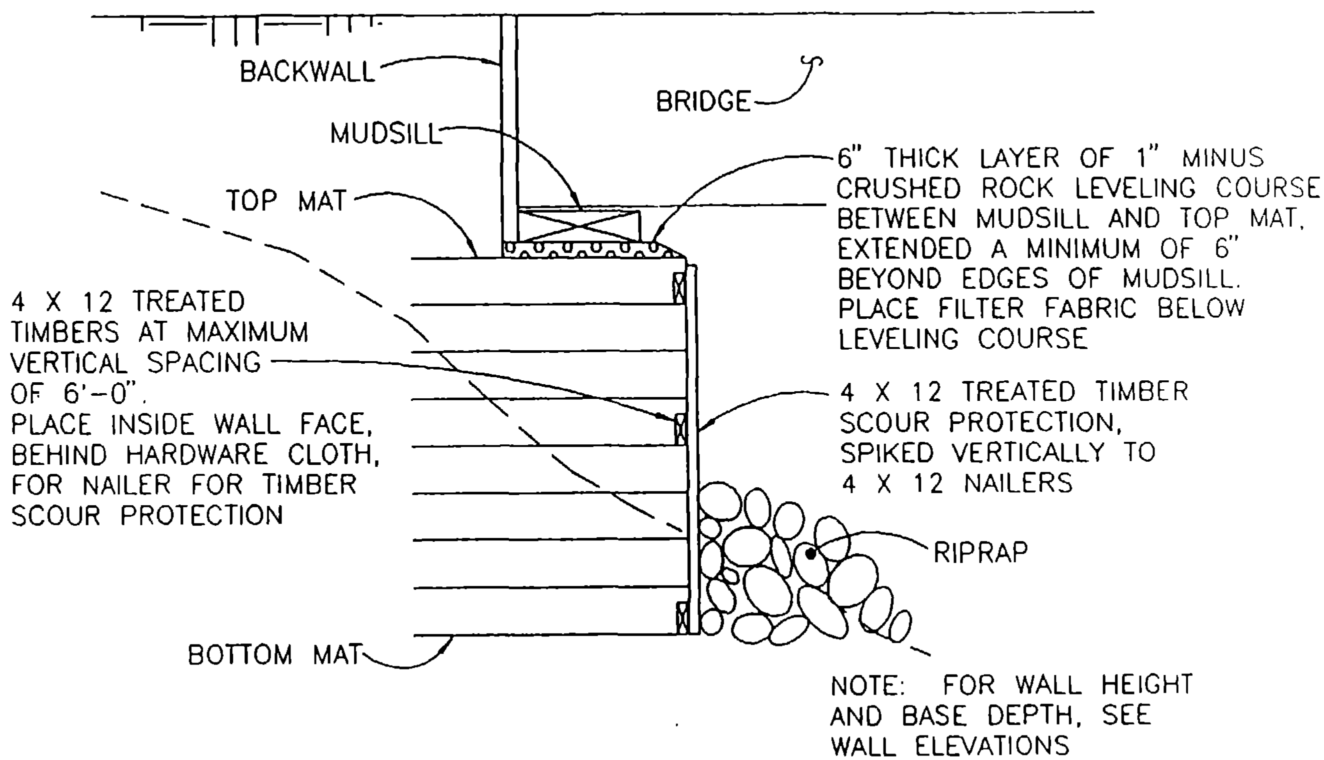


TENSAR GEOGRIDS AND THEIR ORIENTATION

BK11



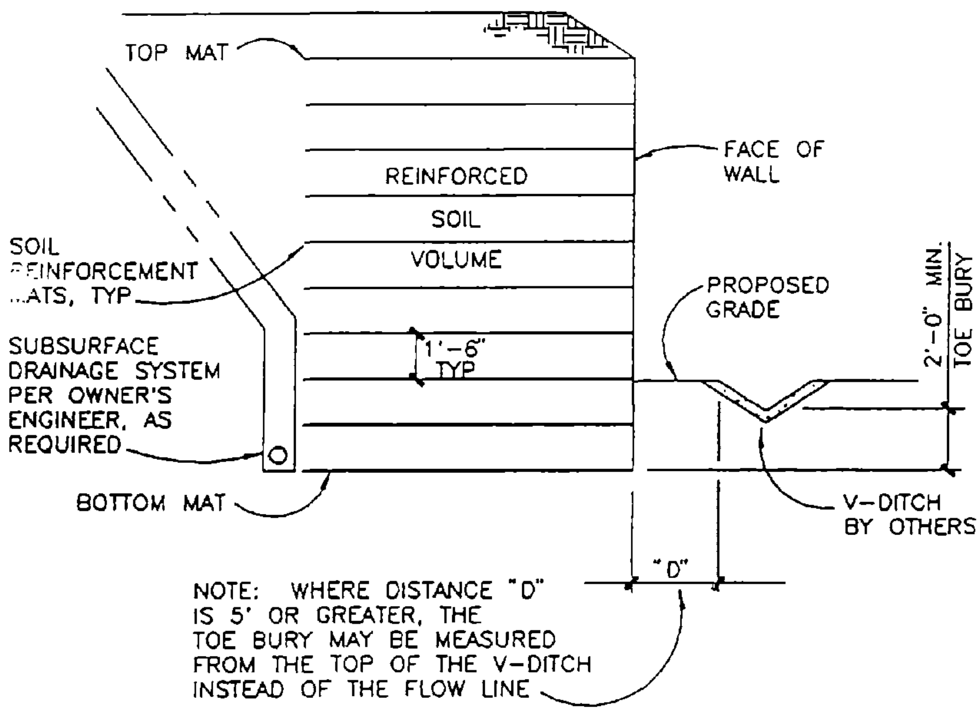
FILTER FABRIC DETAIL
1"=1'-0"



TIMBER FACING DETAIL

1" = 5'

DETAILS\TIMBER



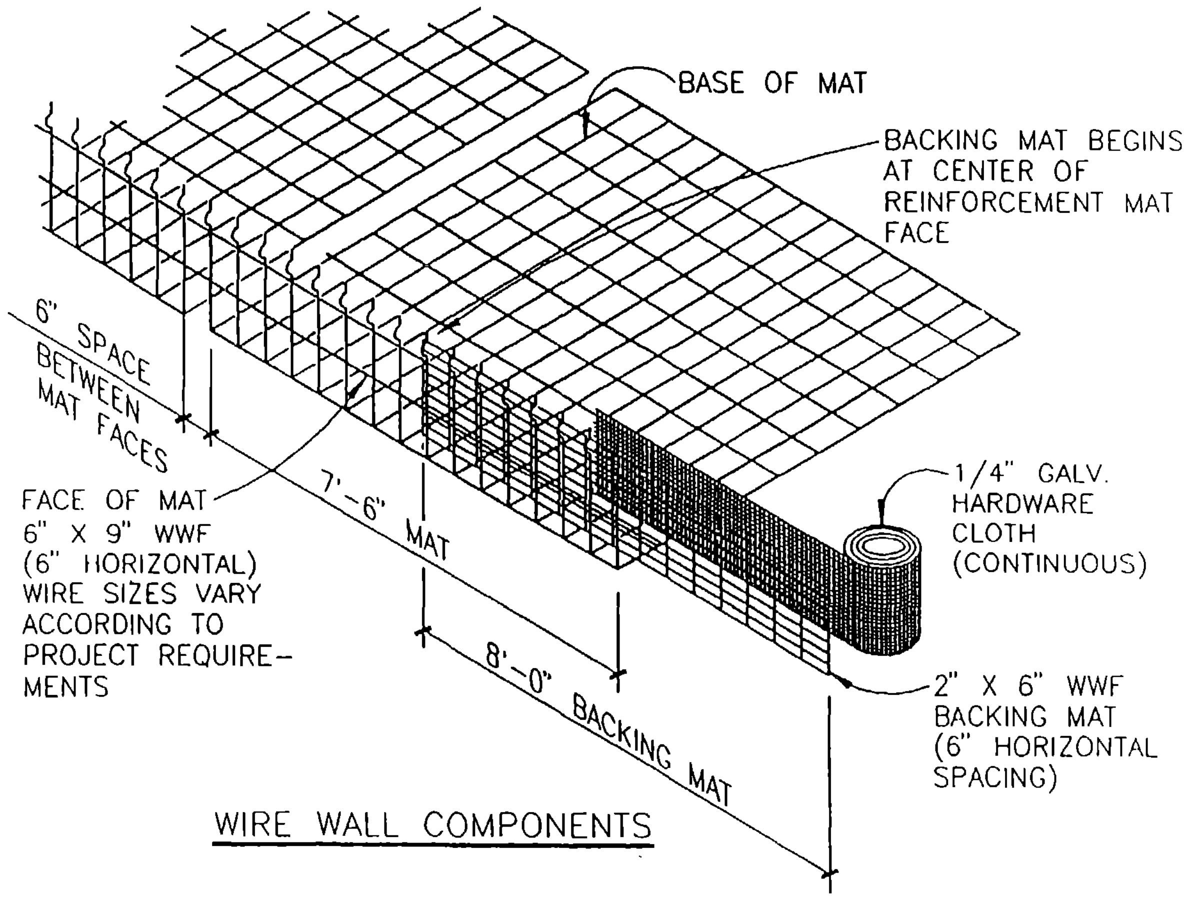
TYPICAL SECTION

1" = 5'

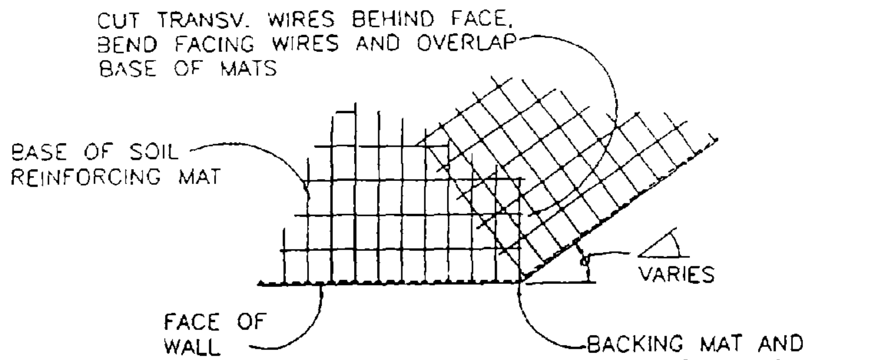
NOTE: THIS DETAIL ILLUSTRATES A STANDARD WELDED WIRE WALL WITH A V-DITCH AT THE TOE. ACTUAL APPLICATION OF THIS CONCEPT MAY REQUIRE DIFFERENT DIMENSIONS THAN THOSE SHOWN HEREON

V-DITCH DETAIL
STANDARD DRAWING

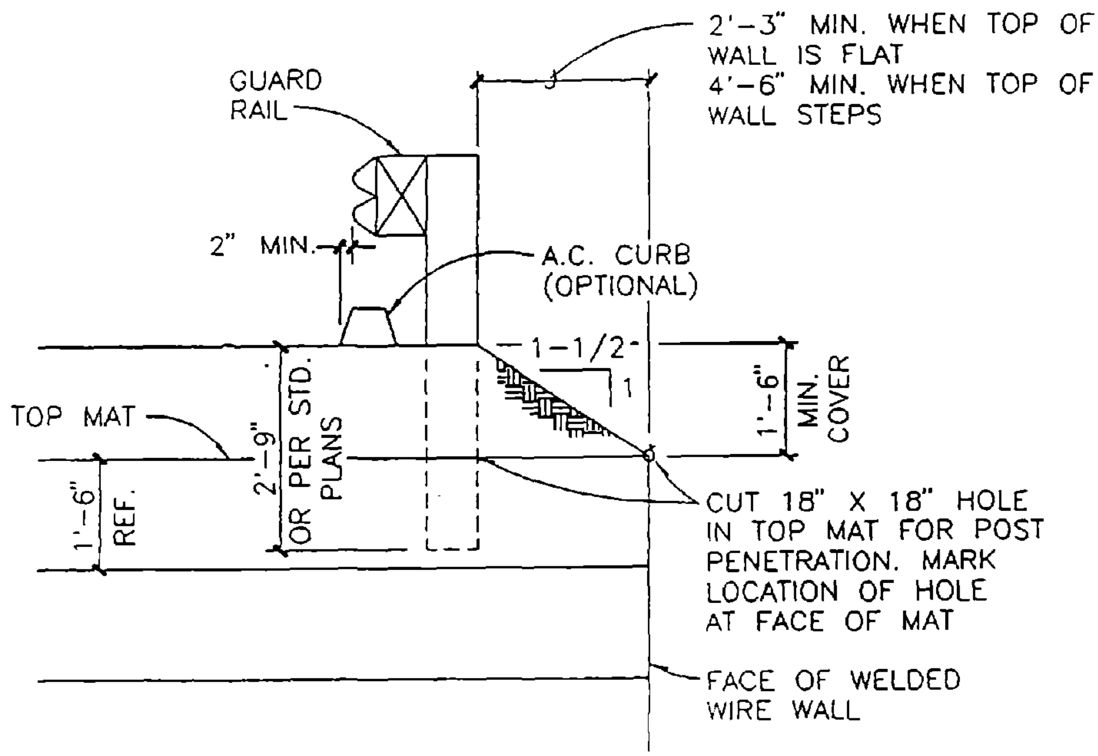
SCALE: NOTED	HILFIKER RETAINING WALLS
DATE: 14 OCT 92	P.O. Box 2012 Eureka, CA 95502-2012 707/443-5093 Fax 707/443-2891 Toll-Free 800/762-8962



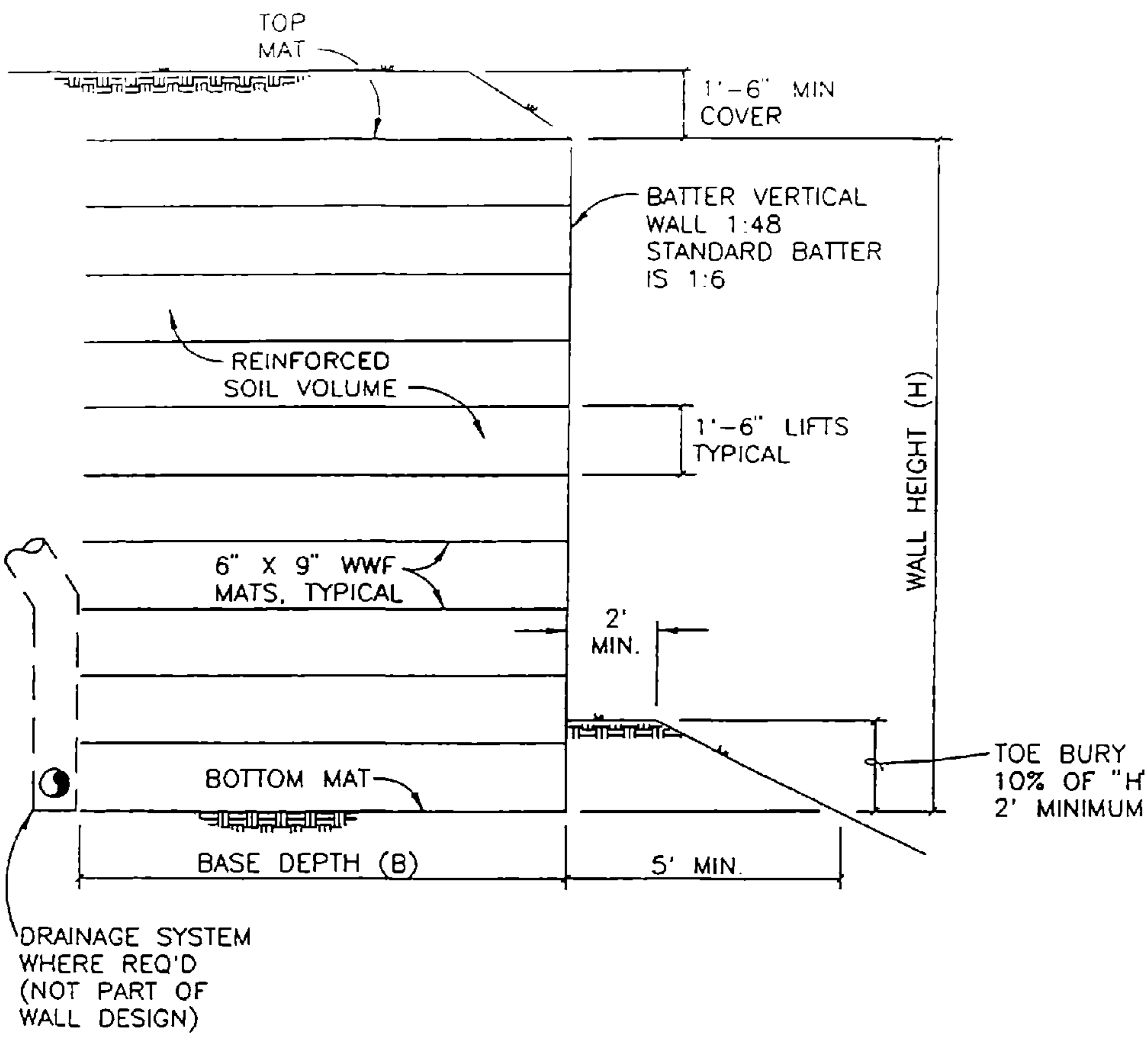
WIRE WALL COMPONENTS



CONVEX ANGLE
NOT TO SCALE

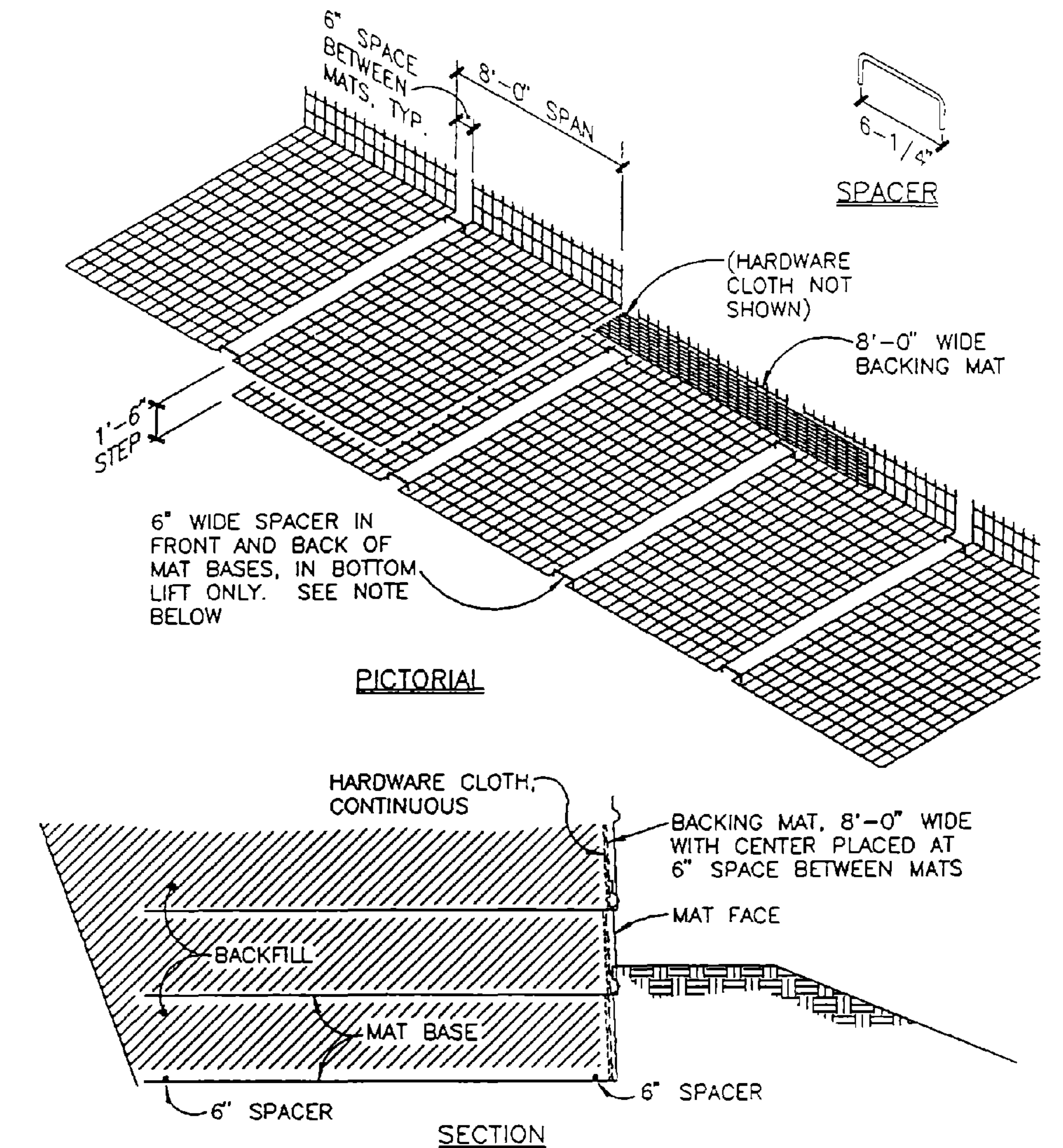


GUARD RAIL DETAIL
1" =2'
(FENCE DETAIL SIMILAR)



TYPICAL SECTION

NOT TO SCALE

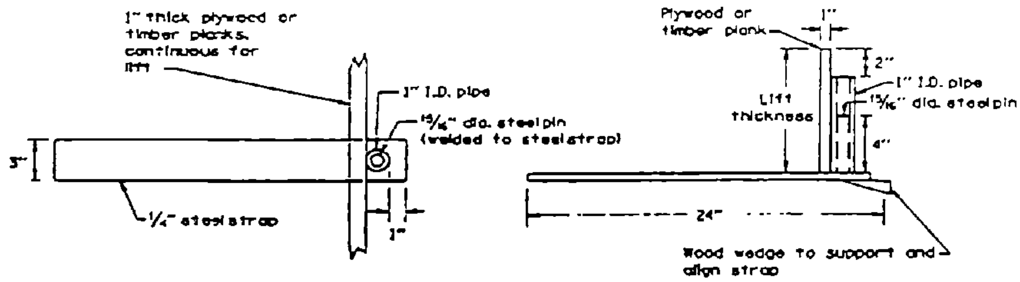


NOTE: WHERE MATS ARE PLACED ON ANGLE OR CURVE, ONLY ONE SPACER IS USED AT FRONT OF MAT BASE.

WELDED WIRE RETAINING WALLS
CONSTRUCTION DETAILS

SCALE: NONE
DATE: 25 NOV 91

HILFIKER RETAINING WALLS
P.O. Box 2012 Eureka, CA 95502-2012
707/443-5093 Fax 707/443-2891



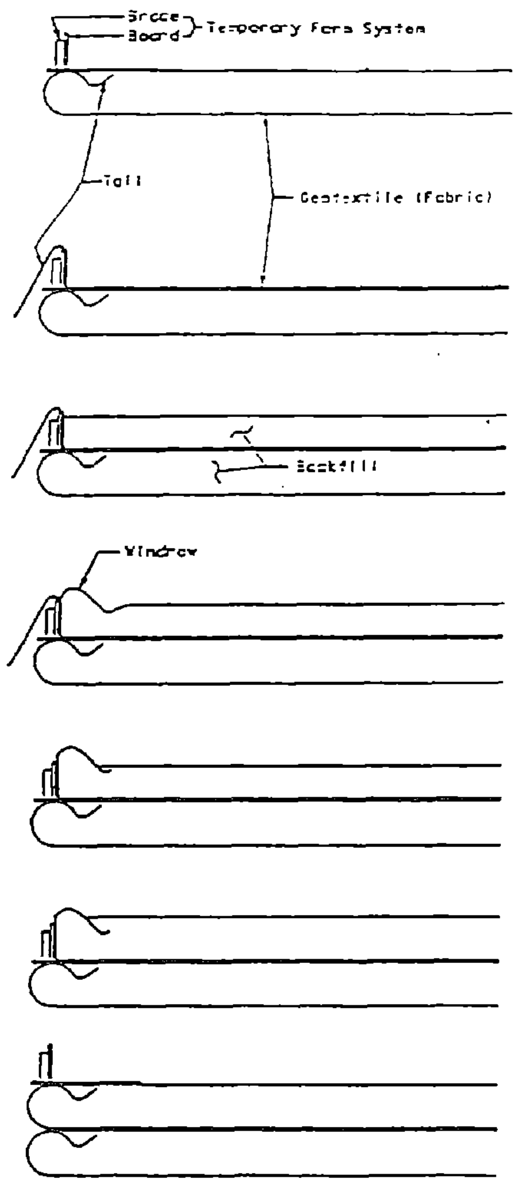
PLAN

ELEVATION

NOTE: Place straps at 4' to 6' centers along wall face.

(OPTIONAL) GEOTEXTILE TEMPORARY FORM SYSTEM DETAIL

(SEE SPECIAL PROVISIONS)



1. Set form on completed lift.

2. Unroll fabric and position so that a 3-foot wide "Tail" drapes over the form.

3. Place the backfill, until the backfill is up to half of the required vertical fabric layer spacing.

4. Place a windrow to slightly greater than full lift height against the form.

5. Place the fabric "Tail" over the windrow and lock into place with backfill.

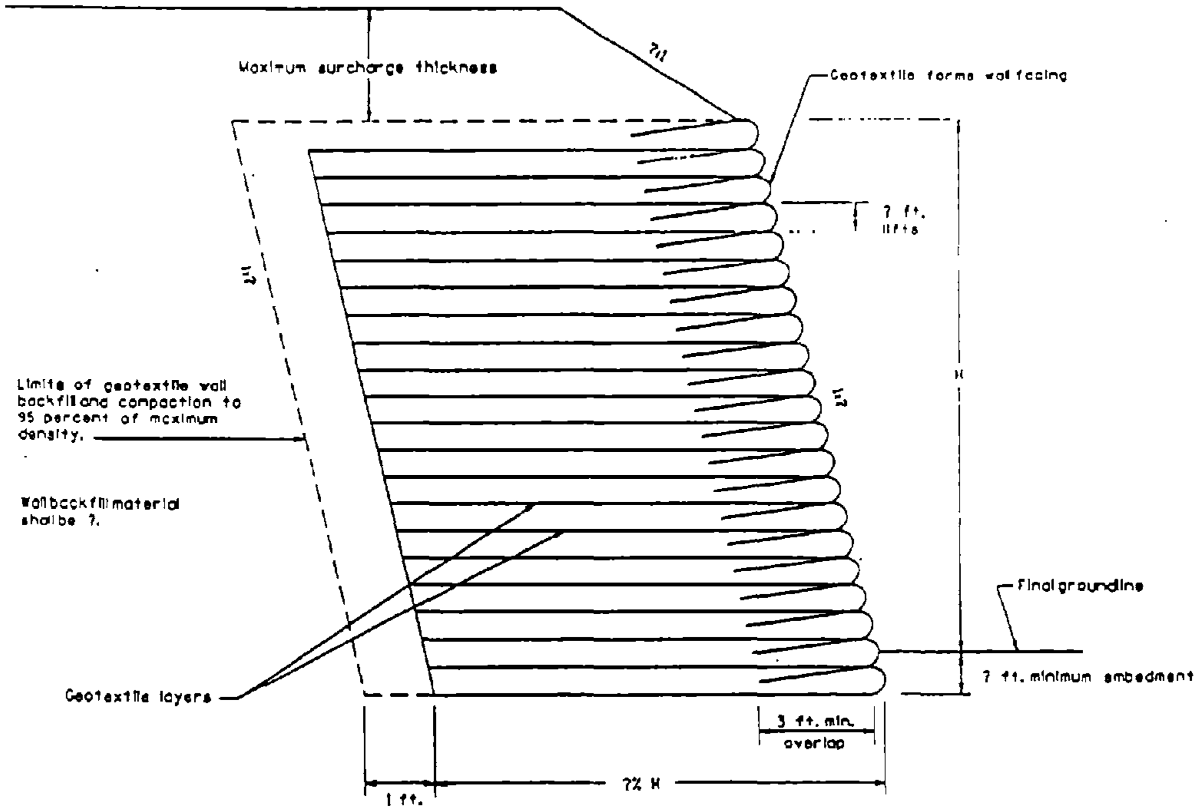
6. Complete backfilling until the compacted backfill layer thickness is equal to the required vertical fabric layer spacing.

7. Reset the form and repeat the sequence.

GEOTEXTILE WALL CONSTRUCTION PROCEDURE

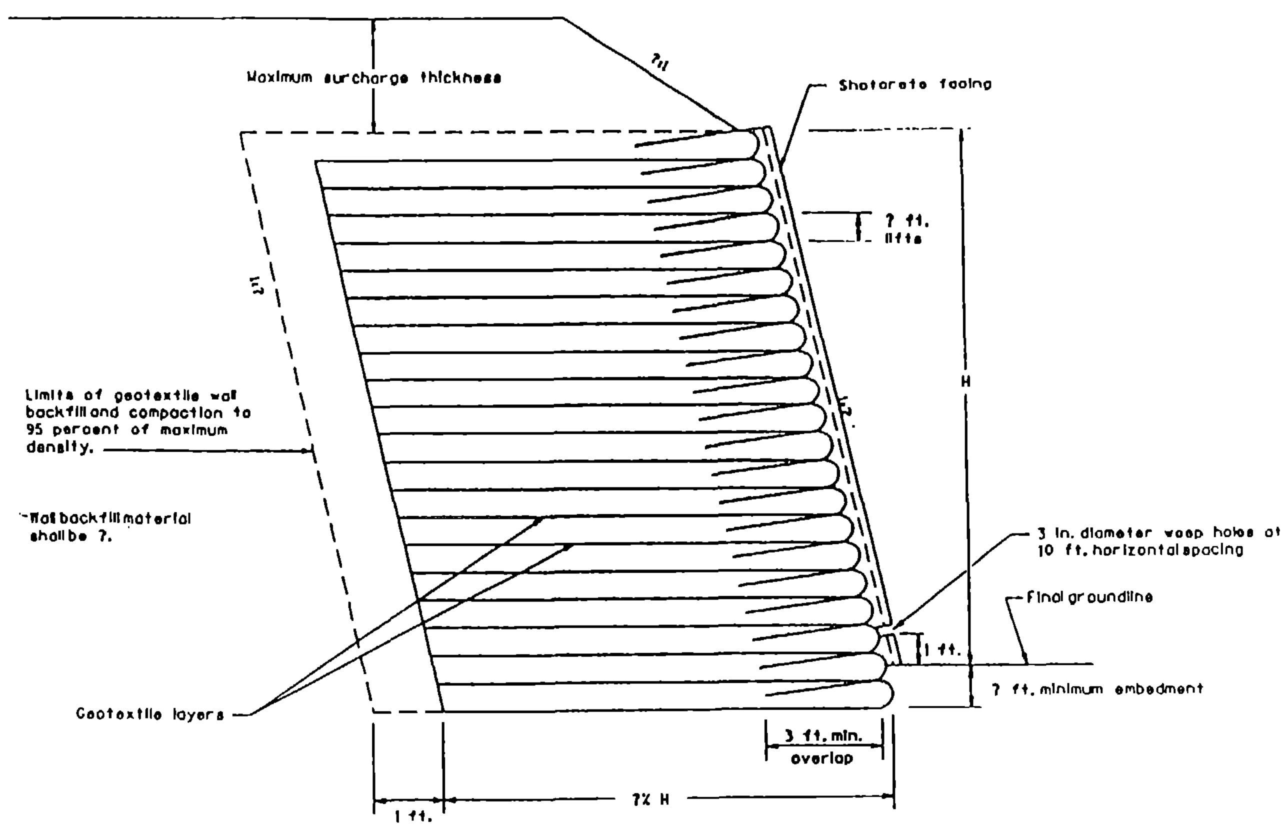
(SEE SPECIAL PROVISIONS)

NOTE: Method of temporary forming is optional.



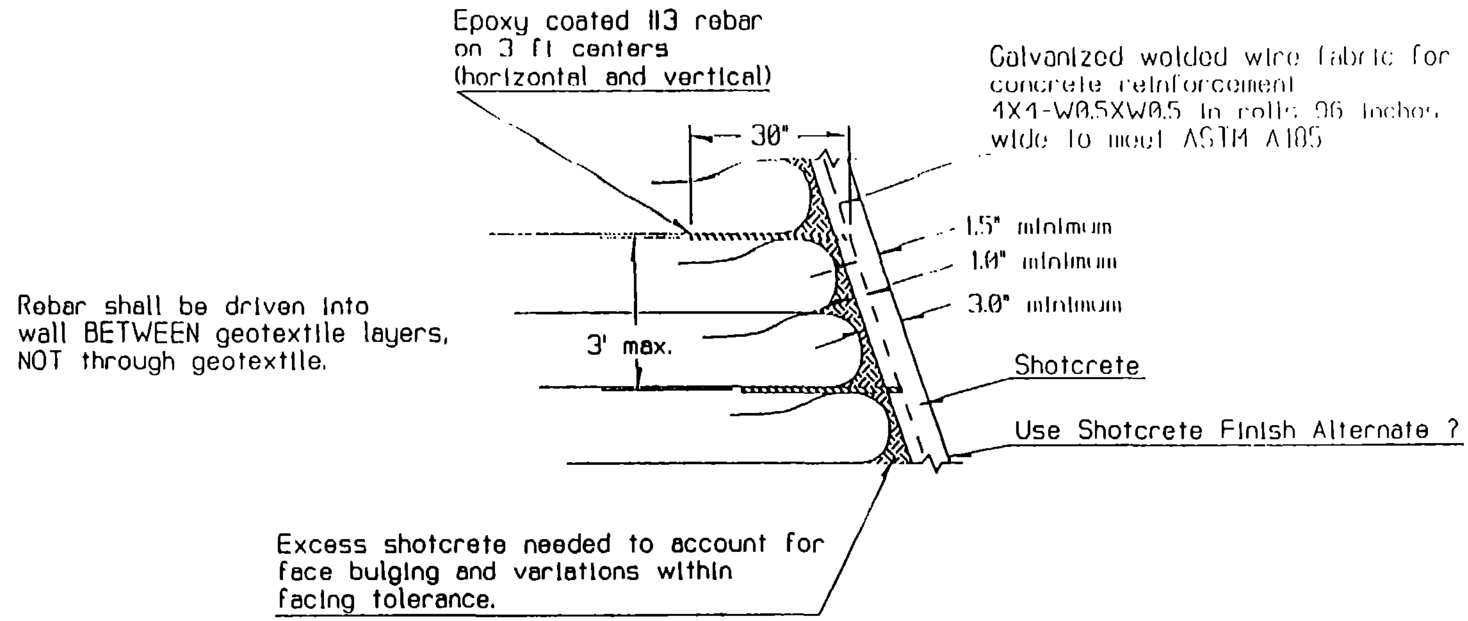
TEMPORARY GEOTEXTILE WALL SECTION

(SEE SPECIAL PROVISIONS)

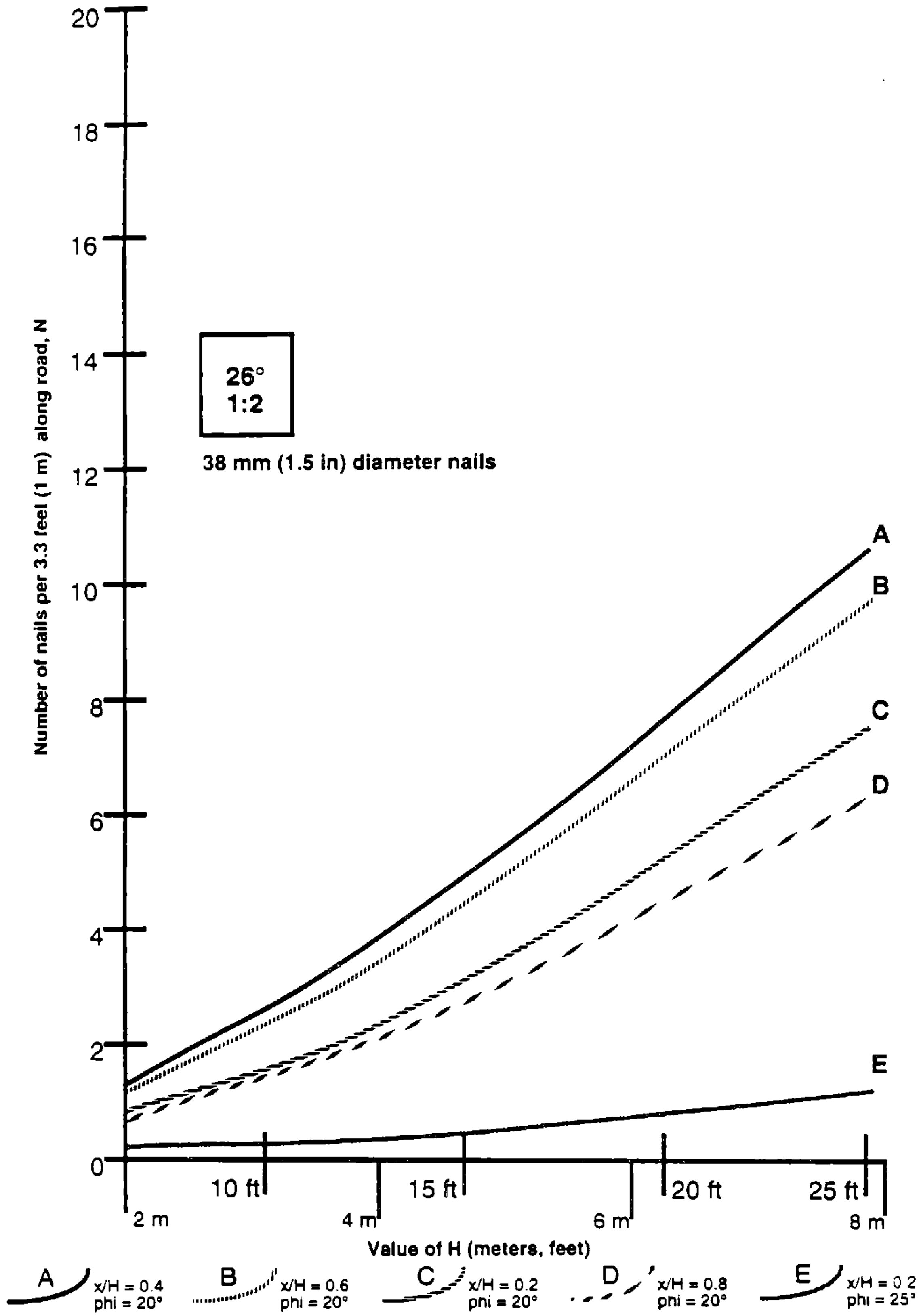


PERMANENT GEOTEXTILE WALL SECTION

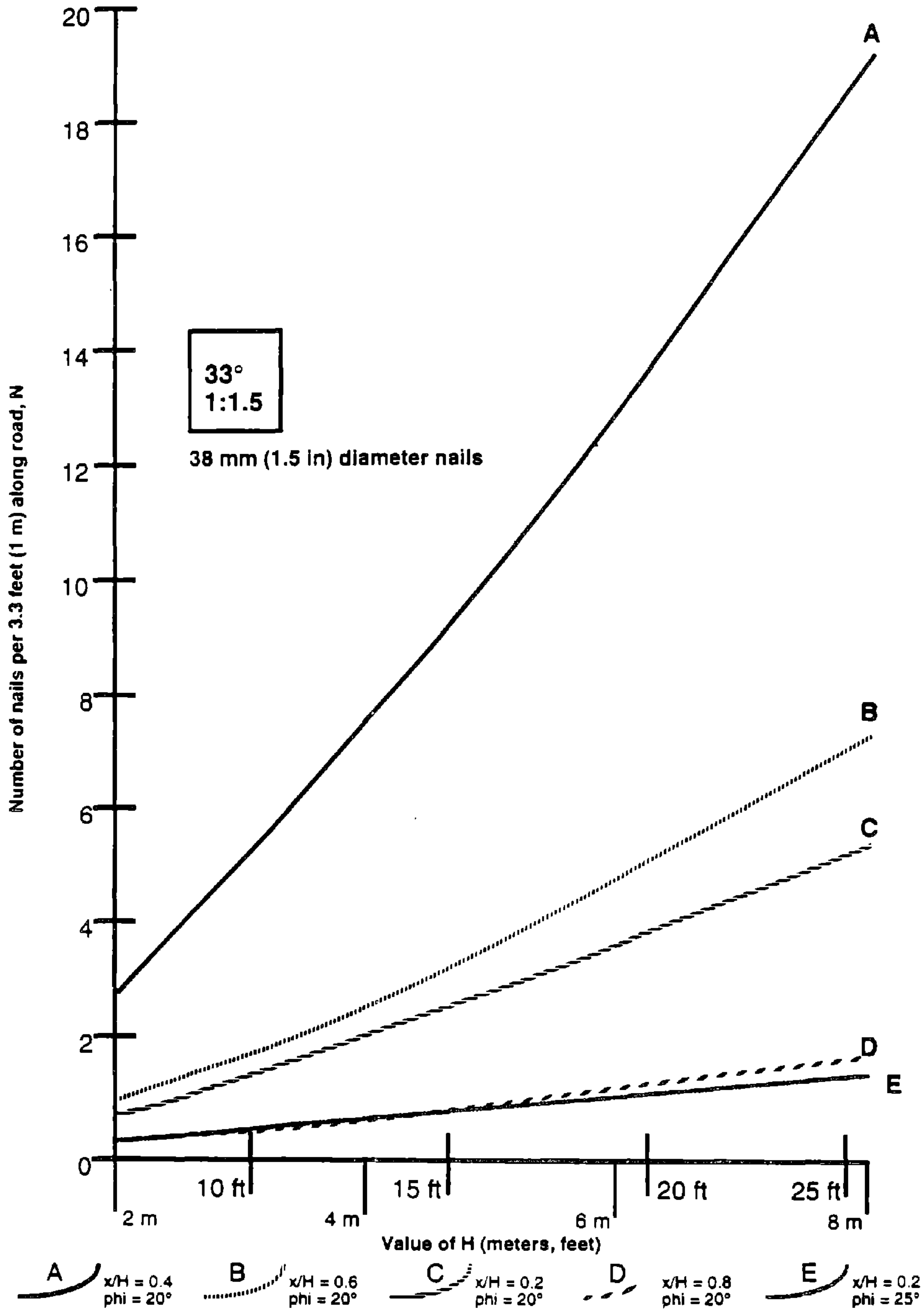
(SEE SPECIAL PROVISIONS)



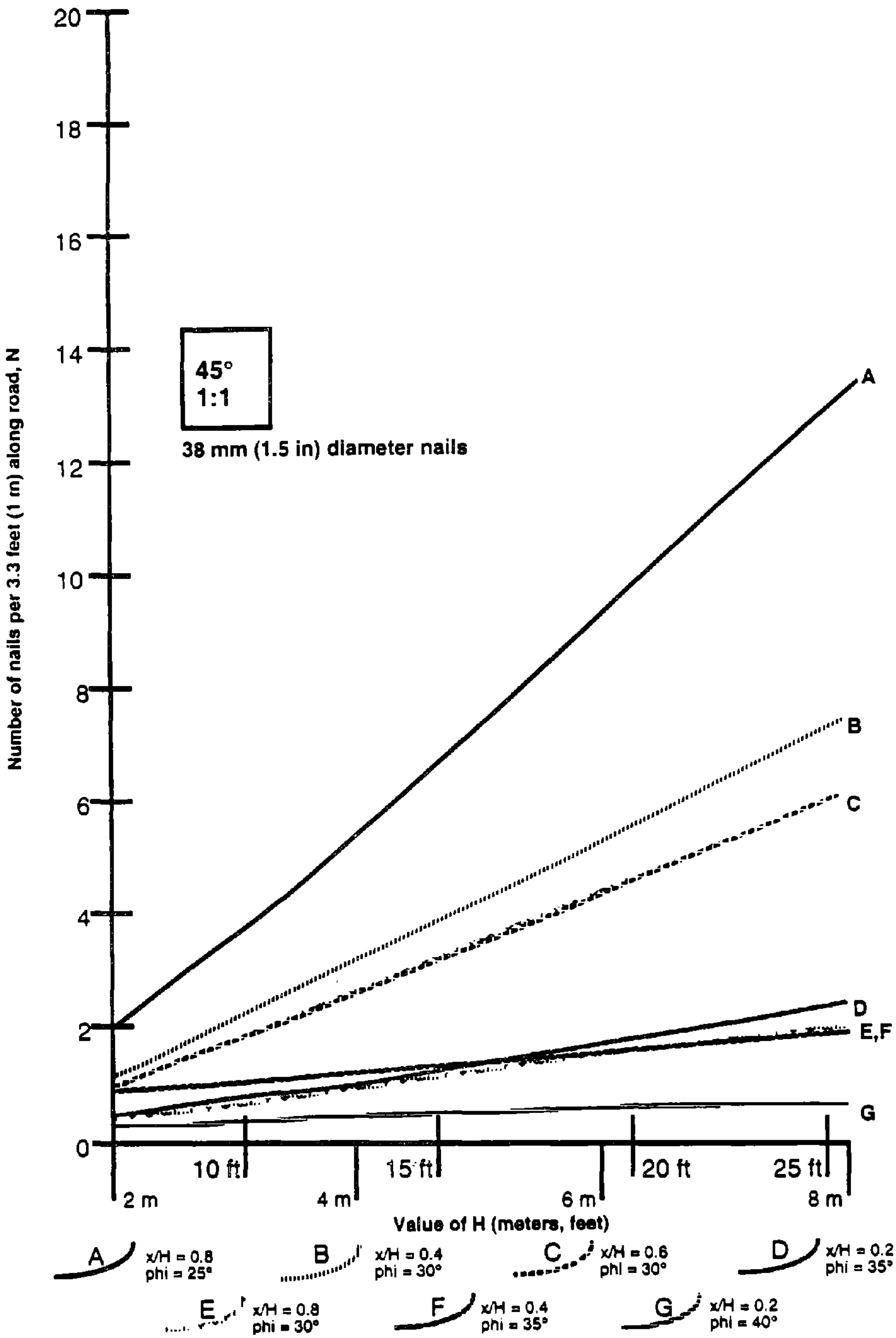
DETAIL: Mesh Reinforced Shotcrete Facing.



Number of nails required to stabilize a road shoulder for 1:2 (26°) slope.



Number of nails required to stabilize a road shoulder for 1:1.5 (33°) slope.



Number of nails required to stabilize a road shoulder for 1:1 (45°) slope.

FIELD DATA FORM FOR LAUNCHED SOIL NAILS

Road Name ____________________ Road No. ____ Data _____

Mile Post/Station ________ Location T. ___ R._____ Sec. ____

General Site Description: ______________________________________

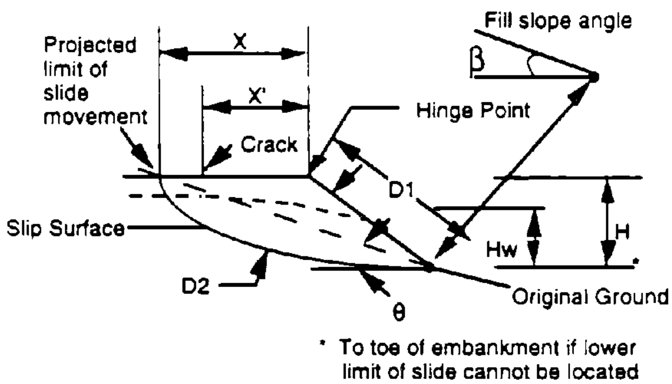
____

____

____

Repair Priority _________ Completed by: _________

(1-10 High)



Dimensions (FT) or (m)

X ________

X' ________

D1 ________

D2 ________

D3 ________

D4 ________

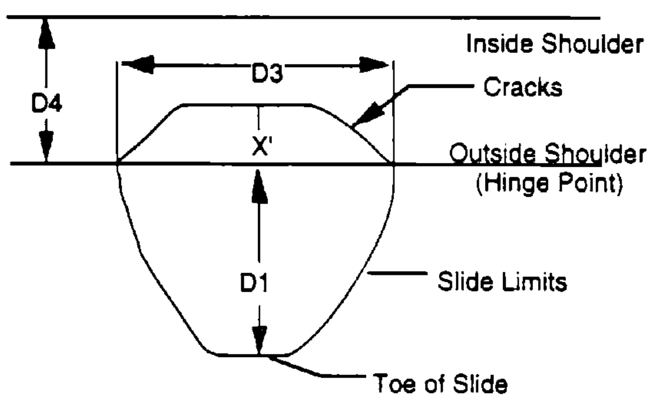
H* ________

Hw ________

B ________

θ ________

Cross-Section



Plan

SITE FACTOR CHECK LIST FOR LAUNCHED SOIL NAILS

SITE FACTORS	EVALUATION**						
	Low		Med		High		Remarks
Steepness of Slope (Slope Ratio)[1]	2:1		1.5:1		1:1		
Depth to Failure Surface, D1[1]	<5'		5'-10'		10'-15'		
Soil Moisture at time of slide[1]	Moist		Wet		Seep		
Decayed Logs or Slash Within Fill[1]	None		Some		Many		
Soil Type*[1]	Sand		Silt		Clay		**
Consequence of Add'l Failure(s)[2]	Low		Med		High		
Potential for Accident or Injury[2]	Low		Med		High		

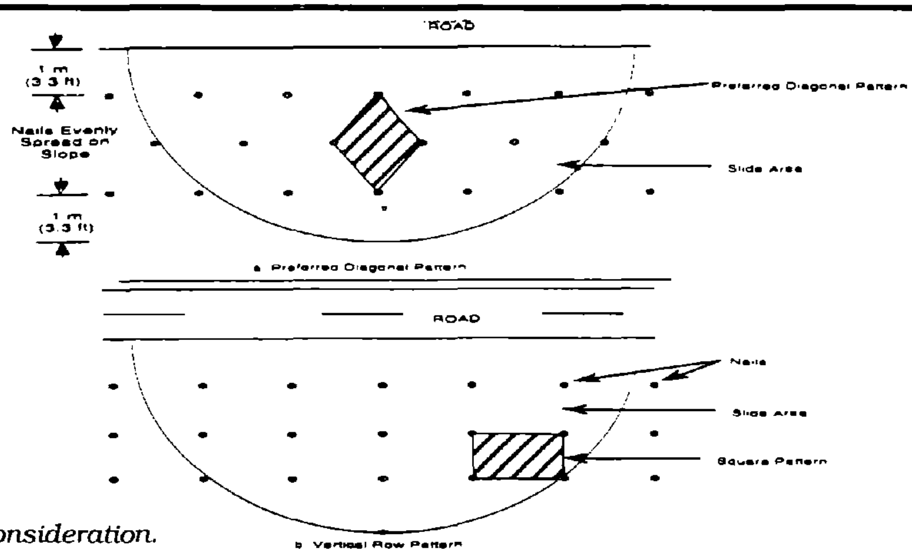
* Unified or AASHTO Classification

** Design charts were developed for medium Risk Level (Site Factor of Safety, $f_n = 1.1$)

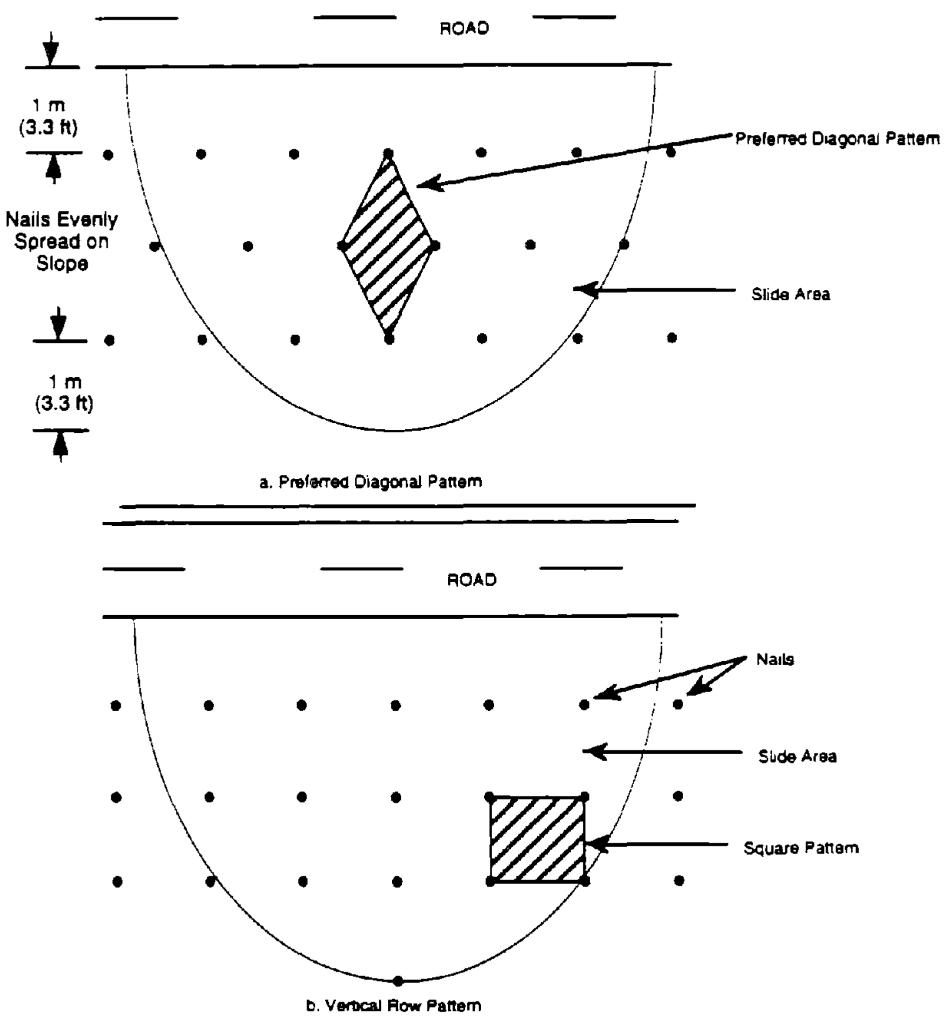
1 Relates to the probability of failure.

2 Relates to the consequence of failure.

SITE SPECIFIC PLAN AND (OR) CROSS-SECTION



Water table consideration.

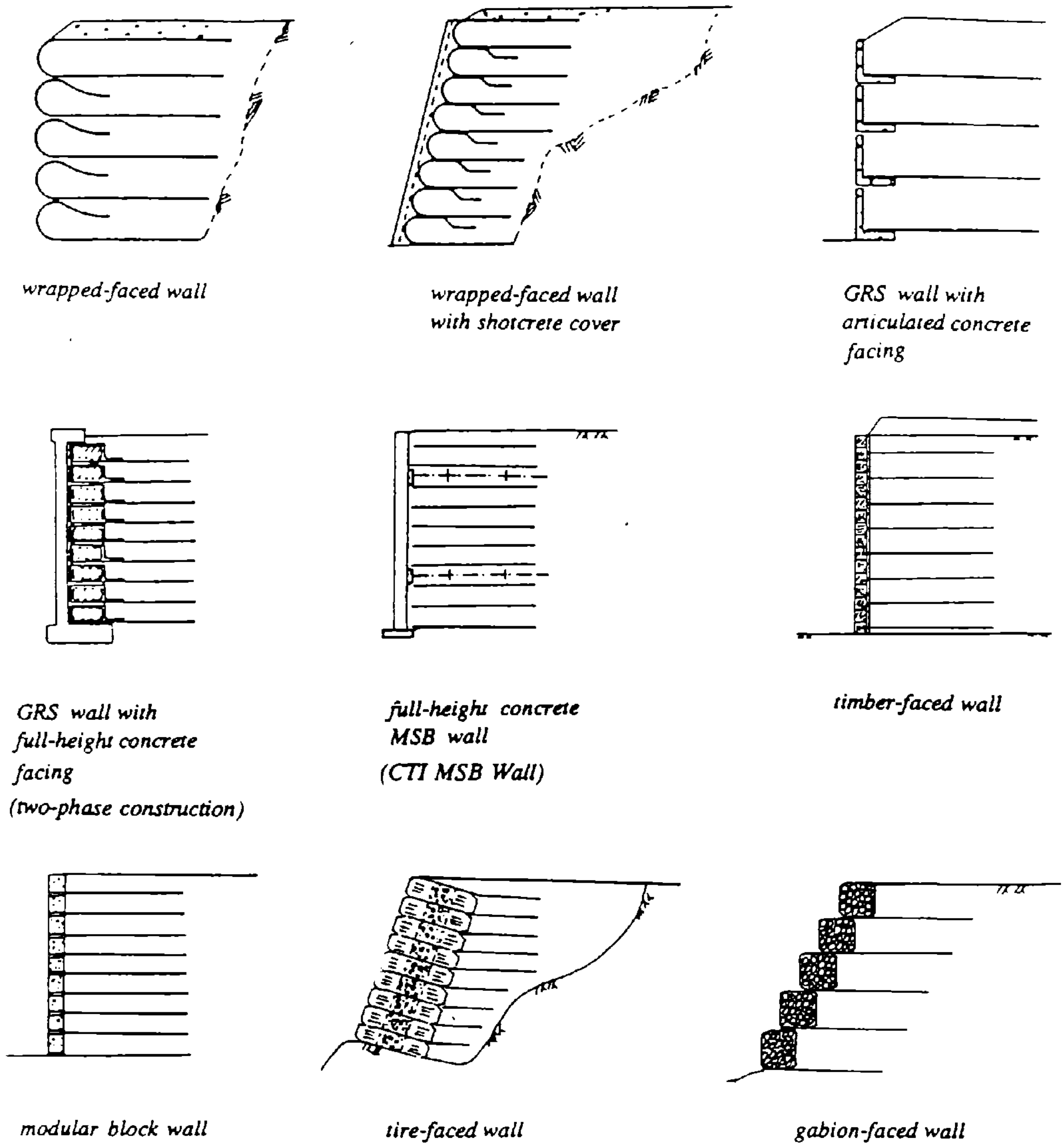


Nail pattern.

Appendix D
List of Manufacturers

The Reinforced Earth Company
VSL Corporation
Hilfiker Retaining Walls
Keystone Retaining Walls
Maccaferri Gabions Incorporated
The Tensar Corporation
Texel Incorporated
Geotechnical Fabrics Report Specifiers Guide
Schnabel Foundation Company
Terra Aqua Incorporated
Contech Construction Products
Permapost Products Company
Criblock Retaining Walls

Appendix E
Standard Designs



Geosynthetic-Reinforced Soil Walls with Different Facings

Simplified CTI (Colorado Transportation Institute) Design Method

A simplified CTI method is presented in this appendix. This design method can be used when the wall height is not greater than 15 feet, the foundation is firm, and the backfill is granular.

Design Procedure

Step 1. Establish wall profile and check design assumptions, which are:

- the wall height should be no more than 15 feet,
- the backfill shoulder be granular, and
- the foundation should be firm.

Step 2. Determine the moist unit weight, γ, and angle of internal friction, ϕ, of the backfill.

The angle of internal friction, ϕ, can be estimated or determined by appropriate direct shear or triaxial tests. The moist unit weight, γ, can be determined in a moist density test. Generally, a unit weight of 95 percent of the AASHTO T-99 maximum density is specified. However, other densities are also allowed, provided that the angle ϕ is consistent with that density.

Step 3. Determine the reinforcement length.

The reinforcement length, L, is

$$L = [\tan (45^\circ - \frac{\phi}{2}) + 0.2]\ H$$

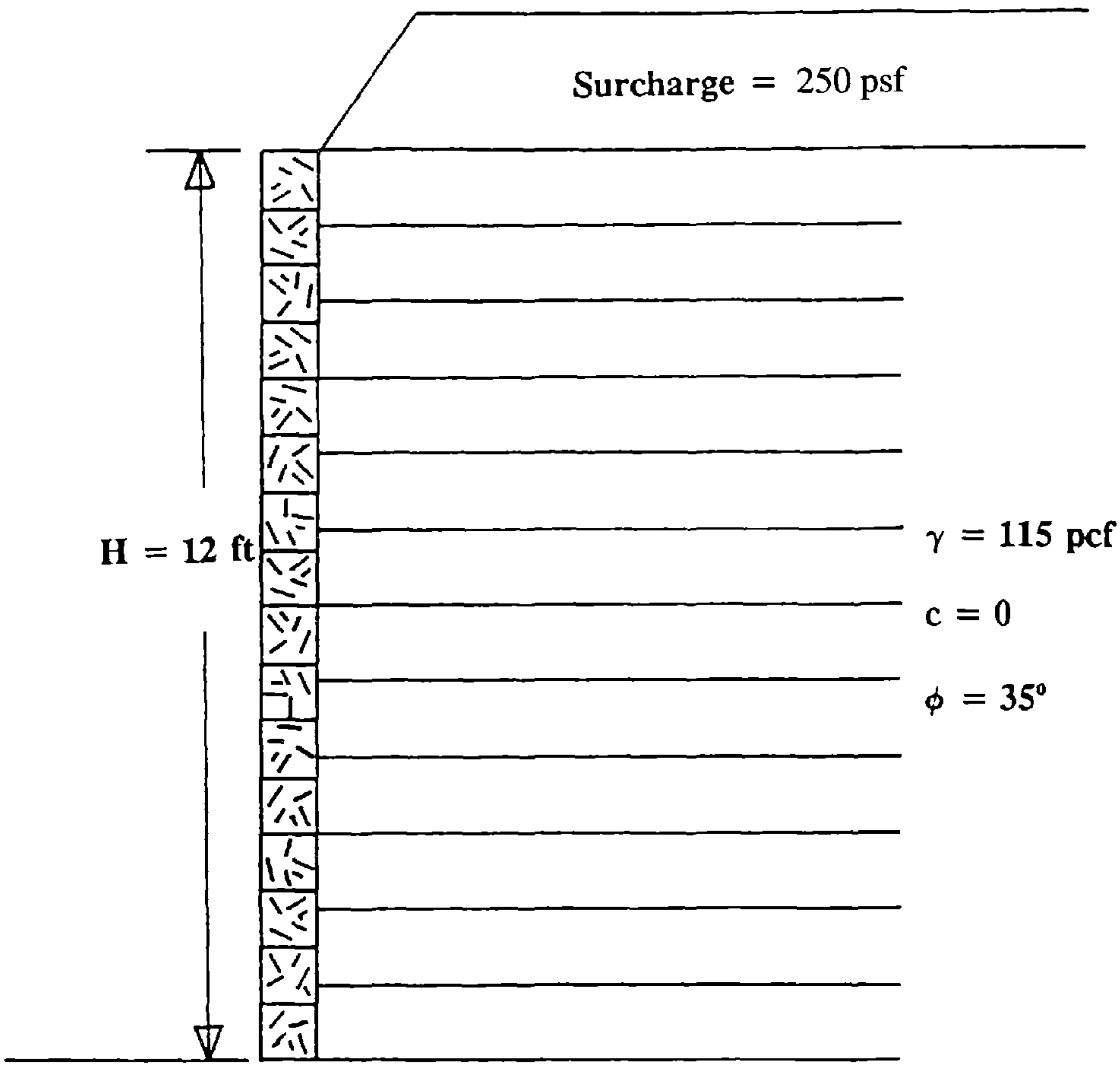
Step 4. Calculate the maximum tensile force in the reinforcement, T_{MAX}, which is calculated as

$$T_{MAX} = S(\gamma H + q)\ \tan^2(45^\circ - \frac{\phi}{2})$$

in which S = vertical spacing between reinforcement layers and H = wall height, and q is a surcharge load. For wall height not greater than 12 feet, a uniform spacing of 1 foot is generally selected for all reinforcement layers.

Step 5. Determine a design limit strain for the reinforcement, ϵ_d, that can be determined as

$$\epsilon_d = 1.25\ (\frac{\Delta_{MAX}}{H})$$



Cross-Section of a GRS Wall - Design Example for the Simplified CTI Method in which Δ_{MAX} is the maximum allowable lateral displacement of the wall, and H is the wall height.

Step 6. Select a geosynthetic.

A geosynthetic reinforcement can be selected so that

$$T(@\ design\ limit\ strain) \geq F_s T_{MAX}$$

and

$$T_{ult} \geq 3\ T(@\ design\ limit\ strain)$$

in which the factor of safety, F_s, should be at least 1.5. As is the case for any wall, external stability (i.e. bearing capacity and global stability) should be checked.

Design Example

Given Conditions

The cross-section of a CTI timber-faced, geosynthetic-reinforced soil retaining wall is shown in figure 2.20.

Characteristics of the wall are:

- uniform wall height: H = 12 feet,
- vertical wall,
- all geosynthetic reinforcement layers are to have the same length, and
- no embedment (wall constructed directly on the ground).

Characteristics of the backfill and retained earth are:

- the same soil is to be used for the backfill and retained earth (the earth behind the reinforced zone), and the density and moisture in the backfill and retained earth are similar;
- the soil is a sandy gravel (γ = 115 pcf, c = 0, and ϕ = 35° at 95 percent AASHTO T-99 maximum density); and
- surface and subsurface drainage are properly provided.

Table 2.4(a) Design Charts based on the Simplified CTI Method for Wall Height, H = 8 ft

Wall Height, H = 8 ft

Friction Angle of Backfill, ϕ (degree)		30°	32°	34°	36°	38°	40°	42°
Reinforcement Length, L (ft)		6.2	6.0	5.9	5.7	5.5	5.3	5.2
$T_{\epsilon d}$ (lb/ft)	s = 8"	380	350	320	290	270	250	220
	s = 12"	570	520	480	440	400	370	340
	s = 16"	750	690	640	590	540	490	450
T_{uh} (lb/ft)	s = 8"	1130	1040	960	880	810	740	670
	s = 12"	1700	1560	1440	1320	1210	1110	1010
	s = 16"	2260	2090	1920	1760	1610	1470	1340

Note:

1. Design limit strain, $\epsilon_d = (1.3 * \Delta_{max})\%$, where Δ_{max} is the maximum allowable lateral wall movement (in inches)
2. $T_{\epsilon d}$: required force/width of geosynthetic reinforcement at design limit strain (ϵ_d)
3. T_{uh} : minimum required ultimate strength of geosynthetic reinforcement
4. s : vertical spacing of reinforcement
5. Surcharge pressure = 250 psf (increase $T_{\epsilon d}$ and T_{uh} each by 22 % for every additional 250 psf surcharge pressure

Characteristics of the foundation soil:

- the standard penetration blow count is 40 uniform to 35 feet below ground surface, and
- a deep water table.

Characteristics of the loading are:

- vertical surcharge uniformly distributed over the crest, q = 250 psf; and
- there is no concern of seismic loading.

Performance limit is:

- maximum allowable lateral movement of the wall is 3 percent of the wall height, i.e., Δ_{MAX} = (12 feet x inch/feet.) (3 percent) = 4.3 inches.

Design Computation

Step 1. Establish wall profile and check design assumptions.

The cross-section of the wall is as depicted in figure 2.20. The design assumptions are verified, including wall height less than 15 feet, granular backfill, and a firm foundation.

Step 2. Determine γ and ϕ of the backfill.

At 95 percent of the AASHTO T-99 maximum density, γ = 115 pcf and ϕ = 35°.

Step 3. Determine the reinforcement length.

$$L = [\tan(45° - \frac{35°}{2}) + 0.2]12 = 8.6 \text{ ft}$$

Step 4. Calculate the maximum tensile force.

$$T_{MAX} = (1)(115 \times 12 + 250)\tan^2(45° - \frac{35°}{2}) = 442(\text{lb/ft})$$

Step 5. Determine the design limit strain.

$$\epsilon_d = 1.25 \ (4.3/12) \ /12 = 3.75\%$$

Step 6. Select geosynthetic.

The geosynthetic selected must have the following tensile properties:

$$T \ (@ \ 3.75\% \ \text{strain}) \geq 1.5(442) = 660 \ (\text{lb/ft})$$

$$T_{ult} \geq 3(660) = 1{,}980 \ (\text{lb/ft})$$

Design Charts

The design charts shown in table 2.4 are based on the simplified CTI design method. The design charts can be used when the wall height is not greater than 15 feet, the foundation is firm, the backfill is granular, and a "quick" design is desired and warranted.

To use the charts for design of a GRS wall, the following four parameters are needed:

- the wall height, H;
- the friction angle of the backfill, ϕ;
- the vertical spacing of reinforcement, s; and
- the maximum allowable lateral wall movement, Δ_{MAX}.

The design charts allow one to determine:

- the minimum reinforcement length, L.
- the minimum force/width of the geosynthetic reinforcement at the design limit strain, $T_{\epsilon d}$ (note: the design limit strain for each wall height is shown in the footnotes of table 2.4).
- the minimum required ultimate strength of the geosynthetic reinforcement, T_{ult}.

Design Example

H = 12 feet, ϕ = 35°, and Δ_{MAX} = (3%) * H.

Use example (c) table 2.4 for wall height H = 12 feet.

$$\Delta_{MAX} = (3\%)\ H = (3\%)(12)(12) = 4.3 \text{ inches}$$

The design limit strain $\epsilon_d = (0.9 \times \Delta_{MAX})\% = (0.9 \times 4.3)\% = 3.9\%$

For ϕ = 35° (interpolate between ϕ = 34° and ϕ = 36°) and s = 12 inches:

$$L = (8.8 + 8.5) / 2 = 8.65 \text{ (ft)}$$

$$T_{\epsilon d} = T(@\ 3.9\%) = (670 + 610) / 2 = 640 \text{ (lb/ft)}$$

$$T_{ult} = (2{,}000 + 1{,}830) / 2 = 1{,}970 \text{ (lb/ft)}$$

Wrapped Face, Geotextile-Reinforced Wall

Construction Procedure

The following construction sequence has been proposed and illustrated in figure 3.1:

Step 1. Level wall site, and place a series of L-shaped forms of height slightly greater than the lift thickness on the ground surface. The L-shaped form is composed of a series of metal L brackets and a continuous wooden brace board running along the wall face (see figure 3.2).

Table 2.4(b) Design Charts based on the Simplified CTI Method for Wall Height, H = 10 ft

Wall Height, H = 10 ft

Friction Angle of Backfill, ϕ (degree)		30°	32°	34°	36°	38°	40°	42°
Reinforcement Length, L (ft)		7.8	7.5	7.3	7.1	6.9	6.7	6.5
T_{rd} (lb/ft)	s = 8"	450	420	380	350	320	290	270
	s = 12"	680	620	570	530	480	440	400
	s = 16"	900	830	760	700	640	590	540
T_{ult} (lb/ft)	s = 8"	1350	1240	1150	1050	960	880	800
	s = 12"	2030	1870	1720	1580	1450	1320	1200
	s = 16"	2700	2490	2290	2100	1930	1760	1610

Note:

1. Design limit strain, $\epsilon_d = (1.05 * \Delta_{max})\%$, where Δ_{max} is the maximum allowable lateral wall movement (in inches)
2. T_{rd} : required force/width of geosynthetic reinforcement at design limit strain (ϵ_d)
3. T_{ult} : minimum required ultimate strength of geosynthetic reinforcement
4. s : vertical spacing of reinforcement
5. Surcharge pressure = 250 psf (increase T_{rd} and T_{ult} each by 18 % for every additional 250 psf surcharge pressure

Table 2.4(c) Design Charts based on the Simplified CTI Method for Wall Height, H = 12 ft

Wall Height, H = 12 ft

Friction Angle of Backfill, ϕ (degree)		30°	32°	34°	36°	38°	40°	42°
Reinforcement Length, L (ft)		9.3	9.1	8.8	8.5	8.3	8.0	7.7
T_{rd} (lb/ft)	s = 8"	520	480	440	410	370	340	310
	s = 12"	790	720	670	610	560	510	470
	s = 16"	1050	970	890	820	750	680	620
T_{ult} (lb/ft)	s = 8"	1570	1480	1330	1220	1120	1020	930
	s = 12"	2360	2170	2000	1830	1680	1540	1400
	s = 16"	3140	2890	2660	2450	2240	2050	1870

Note:

1. Design limit strain, $\epsilon_d = (0.9 * \Delta_{max})\%$, where Δ_{max} is the maximum allowable lateral wall movement (in inches)
2. T_{rd} : required force/width of geosynthetic reinforcement at design limit strain (ϵ_d)
3. T_{ult} : minimum required ultimate strength of geosynthetic reinforcement
4. s : vertical spacing of reinforcement
5. Surcharge pressure = 250 psf (increase T_{rd} and T_{ult} each by 16 % for every additional 250 psf surcharge pressure

Table 2.4(d) Design Charts based on the Simplified CTI Method for Wall Height, H = 15 ft

Wall Height, H = 15 ft

Friction Angle of Backfill, ϕ (degree)		30°	32°	34°	36°	38°	40°	42°
Reinforcement Length, L (ft)		11.7	11.3	11.0	10.6	10.3	10.0	9.7
T_{ed} (lb/ft)	s = 8"	630	580	540	490	450	410	380
	s = 12"	950	880	810	740	680	620	570
	s = 16"	1270	1170	1070	990	900	830	750
T_{ult} (lb/ft)	s = 8"	1900	1750	1610	1480	1360	1240	1130
	s = 12"	2850	2630	2420	2220	2030	1860	1700
	s = 16"	3800	3500	3220	2960	2710	2480	2260

Note:

1. Design limit strain, $\epsilon_d = (0.7 * \Delta_{max})\%$, where Δ_{max} is the maximum allowable lateral wall movement (in inches)
2. T_{ed} : required force/width of geosynthetic reinforcement at design limit strain (ϵ_d)
3. T_{ult} : minimum required ultimate strength of geosynthetic reinforcement
4. s : vertical spacing of reinforcement
5. Surcharge pressure = 250 psf (increase T_{ed} and T_{ult} each by 13 % for every additional 250 psf surcharge pressure

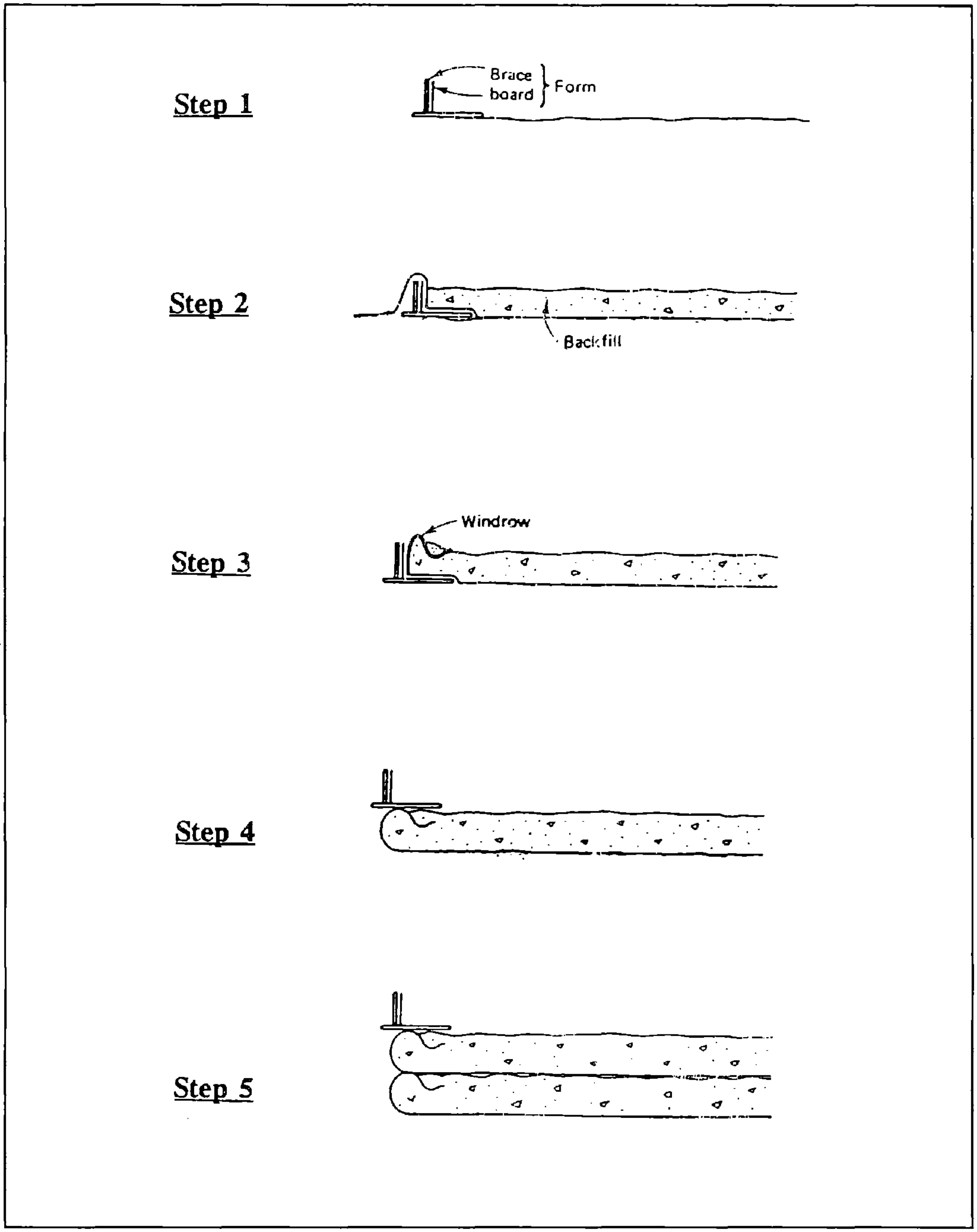


Figure 3.1.—Construction of USFS wrapped-faced wall.

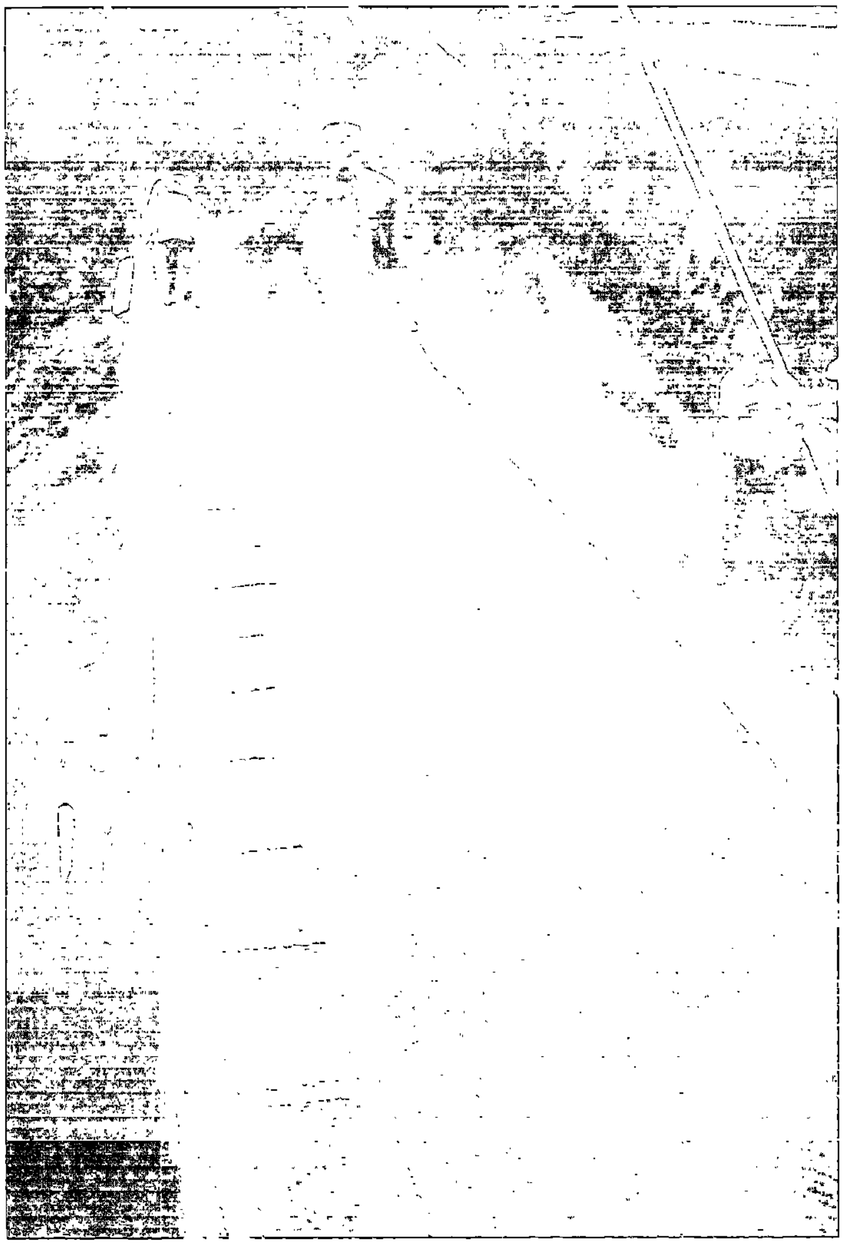


Figure 3.2.—Erection of a USFS wrapped-faced wall with the use of an L-shaped form (cou^rt^esy Gorden Keller, USDA Forest Service).

Step 2. Place a layer of geotextile sheet on the surface and position it so that approximately 3 feet of the geotextile extends over the top of the form and hangs loose. Backfill to approximately three-fourths of the lift thickness and compact with conventional light earth-moving equipment.

Step 3. Make a windrow 12 to 24 inches from the wall face with a road grader or with hands, and fold the loose end of the geotextile ("tail") back over the L-shaped form into the windrow.

Step 4. Backfill and compact the remaining lift thickness. Remove the form and reset it on the top of the first lift.

Step 5. Repeat steps 2 through 4 for the subsequent lifts until the planned height is reached. For the final layer, the tail length must be at least 6 feet.

Step 6. Cover the exposed face of the wall with bituminous emulsions, other asphalt products, or gunite (shotcrete) to prevent weakening of the geotextile due to UV exposure and possible vandalism. Figure 3.3 shows a wrapped face being covered with shotcrete.

Figure 3.3.—A wrapped-faced wall faced being covered with shotcrete.

Construction Guidelines

The following are some construction guidelines for the USFS wrapped-faced, geotextile-reinforced wall:

- If the geotextile is sufficiently wide for the required reinforcement length, it can be unrolled parallel to the wall (i.e., in the longitudinal direction). Two rolls of geotextile can be sewn together if a single roll is not wide enough. Alternatively, the geotextile can be deployed perpendicular to the wall (i.e., in its transverse direction) and adjacent sheets can be overlapped or sewn. In this way, the machine direction of the geotextile, which is usually the strongest, is oriented in the maximum stress direction.
- The backfill is preferably granular. However, a clayey soil with 20 percent smaller than no. 200 sieve can be used as backfill.
- Compaction shall be done with equipment that will not damage the geosynthetic reinforcement, and no compaction is allowed within 1 to 2 feet from the wall face.
- Typical lift thickness ranges from 8 to 18 inches; however, a lift thickness of 1 foot is most common.
- When making the windrow, care must be exercised not to dig, into the geotextile beneath or at the face of the wall. Before applying a coating to a vertical or near vertical wall, a wire mesh needs to be anchored to the geotextile to keep the coating on the wall face.

CTI Timber-Faced, Geosynthetic-Reinforced Wall

Construction Procedure

The typical construction sequence of the CTI timber-faced, geosynthetic-reinforced wall, as illustrated in figure 3.4, can be described in the following steps:

Step 1. Level wall site, place the initial row of ties or timbers, and place the first geosynthetic reinforcement layer with a minimum of 12 inches tail length.

Step 2. Attach the first reinforcement layer to the initial ties or timbers by nailing the forming element (3-1/2 inches in width) to ties or timbers.

Step 3. Backfill to the top of the forming element, compact the lift, and fold back the tail of the reinforcement (see figure 3.5).

Step 4. Place the second tie and block; place the second layer of geosynthetic reinforcement; attach the reinforcement layer to the ties and blocks by nailing through the forming element (12 inches in width); backfill with a nominal lift thickness of 12 inches; and compact.

Step 5. Repeat step 4 for subsequent layers until the planned height is reached. For the final layer, the fold-back tail length should be at least 6 feet.

Construction Guidelines

The following are some construction guidelines for the timber-faced, geosynthetic-reinforced wall:

- The timber typically has a 6- by 8-inch or 6- by 6-inch cross-sectional dimension and shall be treated to an acceptable level with copper chromate or an approved equivalent preservative. The bottom row of timber shall be treated for direct burial. The color may be green or brown, but not mixed.

- Forming elements may consist of wood (minimum 1-inch nominal thickness treated to an acceptable level with copper chromate or approved equivalent), fiberglass, plastic, or other approved material.

- Typical reinforcement used is a nonwoven geotextile, although other geosynthetics that satisfy the design criteria can also be used.

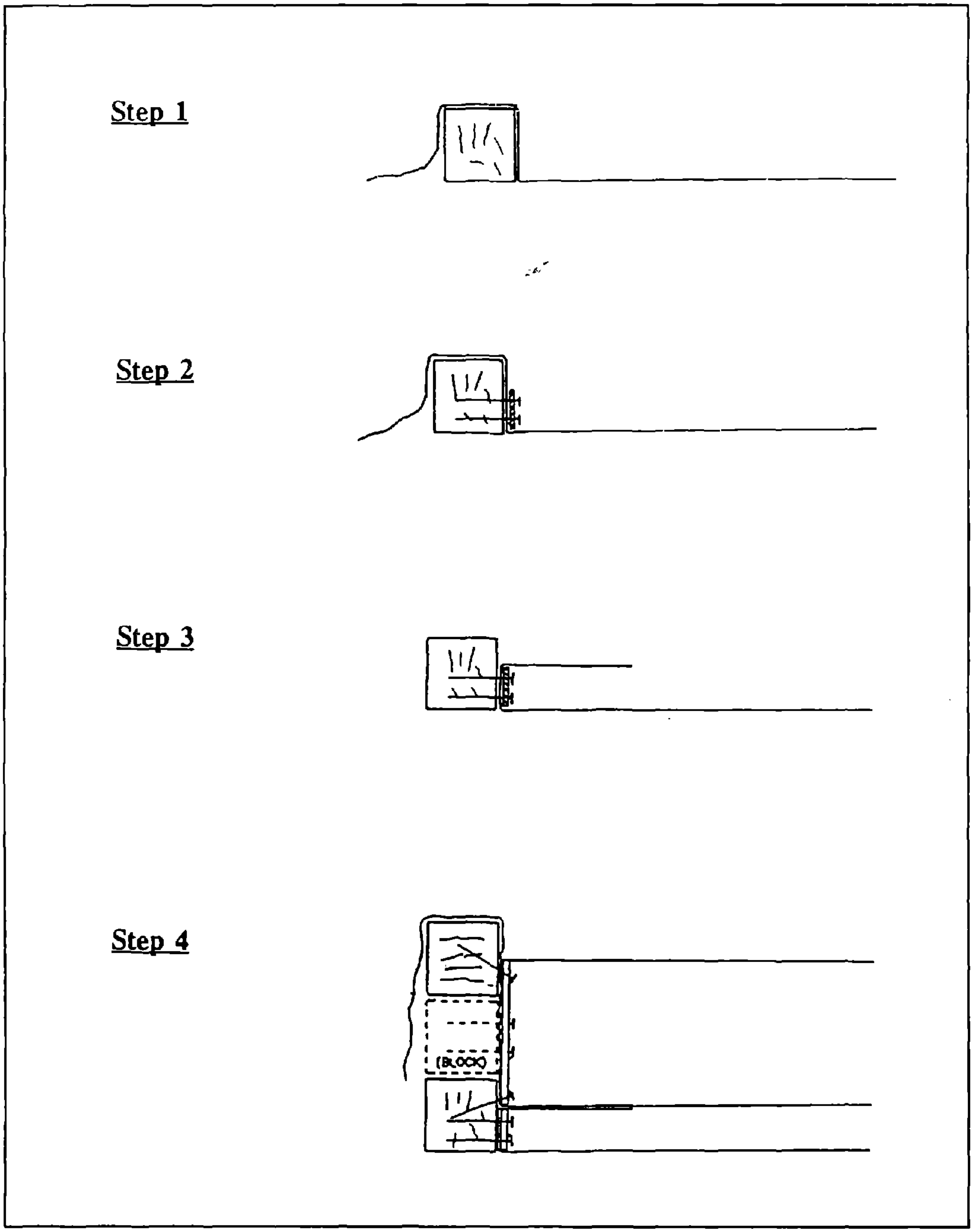


Figure 3.4.—Construction sequence of CTI timber-faced GRS wall.

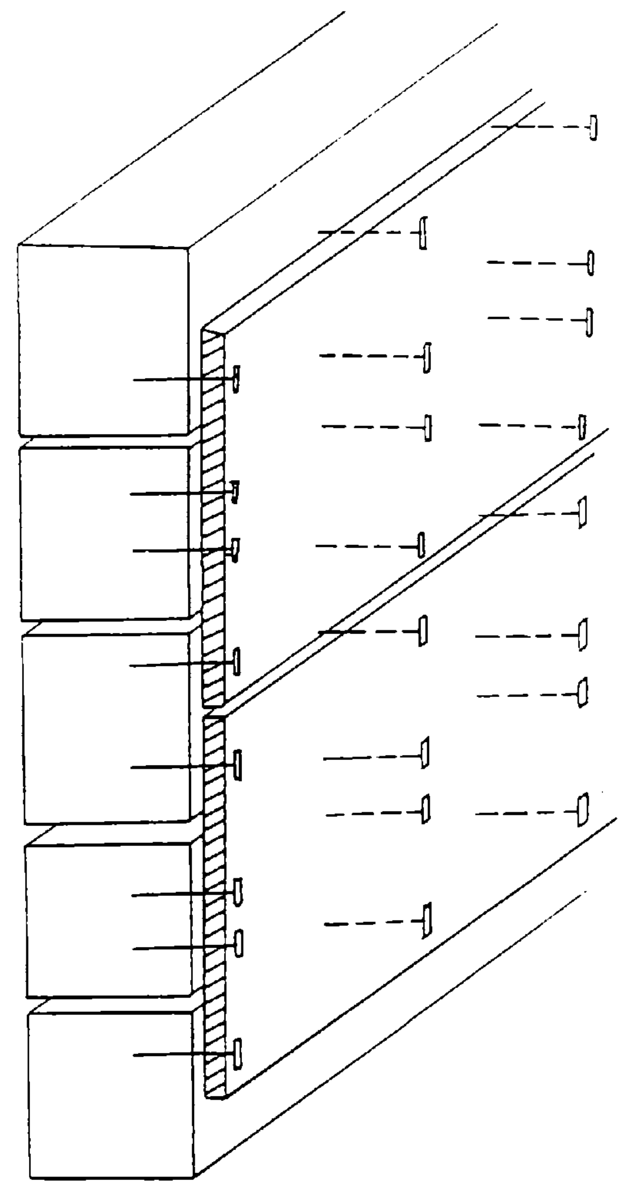


Figure 1.9.—Attachment of forming elements to timber facing (Wu, 1992a).

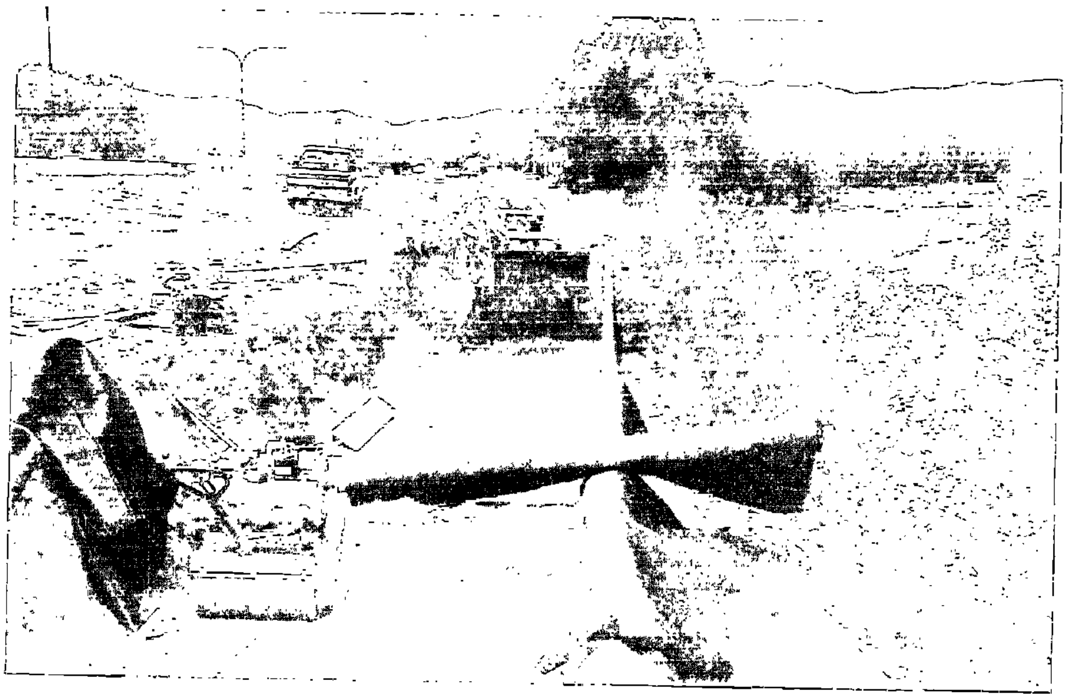


Figure 3.5.—Construction of a CTI timber-faced GRS wall.

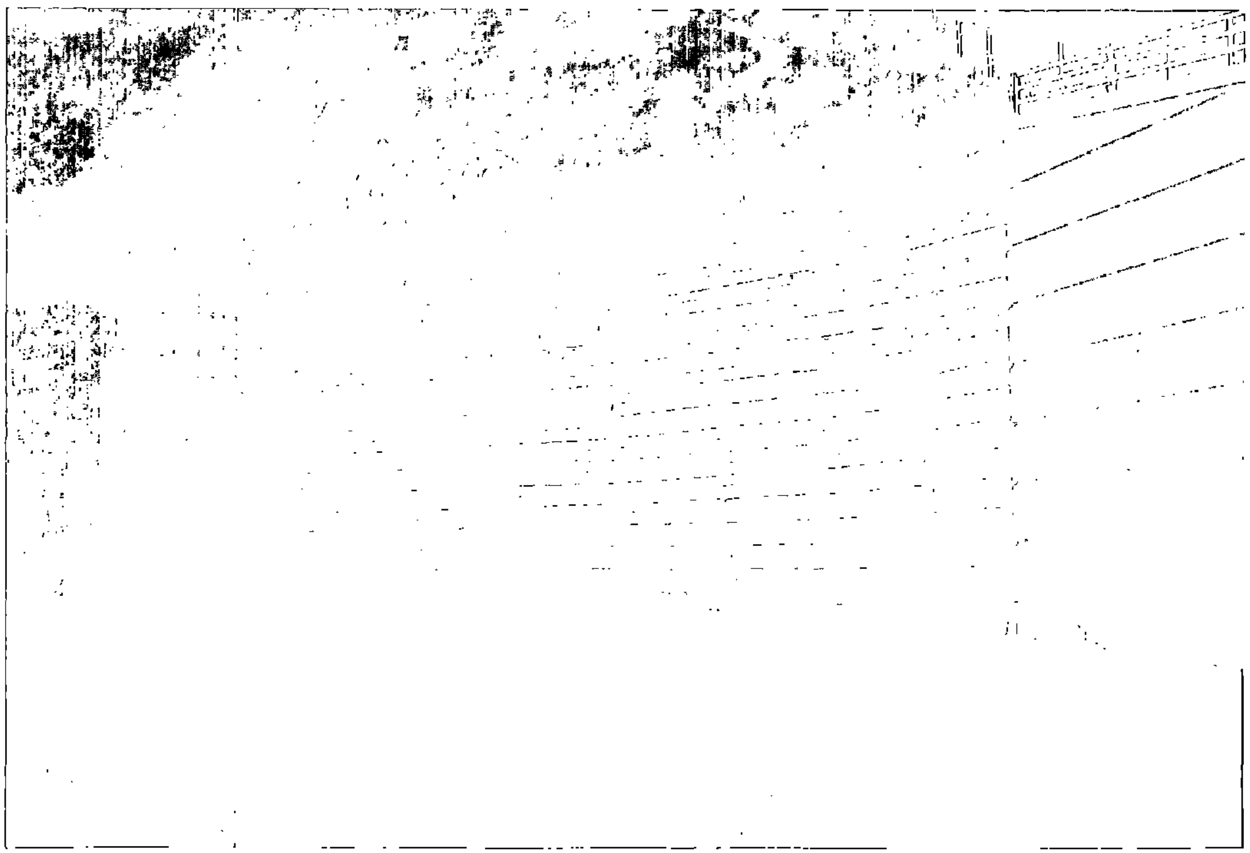
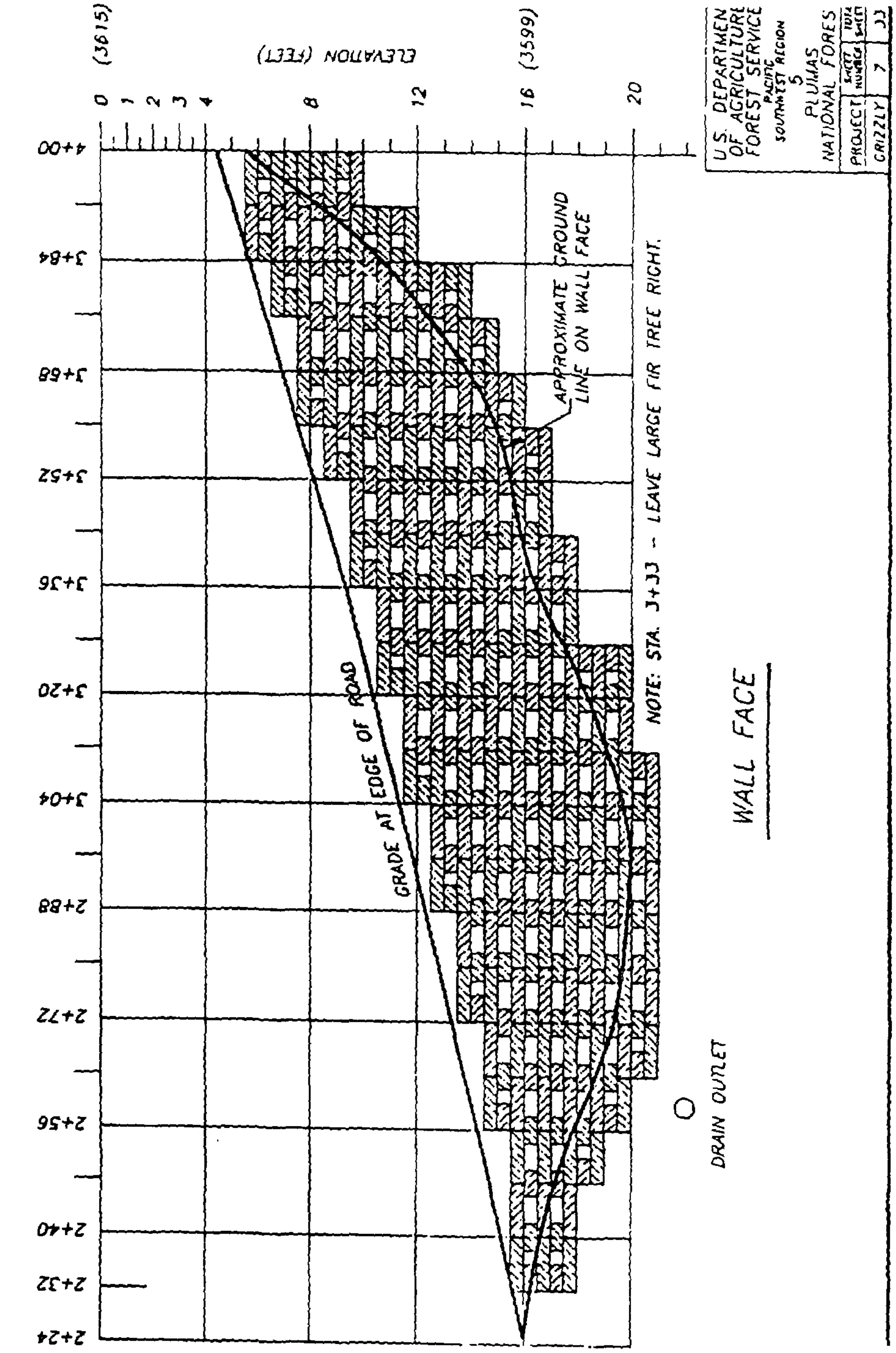
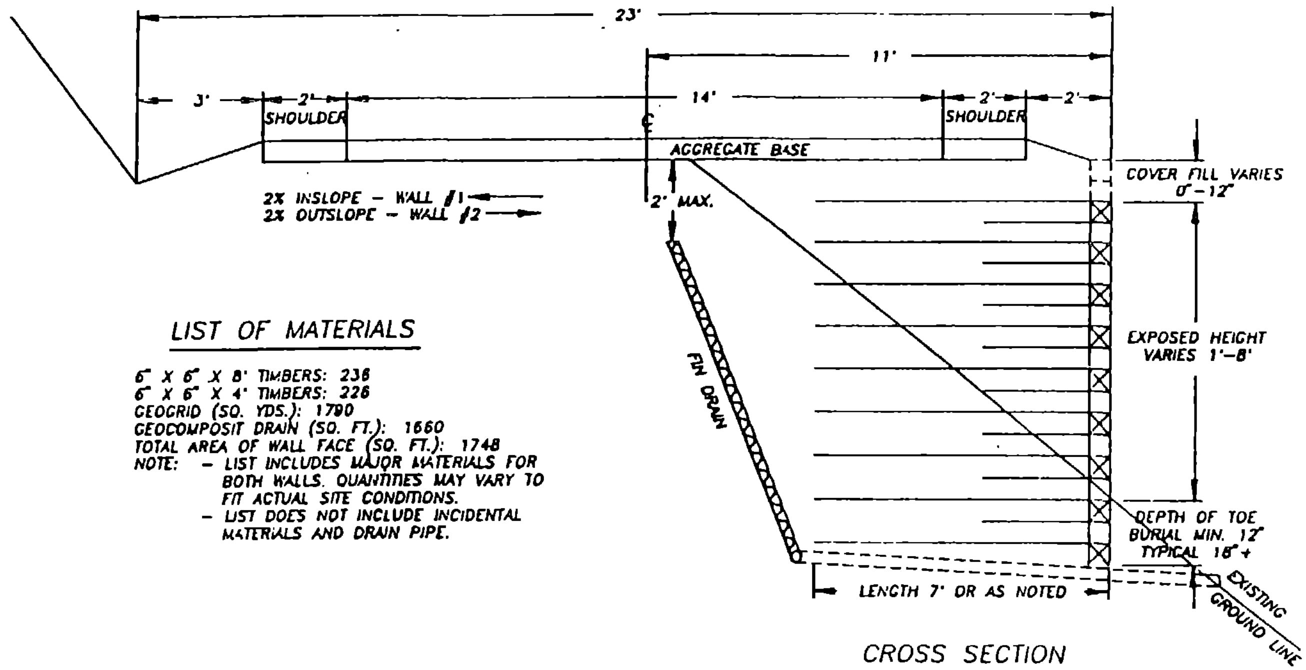


Figure 1.8.—A completed CTI timber-faced GRS wall.

WALL SITE #1 (168') - RD# 24N28



EARTH REINFORCED WALL TYPICAL



CROSS SECTION

LIST OF MATERIALS

6" X 6" X 8' TIMBERS: 238
6" X 6" X 4' TIMBERS: 226
GEOGRID (SQ. YDS.): 1790
GEOCOMPOSIT DRAIN (SQ. FT.): 1660
TOTAL AREA OF WALL FACE (SQ. FT.): 1748
NOTE: – LIST INCLUDES MAJOR MATERIALS FOR BOTH WALLS. QUANTITIES MAY VARY TO FIT ACTUAL SITE CONDITIONS.
– LIST DOES NOT INCLUDE INCIDENTAL MATERIALS AND DRAIN PIPE.

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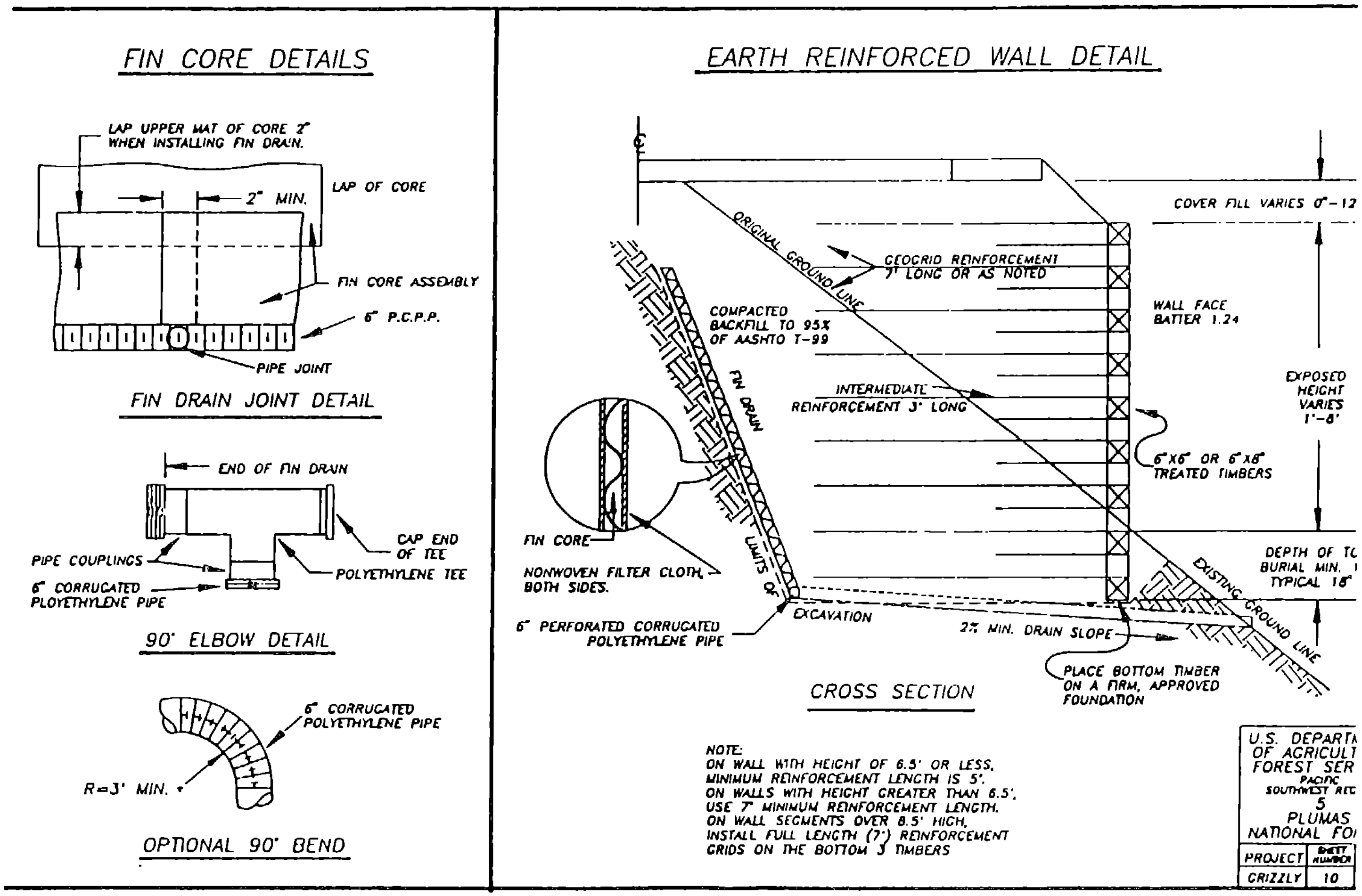
PROJECT	SHEET NUMBER	TOTAL SHEETS
GRIZZLY	9	33


FIN CORE DETAILS
LAP UPPER MAT OF CORE 2" WHEN INSTALLING FIN DRAIN.
LAP OF CORE
2" MIN.
FIN CORE ASSEMBLY
6" P.C.P.P.
PIPE JOINT
FIN DRAIN JOINT DETAIL
END OF FIN DRAIN
CAP END OF TEE
PIPE COUPLINGS
POLYETHYLENE TEE
6" CORRUGATED PLOYETHYLENE PIPE
90° ELBOW DETAIL
6" CORRUGATED POLYETHYLENE PIPE
R=3' MIN.
OPTIONAL 90° BEND
EARTH REINFORCED WALL DETAIL
COVER FILL VARIES 0"-12
ORIGINAL GROUND LINE
GEOGRID REINFORCEMENT 7' LONG OR AS NOTED
COMPACTED BACKFILL TO 95% OF AASHTO T-99
WALL FACE BATTER 1.24
EXPOSED HEIGHT VARIES 1'-8'
INTERMEDIATE REINFORCEMENT 3' LONG
FIN DRAIN
6"X6" OR 6"X8" TREATED TIMBERS
FIN CORE
NONWOVEN FILTER CLOTH, BOTH SIDES.
LIMITS OF EXCAVATION
DEPTH OF TC BURIAL MIN. TYPICAL 18"
EXISTING GROUND LINE
6" PERFORATED CORRUGATED POLYETHYLENE PIPE
2% MIN. DRAIN SLOPE
PLACE BOTTOM TIMBER ON A FIRM, APPROVED FOUNDATION
CROSS SECTION
NOTE:
ON WALL WITH HEIGHT OF 6.5' OR LESS, MINIMUM REINFORCEMENT LENGTH IS 5'.
ON WALLS WITH HEIGHT GREATER THAN 6.5', USE 7' MINIMUM REINFORCEMENT LENGTH.
ON WALL SEGMENTS OVER 8.5' HIGH, INSTALL FULL LENGTH (7') REINFORCEMENT GRIDS ON THE BOTTOM 3 TIMBERS
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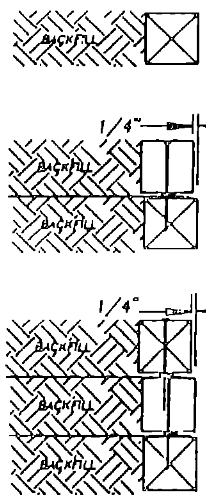
WALL FACE CONSTRUCTION DETAIL

CONSTRUCTION SEQUENCE



SEQUENCE I

STEP 1: LEVEL WALL SITE OR STEPS.
STEP 2: GET ENGINEER'S APPROVAL OF FOUNDATION.
STEP 3: PLACE AND LEVEL INITIAL TIMBERS.
STEP 4: PLACE AND COMPACT BACKFILL.

SEQUENCE II

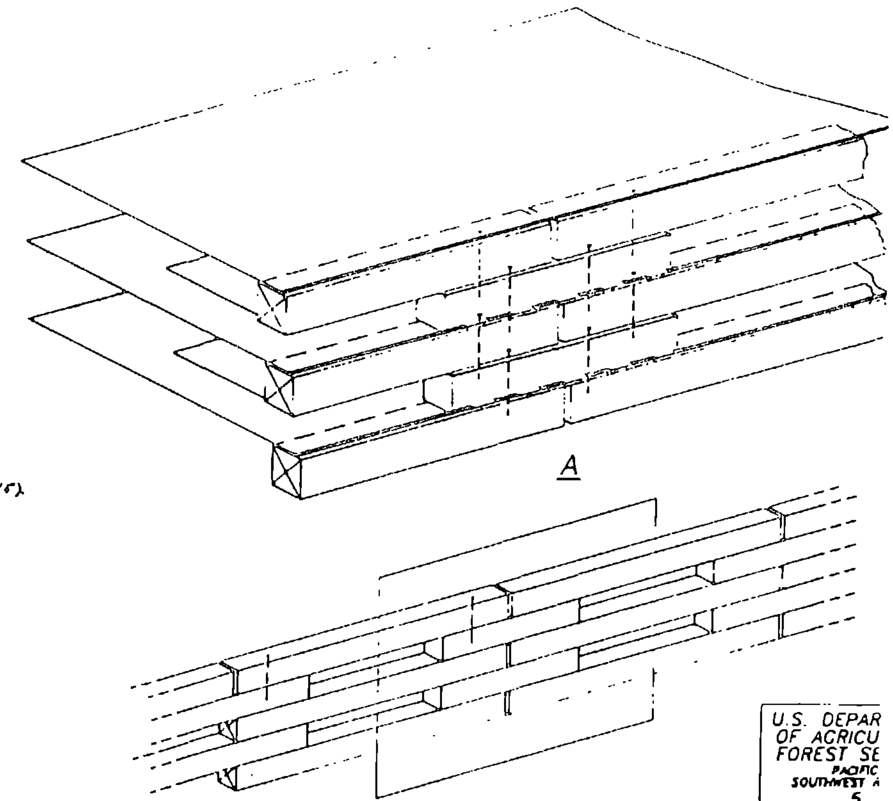
STEP 1: PLACE NEXT LAYER OF GEOGRID.
STEP 2: NAIL GEOGRID TO LOWER TIMBER WITH STRAPS AND Ø10 NAILS.
STEP 3: PLACE NEXT ROW OF 6" TIMBERS (INSET 1/6"). STAGGERED AS SHOWN ON FACE VIEW.
STEP 4: SPIKE TIMBERS TOGETHER.
STEP 5: PLACE AND COMPACT BACKFILL. NO BACKFILL SHALL BE PLACED IN WALL FACE VOIDS.

SEQUENCE III

STEP 1: PLACE NEXT LAYER OF GEOGRID.
STEP 2: NAIL GEOGRID TO LOWER TIMBERS WITH STRAPS WHERE STAGGERED TIMBERS EXIST.
STEP 3: PLACE NEXT SOLID ROW OF 6" TIMBERS (INSET 1/6").
STEP 4: SPIKE TIMBERS TOGETHER (WHERE POSSIBLE).
STEP 5: PLACE AND COMPACT BACKFILL.

SEQUENCE IV

REPEAT SEQUENCE II, THEN III, ETC.



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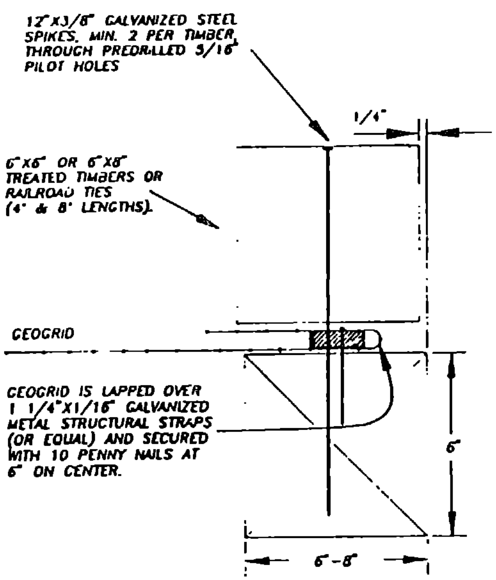
TIMBER–GEOGRID CONNECTION DETAIL



NOTES

1. TIMBER FACING MATERIAL SHALL BE 6"X6" OR 6"X8", CONSTRUCTION GRADE OR BETTER, UNIFORM DIMENSION LUMBER WITH SQUARE ENDS. NON-SQUARE TIMBERS SHALL BE PLACED WITH THE LONGEST DIMENSION HORIZONTAL. HOLE DRILLING SHALL BE DONE PRIOR TO PRESERVATION TREATMENT. TIMBERS SHALL BE PRESSURE TREATED WITH AN APPROVED WOOD PRESERVATIVE, SUCH AS AMMONIATE COPPER ZINC ARSENATE (ACZA), OR ACID COPPER CHROMATE, WHICH MEETS THE REQUIREMENTS OF SPEC 716.03. MINIMUM NET RETENTION SHALL BE 0.60 LBS./ CUBIC FOOT. FIELD CUTS AND APPLICATIONS SHALL BE DONE IN ACCORDANCE WITH SPEC. 557.04, BY A CERTIFIED, LICENSED APPLICATOR.

2. TIMBERS SHALL BE OFFSET 1/4" INWARD, PER LIFT, TO CREATE A 1:24 BATTER. TIMBERS SHALL BE NAILED TOGETHER WITH A MINIMUM OF TWO 12 GALVANIZED SPIKES.

3. GEOGRID USED FOR SOIL REINFORCEMENT SHALL BE A POLY-PROPYLENE POLYMER OR POLYESTER GRID MATERIAL WITH A MINIMUM ALLOWABLE DESIGN TENSILE STRENGTH OF 420 LBS/FT FROM LONG TERM TESTING, AND PEAK STRENGTH OF AT LEAST 2100 LBS/FT. MATERIAL SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURES RECOMMENDATIONS AND SHALL MEET THE REQUIREMENTS OF SPEC. 613E. TENSAR BX-1200 BIAXIAL GEOGRID OR EQUIVALENT MEETS THESE REQUIREMENTS.

4. GEOGRID SHALL BE ATTACHED TO THE TIMBERS BY NAILING AND SANDWICHING THE GEOGRID AROUND METAL STRAPPING AND BETWEEN THE TIMBERS AS SHOWN ON DRAWINGS.

5. BACKFILL MATERIAL SHALL BE A GRANULAR SOIL OR THE NATIVE OR ON-SITE EXCAVATED ROCKY MATERIAL. ROCKS IN THE BACKFILL SHALL BE NO LARGER THAT 6" OR A DIMENSION NO LARGER THAT 75% OF THE THICKNESS BETWEEN GEOGRID LAYERS. LARGER BOULDERS MAY BE PLACED IN THE FILL MATERIAL BEHIND THE GEOGRID REINFORCED ZONE. COMPACTION OF THE BACKFILL MATERIAL SHALL BE TO 95% OF AASHTO T-99 MAXIMUM DENSITY.

6. A DRAIN SHALL BE INSTALLED ALONG THE ENTIRE LENGTH OF THE BACK OF THE WALL EXCAVATIONS. EITHER A PREFABRICATED GEOCOMPOSITE DRAIN MEETING THE REQUIREMENTS OF SPECIAL SPEC. 6[illegible] SHALL BE USED, OR A CONVENTIONAL GRAVEL "CHIMNEY" DRAIN WRAPPED IN A GEOTEXTILE. TENSAR DRAINAGE NET DC 1200 OR EQUIVALENT MEETS THESE REQUIREMENTS. THE EXCAVATION BACKSLOPE SHALL BE AT A STABLE, SAFE, UNIFO[illegible] SLOPE AND SMOOTHED TO ACCOMODATE THE DRAI[illegible]

7. THE WALL FOUNDATION SHALL BE INSPECTED AND APPROVED BY FOREST GEOLOGY OR GEOTECHNICAL PERSONNEL PRIOR TO BEGINNING PLACEMENT OF THE TIMBERS. THE ESTIMATED DEPTH OF WALL SHOWN ON THE DRAWINGS IS APPROXIMATE, BASE[illegible] ON LIMITED INVESTIGATION OF THE SITE. THE WALL TIMBERS MUST REST ON COMPETENT, IN-PLACE SOIL OR ROCK. LOCAL SECTIONS OF WALL MAY HAVE TO BE PLACED AT A DEPTH GREATER THAN THAT SHOWN ON THE DRAWINGS TO INSURE THAT THE ENTIRE WALL FOUNDATION IS ST[illegible]

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- Nails shall be 16d galvanized ring shank nails and shall be placed at the top and bottom of the timbers at 1-foot intervals.

- Compaction shall be consistent with project embankment specifications, except that no compaction is allowed within 1 to 2 feet of the wall face.

- Compaction shall be done with equipment that will not damage the reinforcement.

- Outward batter on the face is not acceptable. An outward batter of 0 to 4 inches horizontal to 10 feet vertical shall be required to maintain verticality of the wall face (see figure 3.6). Shimming of timber to maintain the verticality is permissible.

- Type-3 guardrail posts shall be driven no closer that 30 inches from the face of the wall (30 inches from back of guardrail post to outside face of wall) and shall be metal posts.

- All reinforcement overlaps shall be 1 foot wide and shall be perpendicular to the wall face.

- All exposed fabric shall be painted with a latex paint matching the color of the timbers.

- If the onsite material used as backfill is not free-draining soil (< 5 percent minus no. 200 sieve), a drainage system (such as the one shown in figure 3.7) should be provided.

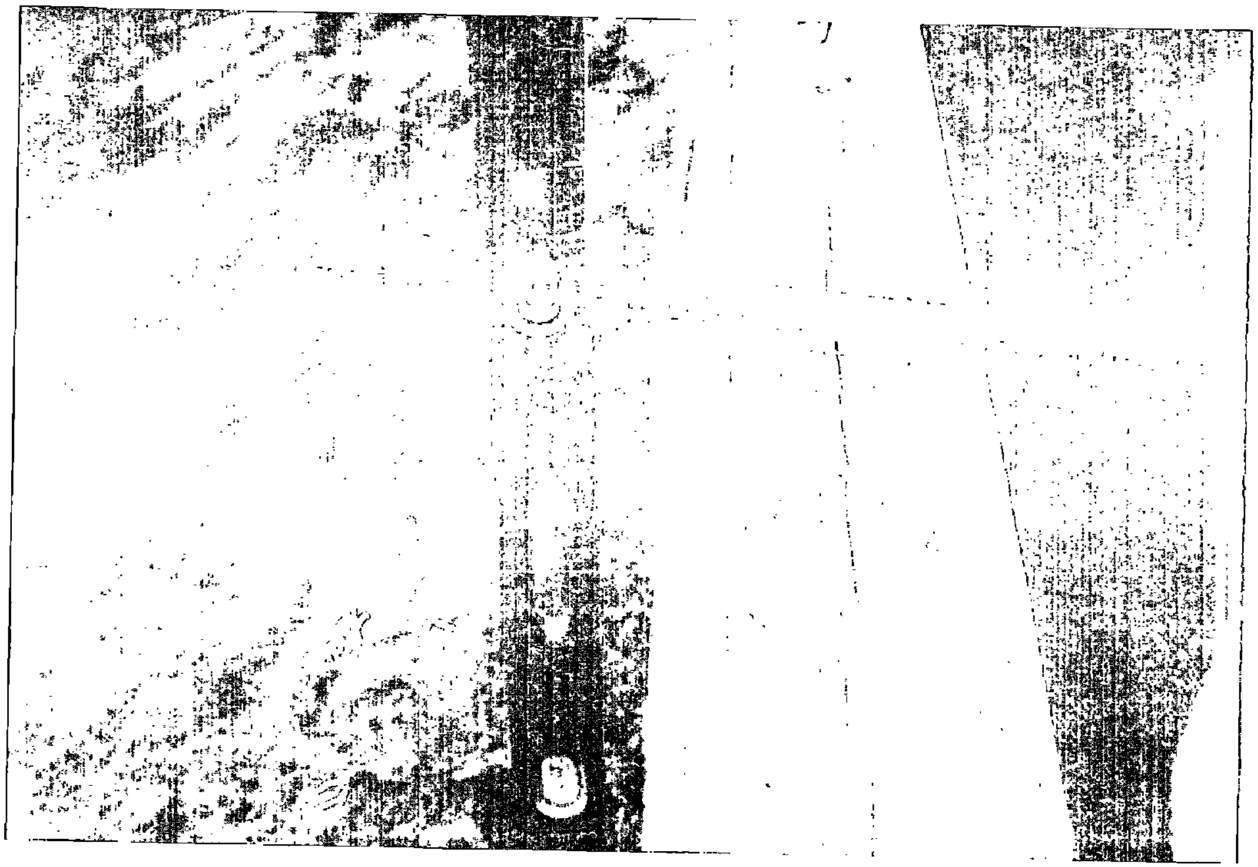


Figure 3.6.—Inward batter of timber wall face.

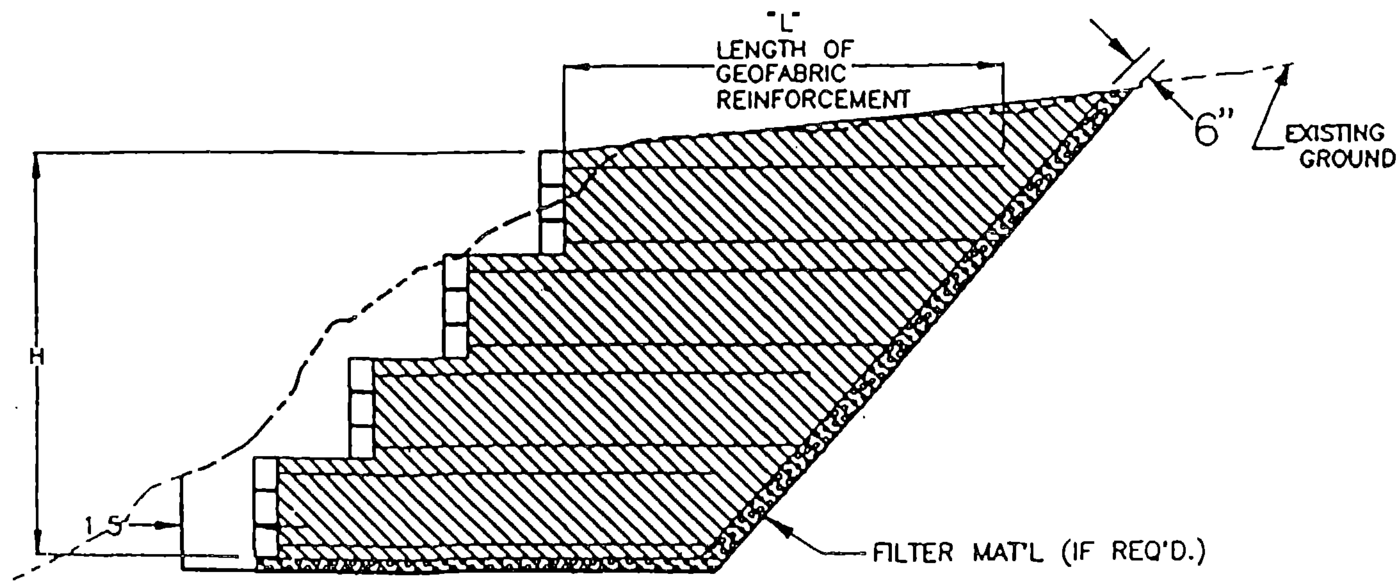


Figure 3.7.—A timber-faced GRS wall with filter material at bottom and back.

Modular Block, Geosynthetic-Reinforced Walls

Construction Procedure

The typical construction sequence of modular block, geosynthetic-reinforced walls, as illustrated in figure 3.8, can be described as follows:

Step 1. Level wall site, cut a shallow trench along the planned location of the wall base, and pour and level an unreinforced concrete pad with a minimum thickness of 3-1/2 to 4 inches.

Step 2. Lay the first course of modular blocks side by side on the concrete pad, check the alignment and level the blocks, and insert pins (if used) into the top of the blocks. Place crushed stone or sand into the hollow cores of the modular blocks and space between the blocks. Clean the surface by sweeping away the debris.

Step 3. Place backfill behind the modular blocks and compact to the top of the block elevation. If the backfill is not a free-draining material, a free-draining gravel 1 foot wide should be placed immediately behind the blocks.

Step 4. Repeat steps 2 and 3 for subsequent courses of modular blocks until a reinforcement layer is to be placed as per the design, install geosynthetic reinforcement across the blocks, and soil fill at the specified elevation (see figure 3.9).

Step 5. Repeat step 2 for the next course of blocks, pull taut and anchor the reinforcement, and backfill behind the blocks and compact.

Step 6. Repeat steps 4 and 5 until the planned height is reached. The last course of blocks are usually capped according to the manufacturer's recommendation.

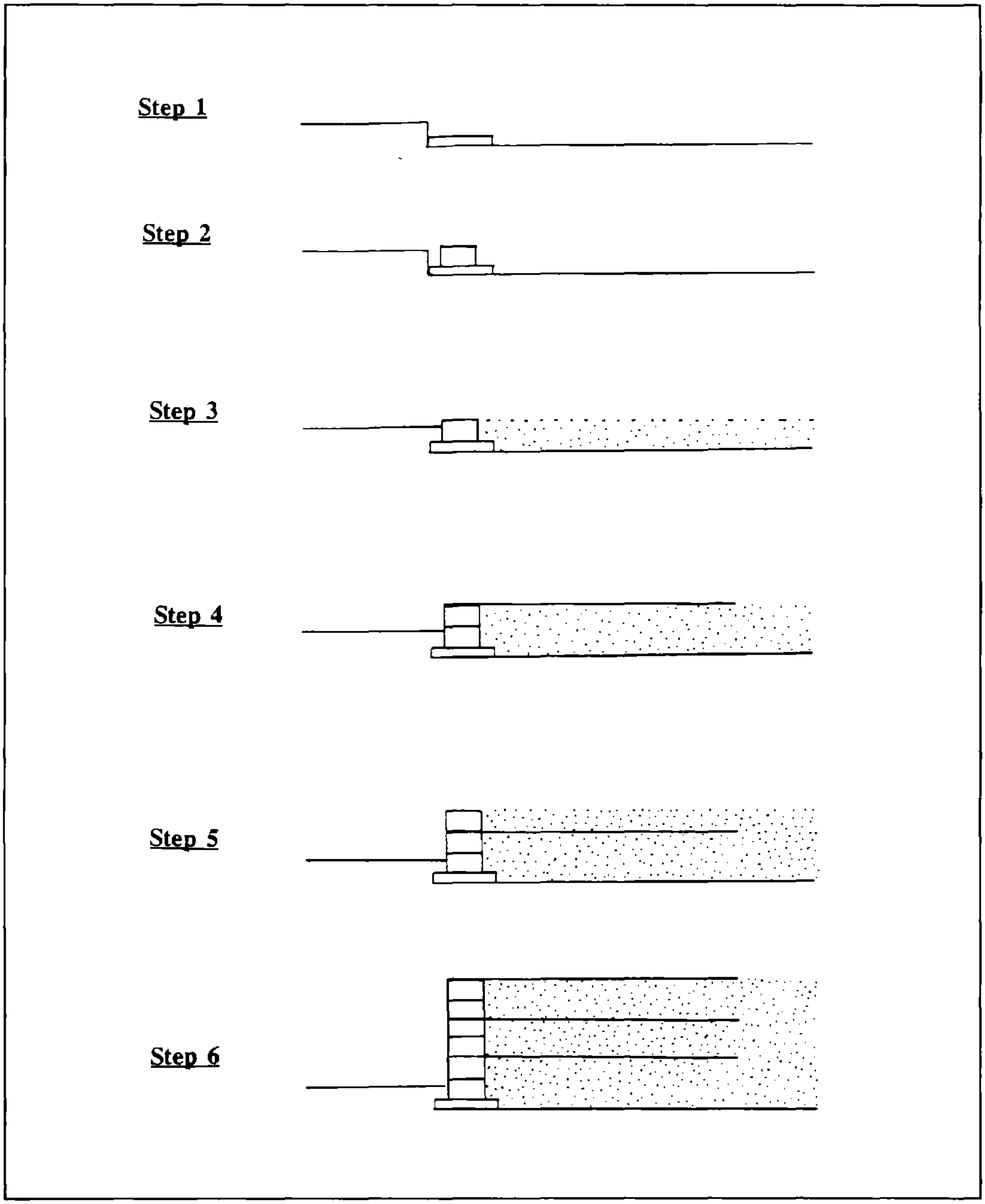


Figure 3.8.—Construction sequence of modular block GRS wall.

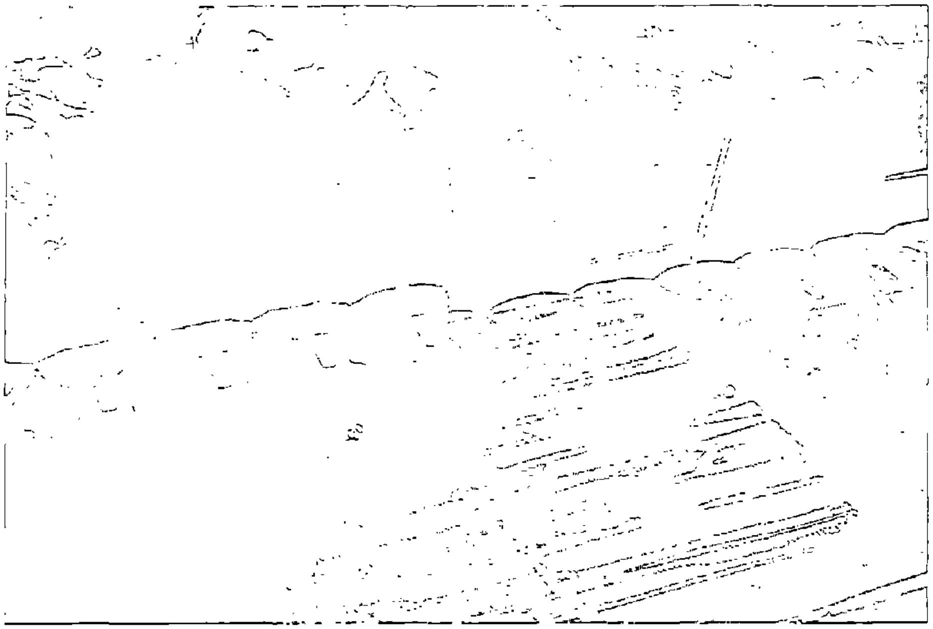


Figure 3.9.—Construction of a modular block GRS wall.

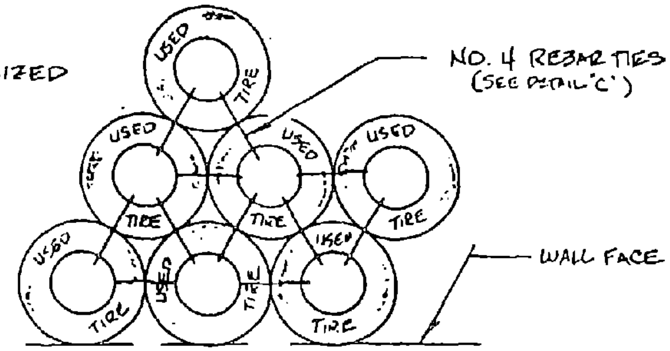
CONSTRUCTION DETAIL

(USED TIRE RETAINING WALL)

NOTES:

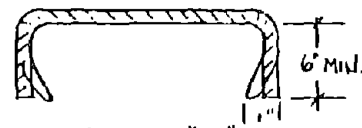
~~TWO~~ 3- WAY TIES (TRANSVERSE TO WALL FACE) ARE REQUIRED WITH REBAR OR 8 GA. GALVANIZED WIRE

THE REQUIREMENTS OF SECTION 819B.02, C) ARE HEREBY WAIVED WHEN THE DESIGNATED MATERIAL SOURCE IS USED.



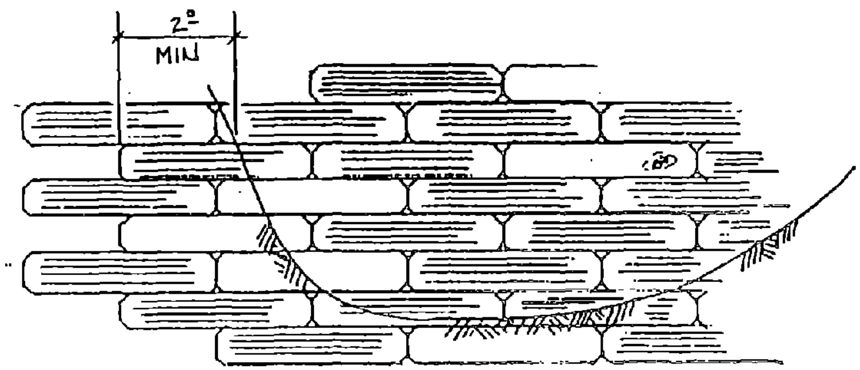
PLAN VIEW

(REBAR TIE PLAN)



DETAIL "C"

(#4 REBAR)



DETAIL "B"

MP 2.38

BUTT LAKE

ALMANOR

Pg 2 of 2
SITE 5-1
RO 27N98
OZZIE 2/38

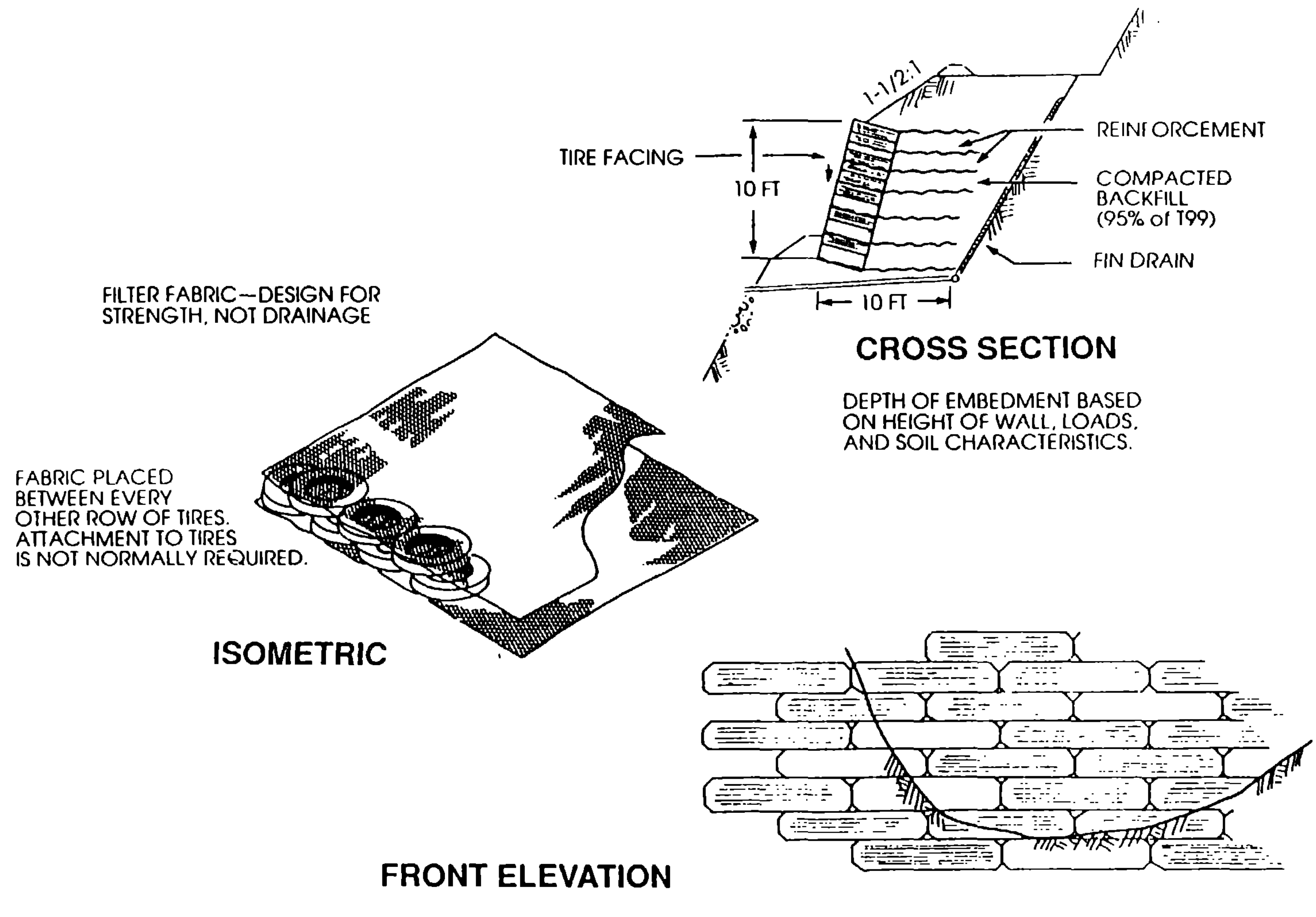


Figure 1c.—Used tire/geographic retaining wall.

Construction Guidelines

The following guidelines should be observed when constructing a modular block, geosynthetic-reinforced wall:

- The concrete leveling pad under the first course of modular blocks can be replaced with a leveling pad of compacted gravel (or compacted in situ soil). However, the use of a concrete leveling pad is recommended when the foundation soil is relatively incompressible and not susceptible to significant shrinkage and swell due to moisture changes. A properly poured and leveled concrete pad will speed up construction, ease the leveling process, and facilitate the construction of a straighter wall.

- Walls with curves along their length require that the leveling pad be poured to the proper radius. In general, a curve radius of 10 feet or greater is not a problem; however, tight curves of 3 to 6 feet radius require special consideration (Moreno et al., 1993). In some cases, field modification of the blocks may be necessary for tight curves.

- The blocks should be laid from one end of the wall to the other to preclude laborious block-cutting and fitting in the middle. When curves are involved in a wall, the blocks on the curves should be laid first as their alignment is more critical and less forgiving. Tight curves often require cutting blocks to fit or breaking off the block tail. A diamond-tipped blade saw is recommended for the cutting.

- When shear pins are used, they should be tapped into well-seated positions immediately after setting each block to avoid getting fill into the block's pin holes.

- Leveling of the first course of blocks is especially important for wall alignment. A string line set over the pins from one end of the wall to the other will help leveling the blocks.

- Geosynthetic reinforcement should be placed up to the front face of the blocks to ensure maximum interface contact with the blocks.

- After the front of the geosynthetic reinforcement is properly secured (i.e., after the hollow cores of the next course are filled and compacted), the reinforcement should be pulled tight and pretensioned while the backfill is being placed.

- Use care when placing backfill over geosynthetic reinforcement. The backfill should be emplaced from the wall face to the back of the wall to ensure that no slack is left in the reinforcement.

- To avoid the movement of blocks during construction, a hand-operated tamper should be used to compact the soil within 3 feet of the wall face, and no construction vehicles are allowed within the 3-foot region.

PERMAPOST
PRODUCTS COMPANY

P.O. BOX 121 · 25600 S.W. TUALATIN VALLEY HWY
HILLSBORO, OREGON 97123 · (503) 648-4156

PRODUCT INFORMATION - MARCH 1975

The Perma-Crib Wall

As used by the U.S. Forest Service and State Highway Department.

The Perma Crib Wall

Permapost Products Company has manufactured timber crib walls throughout the past decade and continues to lead the field in design, manufacture and service. Pressure treatment using Pentaclorophenol insures quality, permanence and a pleasing brown color that blends well into any environment.

Experience and knowledge gained by many years of working with varied designs furnished by the Bureau of Public Roads, various state highway departments, the U.S. Forest Service, Public Works Agencies, and numerous other agencies, and observing their installation, has enabled us to develop many improvements, resulting in our own Perma-Crib design.

This design below was adopted by the U.S. Forest Service for use on several jobs in the Mt. Hood National Forest. Testing and instrumentation of the design was conducted by the University of Idaho under a contract with the U.S. Forest Service. The study and experience gained here enabled us to complete additional improvements to the design and to improve installation. During recent years this tested, proved, crib design has been utilized on many projects including several large cribs for the Colorado State Highway Dept. on Interstate highway No. 70 near Vail, Colorado. Using relatively unskilled labor and a minimum of equipment the contractor reported excellent assembly time with no problems.

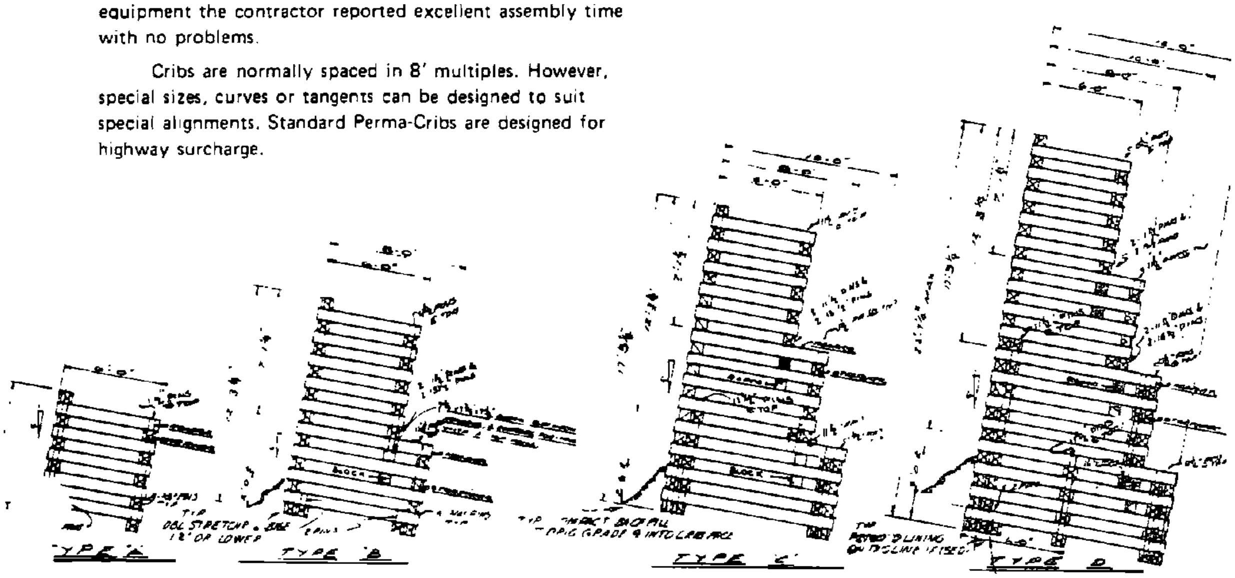
Cribs are normally spaced in 8' multiples. However, special sizes, curves or tangents can be designed to suit special alignments. Standard Perma-Cribs are designed for highway surcharge.

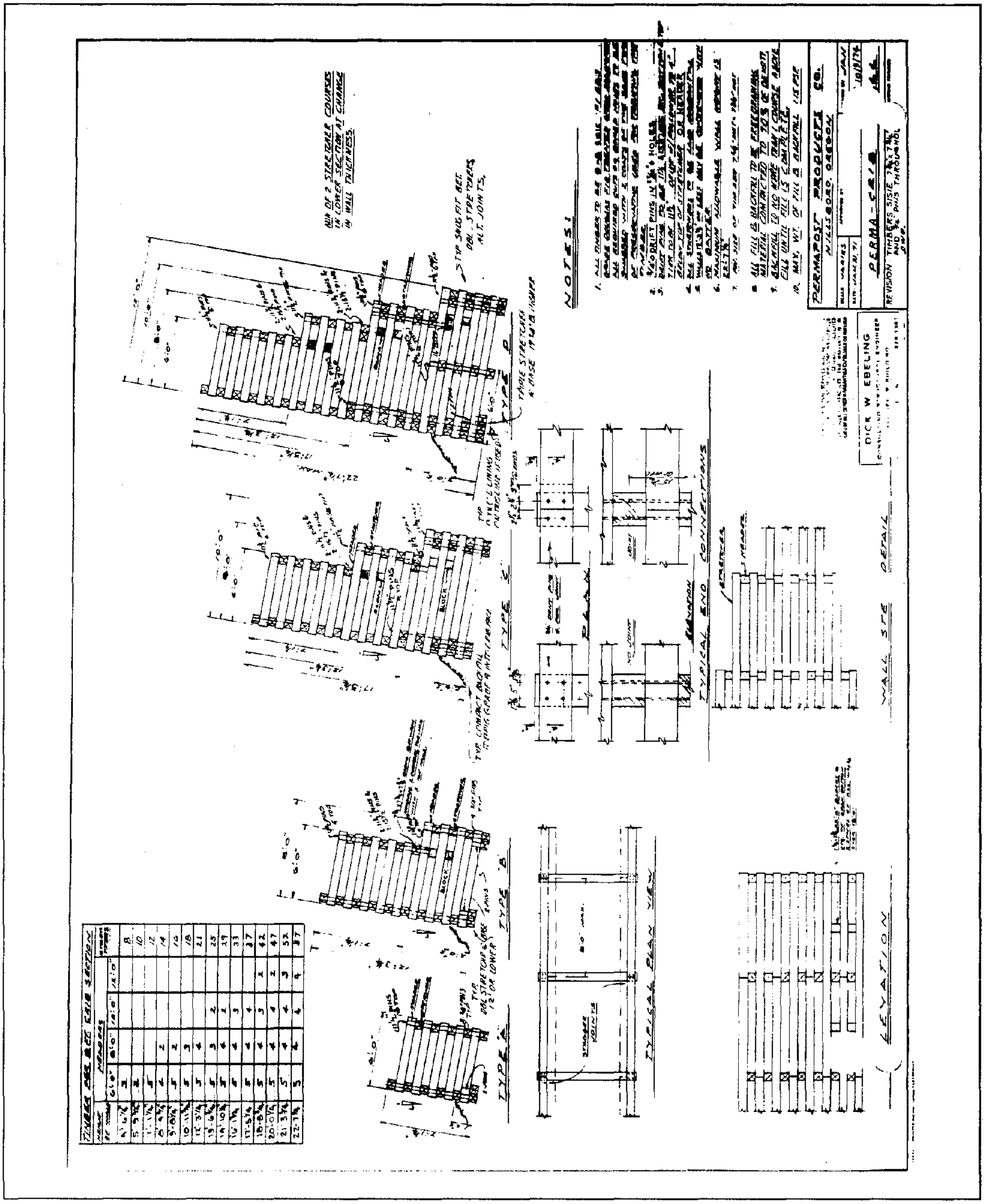
Four types are available to cover various heights:

Type A - Up to 7' 1¼"	Type C - Up to 17' 5¼"
Type B - Up to 12' 3¼"	Type D - Up to 22' 7¼"

Prices upon request.

For lowest per annum cost - specify "Perma-Crib".




PERMAPOST PRODUCTS CO.
HILLSBORO, OREGON
PERMA-CRIB
DICK W. EBELING
NOTES:
TYPE A
TYPE B
TYPE C
TYPE D
TYPICAL PLAN VIEW
TYPICAL END CONNECTIONS
ELEVATION

Used Tire Retaining Wall Specification

Description

819C.01

This work shall consist of constructing retaining walls with used tires, reinforcement with geofabric or geogrid and backfilling with specified select materials.

Materials

819C.02

Used Tires Tires shall be standard passenger car tires structurally intact with the steel bead reinforcing totally covered.

Reinforcement Reinforcement shall be filter fabric or geogrid meeting the strength requirements SHOWN ON THE DRAWINGS.

Backfill Material Backfill material shall be taken from sources SHOWN ON DRAWINGS, and with amount passing the no. 200 sieve less than 25 percent. When the amount passing the no. 200 sieve exceeds 15 percent, the angle of internal friction shall be at least 30 degrees. Small pockets of material may need to be rejected when they show visually that they may not meet this criteria. These requirements may be waived when direction to do so is SHOWN ON THE DRAWINGS.

Requirements

819C.03 General

The areas to receive retaining walls will be SHOWN ON THE DRAWINGS and MARKED ON THE GROUND by the Engineer.

819C.04 Excavation

Excavation shall be in accordance with requirements and limits AS SHOWN ON THE DRAWINGS. Excess excavation shall be disposed of AS SHOWN ON THE DRAWINGS.

819C.05 Foundation Preparation

The foundation for the structure shall be smooth for the length and width equal to or exceeding the length and width of the structure AS SHOWN ON THE DRAWINGS. Prior to wall construction, the foundation shall be compacted by two complete passes with a mechanical compactor, unless constructed on rock.

819C.06 Wall Erection

Tires shall be placed in successive horizontal lifts in the sequence SHOWN ON THE DRAWINGS as backfill material is placed. Tires shall be maintained in the battered position SHOWN ON THE DRAWINGS as backfill placement proceeds. When reinforcement is required, it shall be placed in horizontal or insloped layers as SHOWN ON THE DRAWINGS.

Vertical (plumbness) and horizontal (alignment) tolerance of the front face of the wall shall not exceed 1/2 foot from neat lines SHOWN ON THE DRAWINGS or staked on the ground, when measured along a 10-foot straight edge.

819C.07 Backfill Placement

Backfill placement shall follow the erection of each lift of tires. The moisture content of the solid shall be suitable for compaction prior to placement in the backfill. Backfill material shall be placed in a loose layer in and around 2–3 inches above each lift of tires. The maximum lift thickness allowable above the tires after compaction shall be 1/2 to 2 inches, unless otherwise SHOWN ON THE DRAWINGS. The maximum lift thickness between reinforcement shall be 15 inches after compaction, unless otherwise SHOWN ON THE DRAWINGS. The void inside the tire wall need not be filled.

The tire rim hole shall be filled and compacted by walking on and "healing in" the backfill material as it is placed, before placing the backfill behind the tires. On batters steeper than 6 in 1, place a rock over the hole in the tire between the lower tires to prevent backfill material from falling out.

Each layer of backfill shall be compacted by three complete passes with a vibratory compactor, "whacker", or other approved method made for the compaction of soil embankments. To prevent displacement of the last lift of tires, the outside one horizontal foot of backfill against the tires may need to be compacted by a hand tamper, or other light compaction equipment, prior to compacting the rest of the backfill.

Backfill shall be continued in this manner until 1 foot above the top of the wall; the rest of the fill above the wall shall be wheel-rolled by at least two complete passes, or meet embankment requirements, whichever provides the greater density.

Method of Measurement

819C.08

Retaining walls will be measured by the wall face area according to neat lines SHOWN ON THE DRAWINGS, or by LUMP SUM, as SHOWN ON THE SCHEDULE OF ITEMS.

Basis of Payment **819C.09**

Payment will be made, for all units inspected and accepted, at the unit price SHOWN ON THE SCHEDULE OF ITEMS.

Payment will be made under:

Pay Item	Pay Unit
819C(01) Retaining Wall, Used Tire	S.F.
819C(02) Retaining Wall, Used Tire	LUMP SUM

CONCRETE INFORMATION
PORTLAND CEMENT ASSOCIATION

SMALL RETAINING WALLS—Construction and Tables

RETAINING walls must be safe against overturning and sliding forward. The pressure under the toe (front bottom edge of the base) should not exceed the bearing power of the soil. The friction between the base and the soil on which it rests plus the pressure of the earth in front of the wall must be sufficient to keep it from sliding forward. In addition, the wall must be sufficiently strong to prevent failure at any point in its height due to pressure of the retained material.

It is desirable to have an engineer familiar with local conditions check the design of even small walls and where especially unfavorable soil conditions obtain, such as silt or quicksand or where piles are required under the wall, the services of an engineer are essential both in design and construction.

Choice of Type

Type of retaining walls suitable for comparatively low heights are:

1. The simple vertical face gravity wall, Fig. 1(a) and 1(b).
2. The backward leaning gravity wall, Fig. 2(a), 2(b), 3(a) and 3(b).
3. The cantilever reinforced wall, Fig 4(a) and 4(b).

The choice of type depends on a number of factors peculiar to the location that affect the cost, efficiency and suitability of the different types. The relative cost of materials and labor in a given locality may determine the selection of a gravity or reinforced type. The reinforced type has a slightly lower toe pressure which may make it desirable where soil bearing values are low. The gravity wall has, however, greater resistance to sliding because of its greater weight. The backward leaning wall is economical as to materials, but reduces the usable surface back of the wall. Retaining walls built at property lines or where their bases are fixed by physical conditions and where a maximum of usable surface area back of them is desired, should have a vertical face. The appearance of a backward leaning and vertical face may also influence the choice of type.

Details of Construction

A footing offset of 6 inches at the base is sufficient to support forms for the stem, and allows a close approach of the face of the wall to the property line.

Resistance to horizontal movement of the retaining wall by sliding is obtained by frictional resistance between base and foundation. Often a lug or offset under the base slab is provided to assist in resisting the tendency to slide. The same effect is achieved by requiring that the base slab be well below the ground surface. This requirement applies particularly to walls higher than 7 ft. above the base. The base of the wall should be below the frost line.

Since gravity walls are stable because of their weight, no reinforcement is required in them. It is not necessary, therefore, to dowel the stem to the base with bars. It is satisfactory simply to form a "key-way" by casting a "V" groove in the base.

The stepped back, as shown in Fig. 1, has the advantages of providing more openings for placement of concrete, and also does away with the tendency of the forms to "float." The "treads" of the "steps" need not be formed. Quantities of materials are practically the same for both types.

Backward leaning walls having an inclination so great as to cause them to tip backward before the filling is placed require a longer heel than vertical walls to make them stable. For inclinations greater than 20 deg., the length of heel required will be greater than those shown in Fig. 2 and 3. The forms should be braced to prevent backward tipping while the concrete is being placed and maintained in position until the concrete has thoroughly hardened.

Where the ground surface back of the wall is level, or nearly so, the wall should carry a railing for safety and appearance. Railings vary greatly and may be either precast or cast in place. Cast-in-place railing posts are anchored to the wall with reinforcement dowels. Precast posts may be anchored in the same manner, except that the dowels are cast in the posts and anchored in holes which are cast or drilled into the top of the wall and filled with mortar when the posts are set. Precast posts may also be anchored by means of flange plates held to the posts by bars or bolts embedded in them—the flange plates being anchored to the wall by means of anchor bolts or cinch anchors.

Excavation should be carried to firm ground and below the frost line. A depth of three feet should be sufficient in moderate climates to eliminate the possibility of "heaving."

Proper drainage of retaining walls should be provided. A layer of coarse stone 12 in. thick should be placed against the back of the wall, and weep holes through the stem installed at frequent intervals along the base. Four-inch diameter tile drains spaced about 10 feet apart are usually sufficient.

Suggestions for the design of concrete mixes and a brief discussion of the fundamentals of concrete making are given in *Design of Concrete Mixtures*.* Concrete when used in retaining walls is usually subject to quite severe exposure so the mixture should be based on 6 gal. total water per sack of portland cement. For a mix of medium consistency with gravel aggregate graded up to 2 in. size the proportions will

*Available from the Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

be about 220 lb. damp sand and 425 lb. gravel per sack of cement. The water to be added for each sack of cement is about 4.7 gal. This allows 5 per cent by weight of free moisture in the sand.

From these data it is found that one cu. yd. of concrete requires 5.3 sacks of cement, 1165 lb. of damp sand, and 2250 lb. of gravel.

The quantities of materials required to build any type of wall are easily determined. For example, a gravity type retaining wall with level fill [Fig. 1 (a)] according to the table requires 0.76 cu. yd. concrete per linear foot for a wall of h = 8 ft. The materials necessary are 4.0 sacks cement, 885 lb damp sand, and 1710 lb. of gravel for each foot of wall.

Retaining walls are usually exposed to public view and for best appearance require care in form construction. Forms should be substantial so that bulging does not occur, and sheathing of uniform quality should be driven up tight so that leakage is prevented. Methods of producing ornamentation and the construction of forms are described in *Forms for Architectural Concrete*.*

Many types of surface textures are available for concrete walls. Board-marked surfaces show the lines of rough or dressed lumber in the forms. Wood strips as inserts may be placed in the forms to create a wide variety of patterns and designs. Form plywood and thin plywood liners are available in a variety of textures—smooth, striated, and sandblasted. Materials and methods for producing various wall textures are described in *Textures Produced by Various Form Liners*, *Bushhammering of Concrete Surfaces*, *Sandblasting of Concrete Surfaces*, and *Exploring Color and Texture*.*

Vertical contraction joints should be placed in the wall at 20- to 30-ft. intervals to prevent the occurrence of unsightly cracks due to temperature change and shrinkage. A tongue and groove key may be provided to aid in maintaining alignment of adjacent sections. It is advisable to cover contraction joints with a strip of membrane waterproofing on the back of the wall to prevent seepage through the joint.

The back-filling should be placed in such manner as not to produce impacts, as from large stones rolling down a slope against or dropping on the wall, nor undue variations of pressure against it. It is good practice to bring up the filling material along the wall at a rate as nearly uniform as practicable.

Figures and Tables

The tables accompanying the figures are based on unit weights of earth and concrete of 100 lb. and 150 lb. per cu. ft., respectively. If the filling material or the method of placement used is such as to produce much higher thrusts than from ordinary filling of earth, sand and gravel, the tables do not apply.

The reinforced walls in the tables were all designed so that the resultant of the weight of the wall, the weight of the filling material, and the thrust of the retained material will pass through the outside edge of the middle third of the bottom of the footing. Under this condition the foundation pressure at the toe will be twice the average. The gravity walls are designed so that the resultant will be within the middle third of the bottom of the footing but close to the outside edge.

When the surface back of a wall slopes upward, or where it is level but carries a load—as from a road or building—the wall is said to have a surcharge. Surcharge increases the thrust against the back of the wall and requires a heavier wall than otherwise. The tables give dimensions and quantities for surcharged walls supporting an upward sloping fill and for an unloaded level fill. If the latter carries an external load, such as a building or roadway that will carry heavy loads, the dimensions—but not the quantities—may be taken from the tables for a level fill, by considering the load per square foot as additional depth of fill, and taking the base slab width and thickness as for a wall having a height increased by the surcharge load. Thus, if the surcharge load on the level fill amounts to 200 lb. per sq. ft., base slab dimensions and stem thickness at top of base should be taken for a wall two feet higher than the actual height of the wall (assuming weight of filling material to be 100 lb. per cu. ft.).

Safety against overturning or forward rotation about the front edge of the base is had only if the pressure on the earth under the toe does not exceed its bearing power. The approximate maximum pressure on the soil under the vertical faced walls without surcharge shown in the tables may be obtained by adding together the weight per foot of length of wall and the filling vertically over the base, dividing by the width of the base in feet and multiplying the quotient by 2. Table I gives safe bearing power values for different soils. If the computed pressure exceeds the value shown for the soil in question, the toe or heel or both must be extended.

For walls leaning backward the thickness can be decreased proportionately to the amount of backward leaning. It is evident that if the wall leans backward at an angle equal to the angle of repose of the earth (a slope such that the earth will stand alone) it is not needed to retain the material back of it and becomes simply a revetment (protection for the slope). It is evident, also that the theoretical dimensions would change with each change in the backward inclination. The tables are, however, computed for the *least* backward inclination in each group. Thus, Fig. 2(a) is applicable to a wall with a vertical back and inclined face and for all backward inclinations of the back up to 10 deg. (about 1 horizontal to 6 vertical).

The tables for cantilever walls give the reinforcement in standard bar sizes. The length of the "V_2" bars was determined from the bending moments in the vertical stem of the wall and the ordinary requirement for length to provide anchorage by bond.

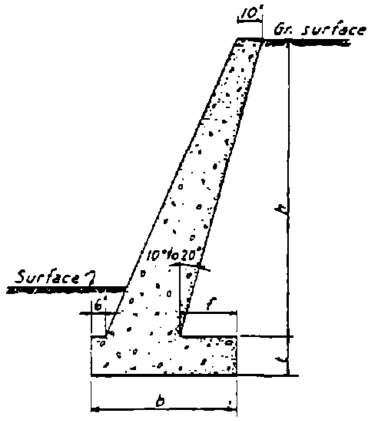
TABLE I—BEARING POWER OF SOILS

Material	Bearing Power (lb per sq ft.)
Clay	2,000
Sand and Clay Mixed	4,000
Alluvium and Silt	5,000
Hard Clay and Firm Compressed Sand	8,000
Fine Sand	9,000
Sand, Compacted and Cemented	10,000

*Available from the Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

BACKWARD INCLINED GRAVITY RETAINING WALLS WITHOUT SURCHARGE

(An inclination varying from 10° to 20°— from 1 horizontal–6 vertical to 1 horizontal– 3 vertical)

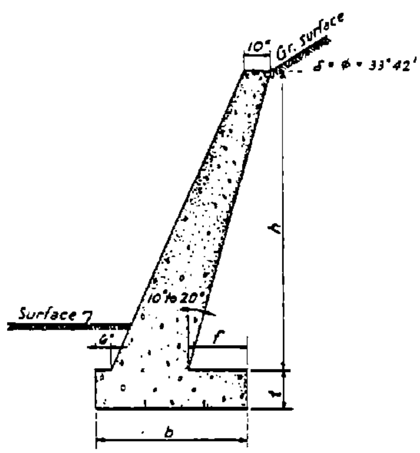


h (ft.)	f (in.)	b (ft.-in.)	t (in.)	Vol. Concrete (Cu. yd. per ft.)
3	8	2-0	8	0.14
4	10	2-4	8	0.19
5	12	2-9	8	0.26
6	14	3-2	12	0.38
7	16	3-7	12	0.47
8	18	4-0	12	0.57
9	20	4-5	16	0.73
10	22	4-10	16	0.86

(a)

BACKWARD INCLINED GRAVITY RETAINING WALLS - SLOPING SURCHARGE

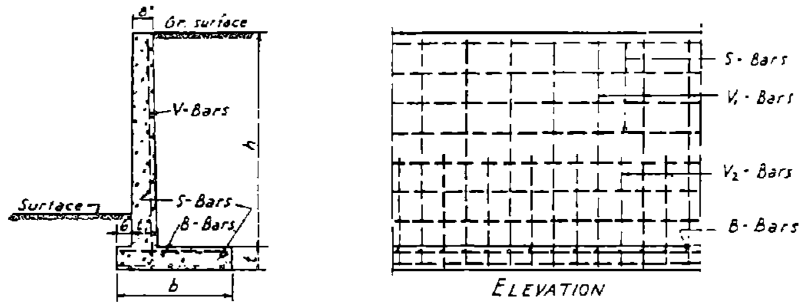
(An inclination varying from 10° to 20°-- from 1 horizontal-6 vertical to 1 horizontal-3 vertical.)



h (ft)	f (in.)	b (ft.-in.)	t (in.)	Vol. Concrete (Cu. yd. per ft.)
3	10	2-2	8	0.15
4	12	2-6	8	0.20
5	14	3-0	8	0.27
6	16	3-6	12	0.41
7	18	3-10	12	0.49
8	20	4-3	14	0.62
9	22	4-8	14	0.73
10	23	5-0	16	0.88

(b)

CANTILEVER RETAINING WALLS
WITHOUT SURCHARGE

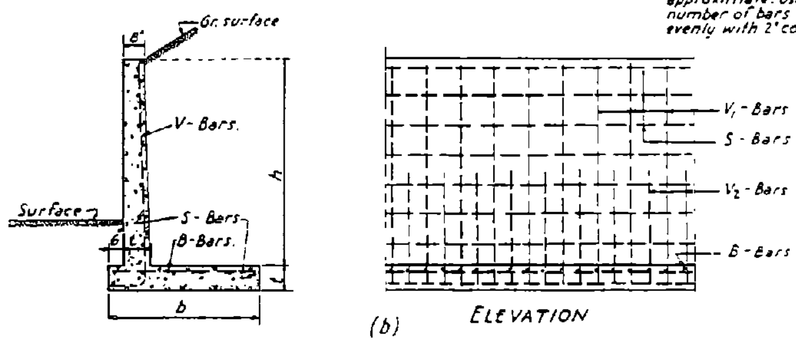


(a)

h	b	t	Vol. Concrete	V-Bars		Lengths		B-Bars			S-Bars			Reinforcem't
(ft.)	(ft-in)	(in)	(Cu yd per ft)	Size	Spacing	V_1-Bars	V_2-Bars	Size	Spacing	Length	Number	Size	Spacing	(Lb per ft)
5	2-9	10	0.22	1/4"φ	12"	6'-6"	—	1/4"φ	12"	2'-6"	8	3/8"φ	12"	4.5
6	3-4	10	0.27	1/4"φ	7"	7'-6"	—	1/4"φ	7"	3'-0"	10	3/8"φ	12"	6.8
7	3-10	10	0.31	3/8"φ	9"	8'-6"	—	3/8"φ	9"	3'-6"	12	3/8"φ	12"	10.4
8	4-6	12	0.41	1/2"φ	12"	9'-8"	5'-0"	1/2"φ	12"	4'-2"	13	3/8"φ	12"	12.6
9	5-0	12	0.46	1/2"φ	9"	10'-8"	5'-4"	1/2"φ	9"	4'-8"	15	3/8"φ	12"	16.9
10	5-6	12	0.51	5/8"φ	11"	11'-8"	6'-2"	5/8"φ	11"	5'-2"	16	3/8"φ	12"	22.1

CANTILEVER RETAINING WALLS
SLOPING SURCHARGE

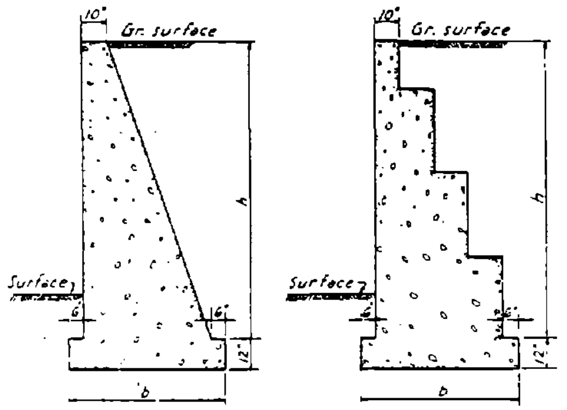
Note. Spacing of S-Bars is approximate. Use tabular number of bars and space evenly with 2" cover



(b)

h	b	t	Vol. Concrete	V-Bars		Lengths		B-Bars			S-Bars			Reinforcem't
(ft.)	(ft-in)	(in.)	(Cu yd per ft)	Size	Spacing	V_1 Bars	V_2 Bars	Size	Spacing	Length	Number	Size	Spacing	(Lb. per ft)
5	3-6	10	0.25	3/8"φ	10"	6'-4"	—	3/8"φ	10"	3'-2"	9	3/8"φ	12"	7.7
6	4-3	10	0.30	1/2"φ	10"	7'-4"	—	1/2"φ	10"	3'-10"	11	3/8"φ	12"	13.1
7	5-0	10	0.35	1/2"φ	7"	8'-4"	—	1/2"φ	7"	4'-8"	13	3/8"φ	12"	19.8
8	6-0	12	0.47	5/8"φ	9"	9'-8"	6'-0"	5/8"φ	9"	5'-8"	15	3/8"φ	12"	24.4
9	7-0	13½	0.59	5/8"φ	7"	11'-0"	6'-8"	5/8"φ	7"	6'-8"	17	3/8"φ	12"	34.1
10	7-9	15	0.71	3/4"φ	9"	12'-2"	6'-10"	3/4"φ	9"	7'-4"	19	3/8"φ	12"	40.9

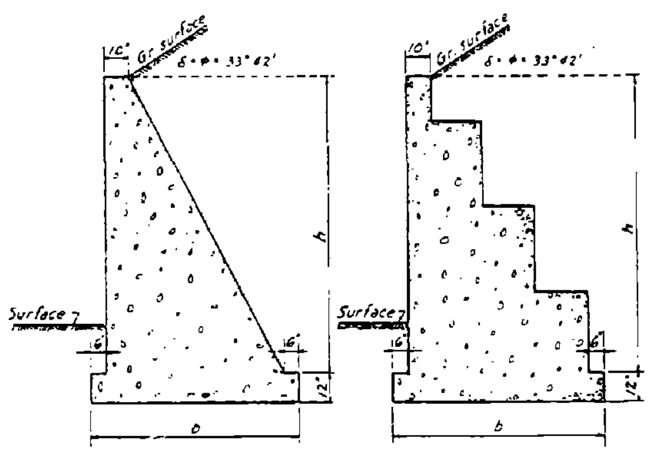
VERTICAL FACED GRAVITY RETAINING WALLS WITHOUT SURCHARGE



h (ft.)	b (ft-in)	Vol. Concrete (Cu yd per ft)
3	2-3	0.20
4	2-7	0.27
5	3-0	0.37
6	3-4	0.48
7	3-9	0.60
8	4-3	0.76
9	4-9	0.94
10	5-3	1.14

(a)

VERTICAL FACED GRAVITY RETAINING WALLS SLOPING SURCHARGE

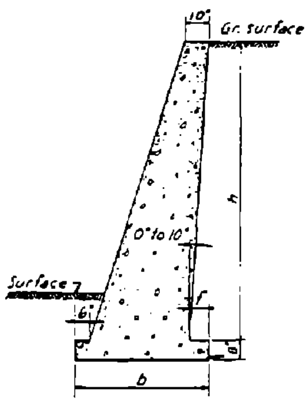


h (ft)	b (ft-in)	Vol Concrete (Cu. yd. per ft.)
3	2-6	0.22
4	3-0	0.32
5	3-6	0.44
6	4-2	0.60
7	4-10	0.78
8	5-6	0.99
9	6-3	1.25
10	7-0	1.52

(b)

BACKWARD INCLINED GRAVITY RETAINING WALLS WITHOUT SURCHARGE

(Inclination from 0° to 10° -- Vertical back to inclination of 1 horizontal to 6 vertical)

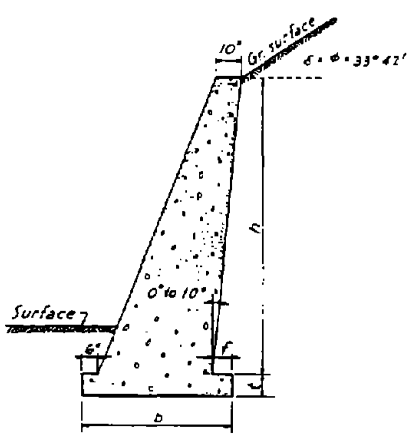


h (ft)	f (in)	b (ft-in)	Vol. Concrete (Cu yd per ft)
3	6	2-0	0.15
4	6	2-4	0.22
5	6	2-8	0.30
6	6	3-0	0.39
7	6	3-4	0.49
8	7	3-9	0.61
9	7	4-1	0.74
10	8	4-6	0.88

(a)

BACKWARD INCLINED GRAVITY RETAINING WALLS SLOPING SURCHARGE

(Inclination from 0° to 10° -- Vertical back to inclination of 1 horizontal to 6 vertical)



h (ft)	f (in)	b (ft-in)	t (in.)	Vol. Concrete (Cu yd per ft)
3	6	2-4	8	0.18
4	6	2-8	8	0.25
5	6	3-0	8	0.34
6	6	3-4	8	0.43
7	7	3-9	8	0.55
8	8	4-2	8	0.67
9	8	4-6	8	0.80
10	9	5-0	9	0.99

(b)

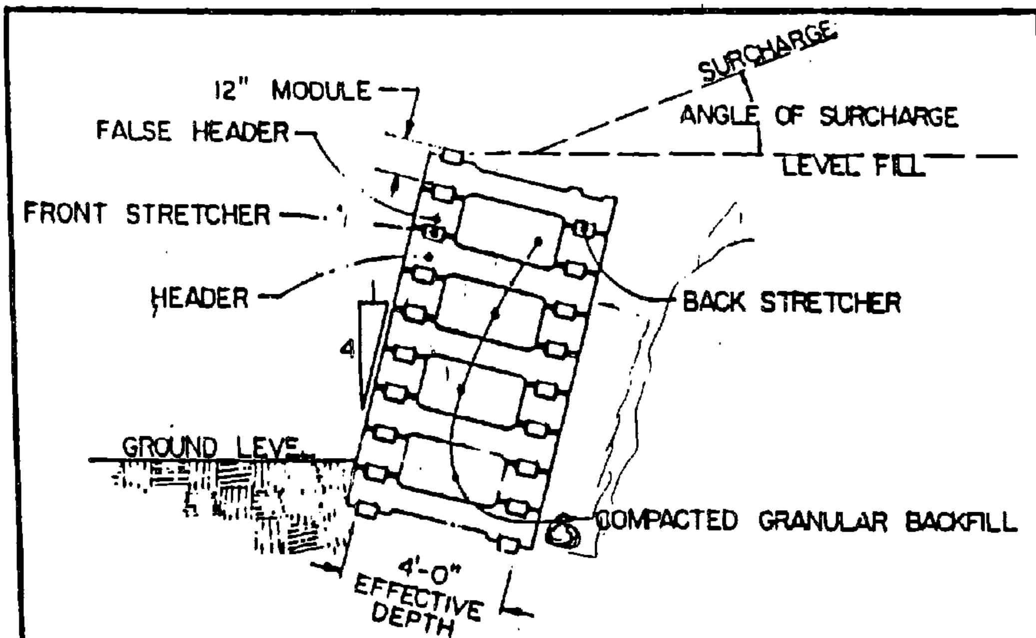
DESIGN ASSUMPTIONS & CONSIDERATIONS

GENERAL

It is assumed that the design of a *CRIBLOCK*® retaining wall will be undertaken only by a qualified civil/structural engineer. Attention must be given to soils data provided by a qualified soils/foundation engineer: *equivalent fluid pressures, coefficient of friction, allowable bearing pressures, passive soil resistance*, as well as identification of any unusual soil conditions, or local factors known to the designing engineer.

DEFINITION

A *CRIBLOCK*™ retaining wall is a composite structure forming a skeletal box which in turn is filled with free draining or suitable onsite material under a drained condition. This composite structure, as a whole, acts as a gravity retaining wall that resists the pressures exerted by the backfill behind the wall through friction at the base of the wall, and by passive pressure developed at the front face of the wall. The tendency toward overturning is, in turn, resisted by the weight of the wall itself. The units are held together by friction at their junction and friction in the fill. Generally, engineering literature pertaining to gravity walls is applicable.



SINGLE DEPTH WALL (TYPICAL)
NO SCALE

For either known or estimated earth pressures, the gravity wall is proportioned so as to provide acceptable minimum safety standards against overturning, sliding and foundation bearing failure. The last requires that the resultant of all forces pass within the middle third of the base, thereby insuring that the heel of the wall is always under compression.

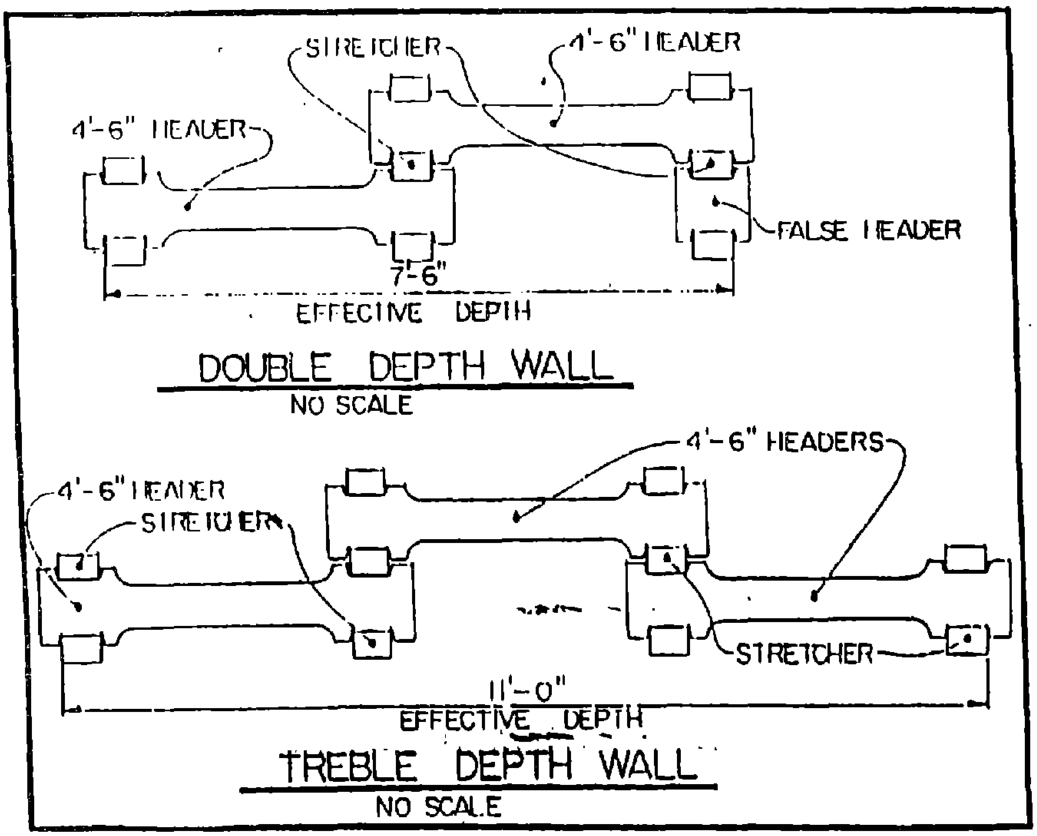
The standard *CRIBLOCK*® design provides for a batter (angle of wall to horizontal) of 1:4; thus for each foot in height the wall will batter back three inches. The degree of batter for a given height of wall has a dramatic effect on the factors of safety against overturning and sliding. The net effect of the batter is twofold:

a. The effect of the active earth pressure is reduced
b. The lever arm of the wall weight from the toe of the wall is decreased

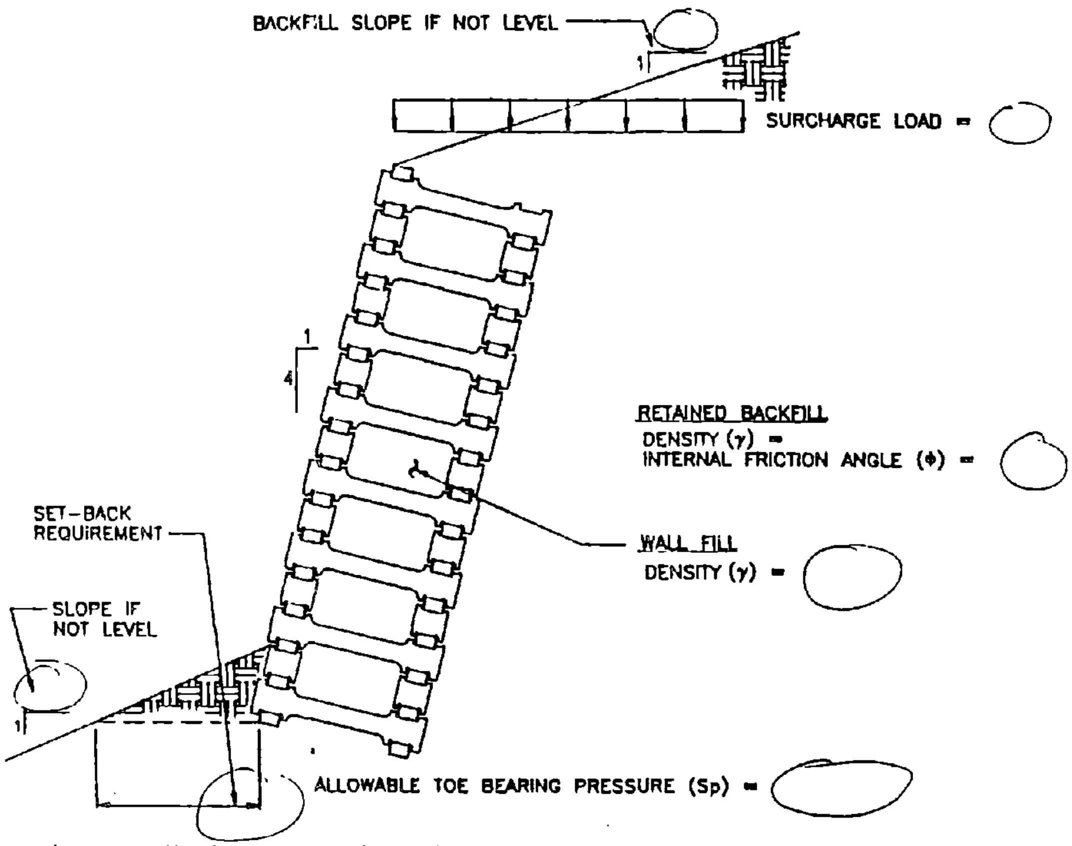
CRIBLOCK® retaining walls may also be erected on a batter of 1:6. This is the standard practice followed in most state and county construction manuals oriented to highway applications.

MULTIPLE DEPTH WALLS

The height limitation of a single depth wall is the function of the design criteria and the resultant calculations. When these factors so dictate, it is necessary to increase the base by building double, triple or greater depth walls. This is accomplished by interlocking headers to create a multi-unit base.



CRIBLOCK RETAINING WALL DESIGN CRITERIA



The above criteria is required to perform a stability analysis. Prior to issuing final design calculations a professional soils engineer needs to certify that the design criteria is acceptable for determining active loads on the wall and for resisting these loads at the base of wall.

PROJECT NAME : DATE :

Soils Engineer

FIRM : PHONE NO. :

ADDRESS : FAX NO. :

SIGNATURE :

ˈITACT :

CRIBLOCK **Retaining Walls Northwest, Inc.**

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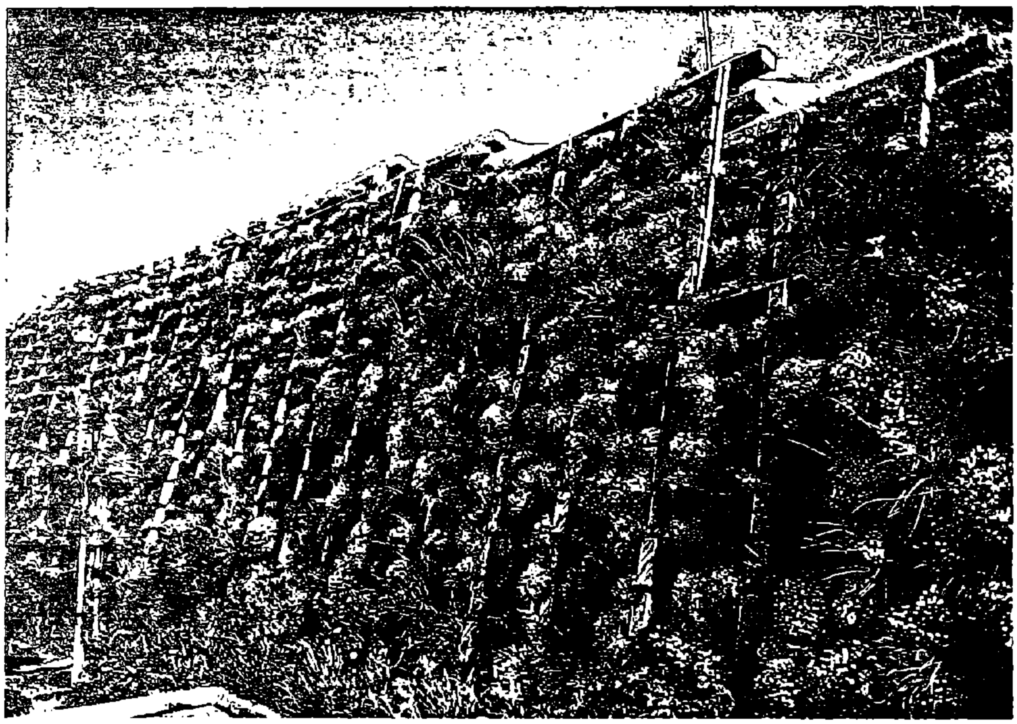

CRIBWALL ESTIMATING

In an effort to provide our clients with as much useful information as possible we have formulated this technical review to be of assistance in estimating the costs of Criblock retaining walls. Every retaining wall is individually designed for the specific site applications unique to both the wall and the project. Several factors have direct bearing on the design of a cribwall. Each of the following factors are considered when designing a cribwall:

- MAXIMUM HEIGHT OF WALL
- SOILS CONDITIONS
- SURCHARGES (i.e. LEVEL, SLOPING, TRAFFIC . . .)
- ANGLE OF REPOSE - VERTICAL, BATTERED
- PROTECTION OF THE WALL TOE
- CURVES AND CORNERS IN WALL ALIGNMENT
- DRAINAGE REQUIREMENTS
- TYPE OF BACKFILL- NATIVE, IMPORTED

TECHNICAL REVIEW — ESTIMATING

Continued from Front Page

Because soils conditions vary widely from region to region, each wall must be considered for the soils native to the site. Taken into consideration are such factors as:

- EQUIVALENT FLUID PRESSURE FOR THE GIVEN SURCHARGE
- BEARING CAPACITY OF THE FOUNDATION MATERIAL
- INTERNAL ANGLE OF FRICTION
- UNIT WEIGHT OF THE SOIL

This soils criteria is usually available through a Soils Report prepared by a qualified Soils Engineer.

When provided with sufficient information Criblock Retaining Walls, Inc. is able to calculate the wall design and provide the client with timely estimates for the amount of cribwall needed and the costs associated with the cribwall. These estimates, even when preliminary, provide the client with accurate information about their project's cribwall needs. At Criblock we strive to give our clients the most accurate design and estimation possible in an expedient manner.

When your projects require retaining walls and you need estimates for quantities and prices we will gladly assist you in both the design and the estimation of your earth retention needs. With the soils report and some supplemental inforr we can give you a preliminary structural desig. and cost estimate for your cribwalls. You can obtain this information by calling our office. Typically, we request a copy of your site plan that shows the wall locations with topography, and a copy of the soils report. After we have compiled the necessary information we do a preliminary computer analysis of the structural requirements specific to the site and soils conditions. We also have available to us computerized cost estimating programs that speed up the estimation process. When estimating your cribwall costs you will want to have the following information at hand:

- BASIC WALL DIMENSIONS
- SOILS REPORT
- PARTICULARS IMPORTANT TO WALL AND SITE

Criblock Retaining Walls, Inc. is a full-service earth retention specialist uniquely qualified to stru ly design, engineer, and build the Criblock s Concrete Cribwalls are both a cost effective and aesthetically pleasing alternative to standard cast-in-place retaining walls. If you would like more information about Cribwalls, or need an estimated cost for your project, contact our office for assistance.

CRIBLOCK

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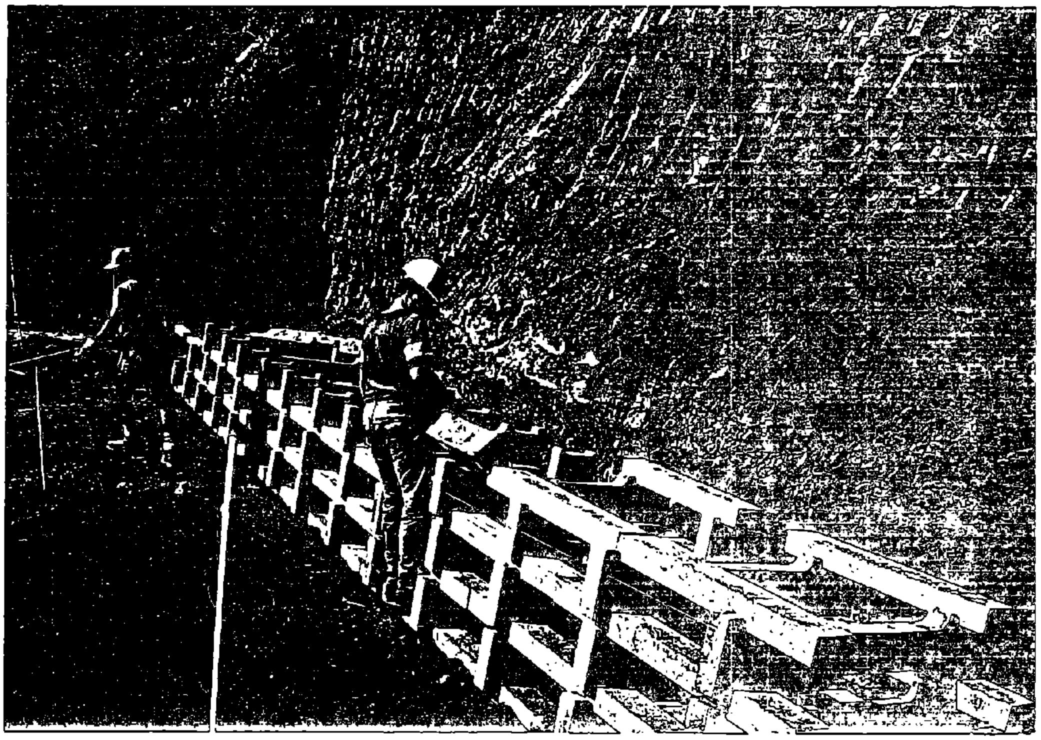


SLOT-CUT CRIBWALLS

Criblock retaining walls are one of the most versitile and cost effective earth retention systems available. Potential applications for the Criblock system include walls constructed in both cut and fill situations. Typically, large retaining walls, (15 feet and up), present numerous problems for construction in cut situations. The modular nature of the Criblock system affords several very unique advantages for construction in these awkward cut situations.

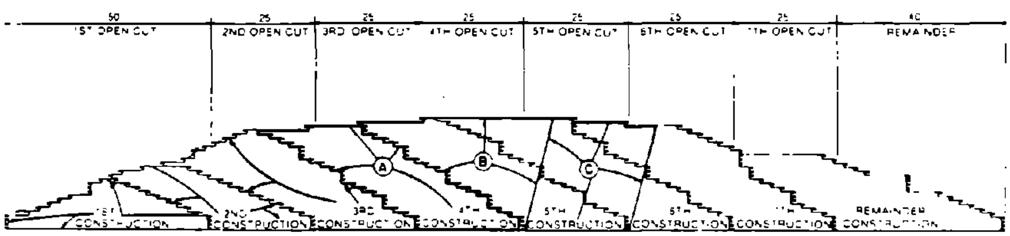
Most earth retention systems require some sort of temporary shoring to stabilize the steep excavations needed to install the retaining wall. The expense for this temporary solution to slope instability often doubles the total costs of the retaining wall installation. Walls needing temporary shoring also require nearly double the construction time of walls not needing shoring. Temporary shoring increases the builder's costs, exposure, liability, and construction time.

Continued from Front Page

Because Criblock is an interlocking, modular system it is ideally suited for use on these higher, cut walls. Criblock retaining walls do not require temporary shoring in most cut applications. The interlocking, modular crib components offer the unique advantage of the "slot-cut" method of construction. Slot-cutting eliminates the need for temporary shoring and requires no special equipment or conditions.

The "slot-cutting" process involves the use of deep, but shorter cuts to provide the grades needed for wall construction. When done under the supervision and direction of a qualified soils engineer these short but deep cuts do not undermine the stability of the slope being exca\ Slot-cutting is done in phases that include both excavation and wall construction. The initial phase usually involves making a 50-70 foot cut at one end of the wall. This cut resembles a sort of notch with a steep backslope. Criblock wall is then stacked and backfilled within this "slot" in a pyramid fashion. (See diagram "A") Once the toe of the excavation has been stabilized with cribwall, then another 40-50 feet of excavation is opened up. Each slot is stabilized with completed cribwall before the next is cut. Successive cuts allow earlier portions of the cribwall to be topped out to finished grade elevations.

DIAGRAM "A"



TYPICAL "SLOT CUT" CONTRUCTION PROCEDURE STEP

1 IMMEDIATELY AFTER 4 TH CUT IS MADE, AREA (A) AND (B) ARE OPEN
2 AFTER 4 TH CONSTRUCITON IS COMPLETED, AREA (B) IS OPEN
3 NEXT STEP IS TO MAKE 5 TH CUT RESULTING IN AREA (B) AND (C) BEING OPEN

NOTES

1 25-FOOT SLOTS ARE SHOWN ABOVE ACTUAL LENGTH CAN VARY AND MUST BE AS SPECIFIED BY SOIL ENGINEER
2 STEPS IN WALL ALONG TERMINAL EDGE OF EACH CONSTRUCTION SEQUENCE CAN VARY IN CONFIGURATION (STEEPER OR SHALLOWER)

Cribwalls 20-30 feet high, with excavations 40-50 deep have been constructed using this unique, cost saving technique. Over-all cribwall installation costs have typically meant savings of 35-45% over the costs of standard retaining walls using temporary shoring. This proven method of construction not only allows for tremendous cost savings, but also provides the builder with an aesthetically pleasing wall that can readily be landscaped.

If you would like more information ab' th Criblock system or the use of "slot-cuttin construction techniques, Criblock Retaining Walls, Inc. is uniquely qualified to assist you.

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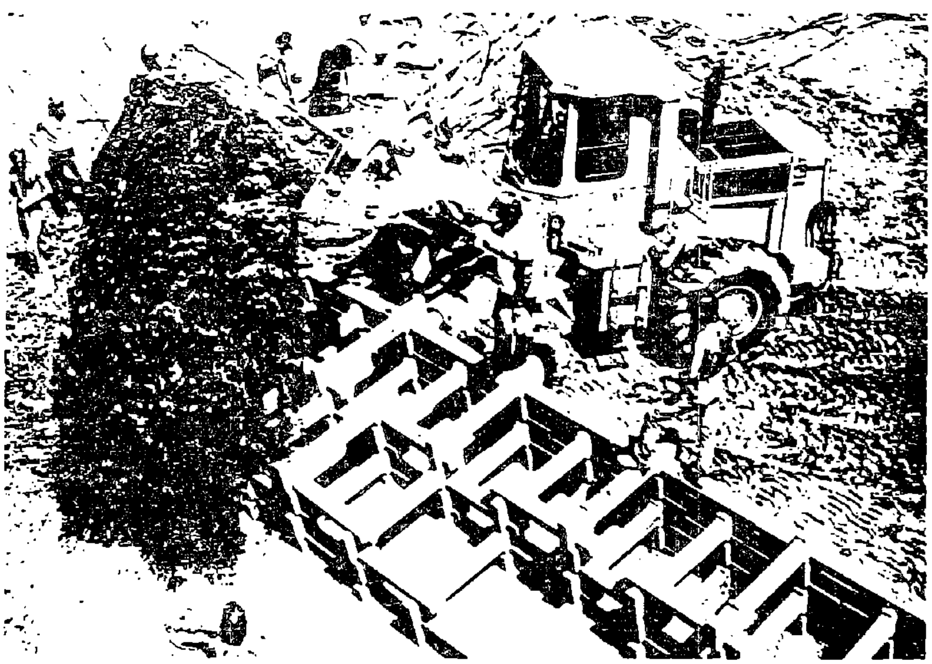


MULTIPLE DEPTH CRIBWALLS

Precast concrete cribwalls are one of the most versatile retaining wall systems available. Criblock cribwalls have been used effectively for many earth retention needs. Cribwalls are commonly used to expand and maximize the building pads for commercial and residential projects; as ramp walls at highway intersections; to widen roadways; for drainage and flood control projects. Crib- can be engineered for walls up to 60 feet high. Because the criblock walls are an inter-locking crib system they are readily adaptable to almost any situation. Multiple depth cribwalls are commonly used for higher walls and for walls in very poor soils. Traditionally, walls over 15 feet high required the use of cast-in-place concrete structures that are expensive and aesthetically limited. Today multiple depth Criblock walls offer a cost-effective alternative for the demands of larger earth retention needs. Besides offering substantial cost savings; cribwalls are asthetically much more versatile and readily lend themselves to improved landscaping concepts. Criblock walls have the unique advantage of serving as both a structurally sound and environmentally pleasing retaining wall system.

TECHNICAL REVIEW — MULTIPLE CRIBWALLS

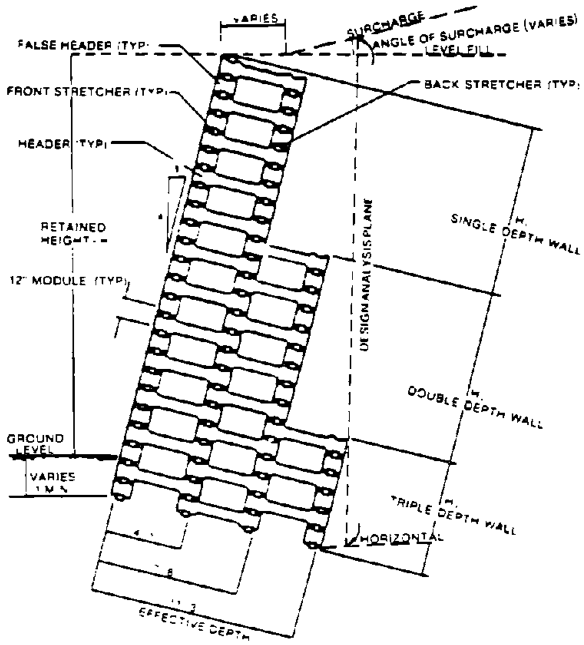
Continued from Front Page

Stages of multiple depth wall can be interspersed with single depth wall depending upon the needs of the particular wall profile. This flexability allows for maximum efficiency in the use of the crib material. Walls can also be designed to be raised in height at a later date without needing to alter the original wall foundation.

Criblock engineers each cribwall for optimum design. The need for multiple depths depends upon several factors such as: wall height, type of surcharge and specific soils conditions. The number of wall depths is then adjusted to guarantee proper factors of safety for sliding and overturning, and to distribute the weight of the wall over a base width acceptable to the specific bearing capacity of the site material.

In addition to varying the number of crib de used, the height of each depth is adjusted to offer the most efficient use of the crib material and to distribute the weight of the structure optimumly between the toe and the heel of the wall. For example, a wall 25 feet high might break out to require 15 feet of single depth, 6 feet of double, and 4 feet of triple depth.

The design flexability afforded the modular, interlocking crib system enable it to be used for walls of all heights in almost any application requiring a retaining wall. Criblock Retaining Walls, Inc. is uniquely qualified to design, engineer, and build the Criblock earth retention system. Please contact our office if we may be of service to you.



CROSS SECTION
TYPICAL MULTIPLE DEPTH WALL

CRIBLOCK

Retaining Walls Northwest, Inc.

Lic # RET-AI-WN 1150J

1299 156th Ave. N.E. #L-150 • Bellevue, WA 98007
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CRIBWALL LANDSCAPING

Perhaps one of the most unique feature advantages of the Criblock system is the aesthetic value of being able to landscape the face of the cribwall. The open cells that compose the face of the wall readily lend themselves to various landscaping concepts. Because the wall is 'cally built on a batter of 4:1 (vertical: ızontal) the sloped face of the wall retains the recompacted soils at a slope condusive to supporting plant life. The battered face of the wall also simplifies the irrigation design should irrigation be called for.

The Enclave cribwall shown above is an excellent example of the feasibility of landscaping cribwalls even in adverse climates such as Colorado. After only one growing season, the wall already supports healthy and colorful plant life. The Enclave cribwall was planted with five different varieties of Ivy and seeded with Honeysuckle. The irrigation system (visable in the background of the photo) is a simple fine-mist spray system readily available in all parts of the country. These fine-mist sprayers not only conserve water but also reduce the risk of over

TECHNICAL REVIEW — LANDSCAPING

Continued from Front Page

saturating the backfill within the cribwall. The Enclave wall required only one row of sprinklers across the top coarse even though the wall reaches 14 feet in height.

Criblock walls can be landscaped in almost any climate. Typically, native plant varieties that require little or no additional irrigation are used. Planting of the wall face is usually accomplished by "hydro-seeding" and/or hand-planting. Planting schemes can include multiple growth stages that may allow for heartier, rapid growth plants to fill in during the initial stages while the more colorful plants have time to take root and develop. Numerous landscaping themes have been successfully used in cribwalls. These themes required that various straints be considered, such as: walls in highly visable locations requiring colorful and dense growth; walls for highway projects that must utilize native plant varieties but cannot be irrigated; plantings that are low maintenance for use in areas with limited access; subterranean walls that receive little or no direct sunlite; and plantings designed to endure the rigors of extreme temperatures found at altitudes over 8,000 feet.

For further information on landscaping Criblock retaining walls please contact our office, or consult a qualified Landscape Architect.

CRIBLOCK Retaining Walls Northwest, Inc.
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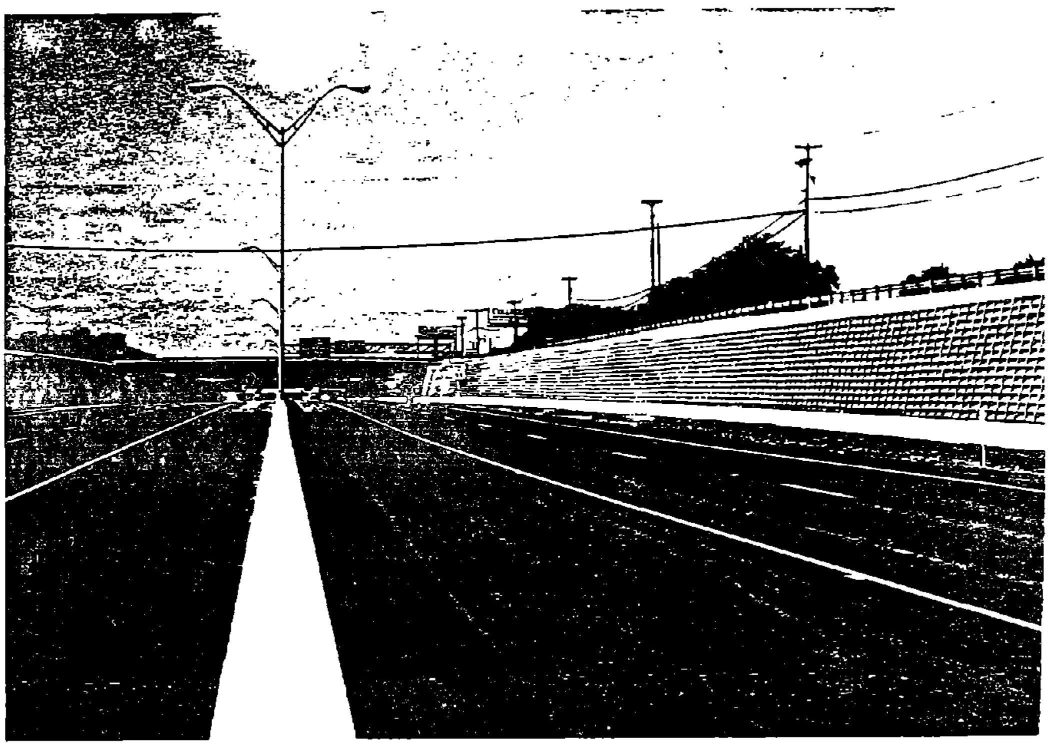


HIGHWAY APPLICATIONS

Criblock retaining walls are possibly the most versatile wall systems available today. One important application of the Criblock system is the use of cribwalls in Federal and State highway projects. Cribwalls have been successfully used for numerous earth retention needs such as:

- INTERCHANGE RAMP WALLS
- GRADE SEPARATIONS
- BRIDGE ABUTMENTS
- WING WALLS
- SOUND ATTENUATION WALLS

Because of their aesthetic landscaping qualities, cribwalls are being used more and more in various highway applications. The design flexibility afforded by cribwalls allows for both vertical and battered wall designs. Vertical cribwalls are most commonly used when there is limited right-of-way in which to place the wall. Typically, these walls utilize a closed face panel inserted inside the front of the wall to retain the soils within the wall. Several installations have made use of a topical, acrylic color-coating applied to the surface of the finished wall for architectural continuity with surrounding structures.

TECHNICAL REVIEW — HIGHWAY

Continued from Front Page

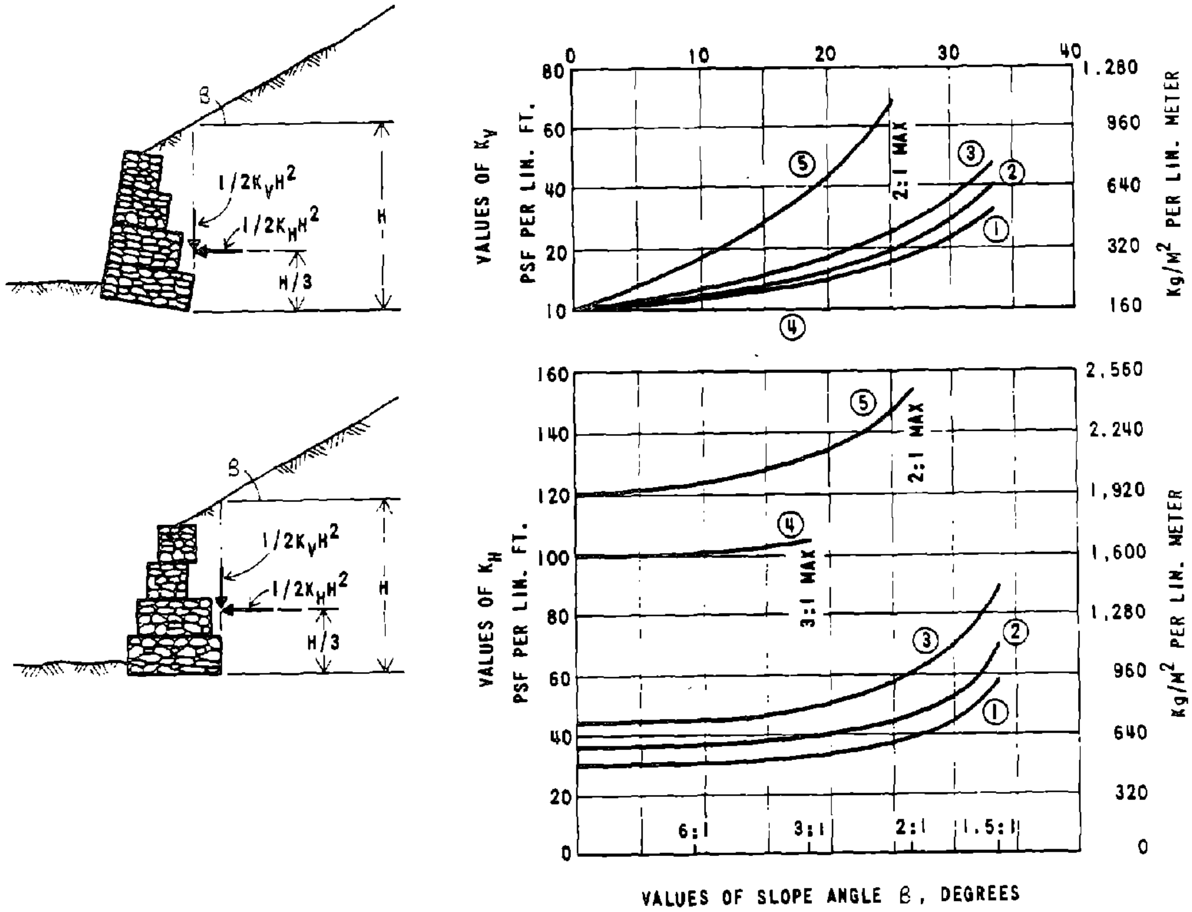
Cribwalls designed and built with a batter of at least 4:1 (vertical : horizontal) readily lend themselves to being landscaped with native vegetation. The landscaped cribwall concept has received extensive use in heavily traveled corridors where landscaping is of primary importance. Many of these corridors don't have the room for greenbelts composed of gentle slopes; thus the landscapability of the Criblock system is ideally suited for providing an appealing alternative to deep concrete traffic channels. Metropolitan freeway systems are well suited for this sort of application.

The Criblock system has been approved I Federal Highway Administration and by virtua.., a.. state highway departments.

In addition to providing technical assistance and consultation to those designing retaining walls for highway projects, Retention Engineering can provide complete civil and structural engineering design, specifications, and drawings for inclusion in bid documents. If we can be of assistance to you on highway projects that require retaining walls, please contact our office.

DESIGN LOADS FOR LOW GABION RETAINING WALLS

(STRAIGHT SLOPE BACKFILL)



CIRCLED NUMBERS INDICATE THE FOLLOWING SOIL TYPES:

1 CLEAN SAND AND GRAVEL : GW, GP, SW, SP.

2 DIRTY SAND AND GRAVEL OF RESTRICTED PERMEABILITY: GM, GM-GP, SM, SM-SP.

3 STIFF RESIDUAL SILTS AND CLAYS , SILTY FINE SANDS, CLAYEY SANDS AND GRAVELS: CL, ML, CH, MH, SM, SC, GC.

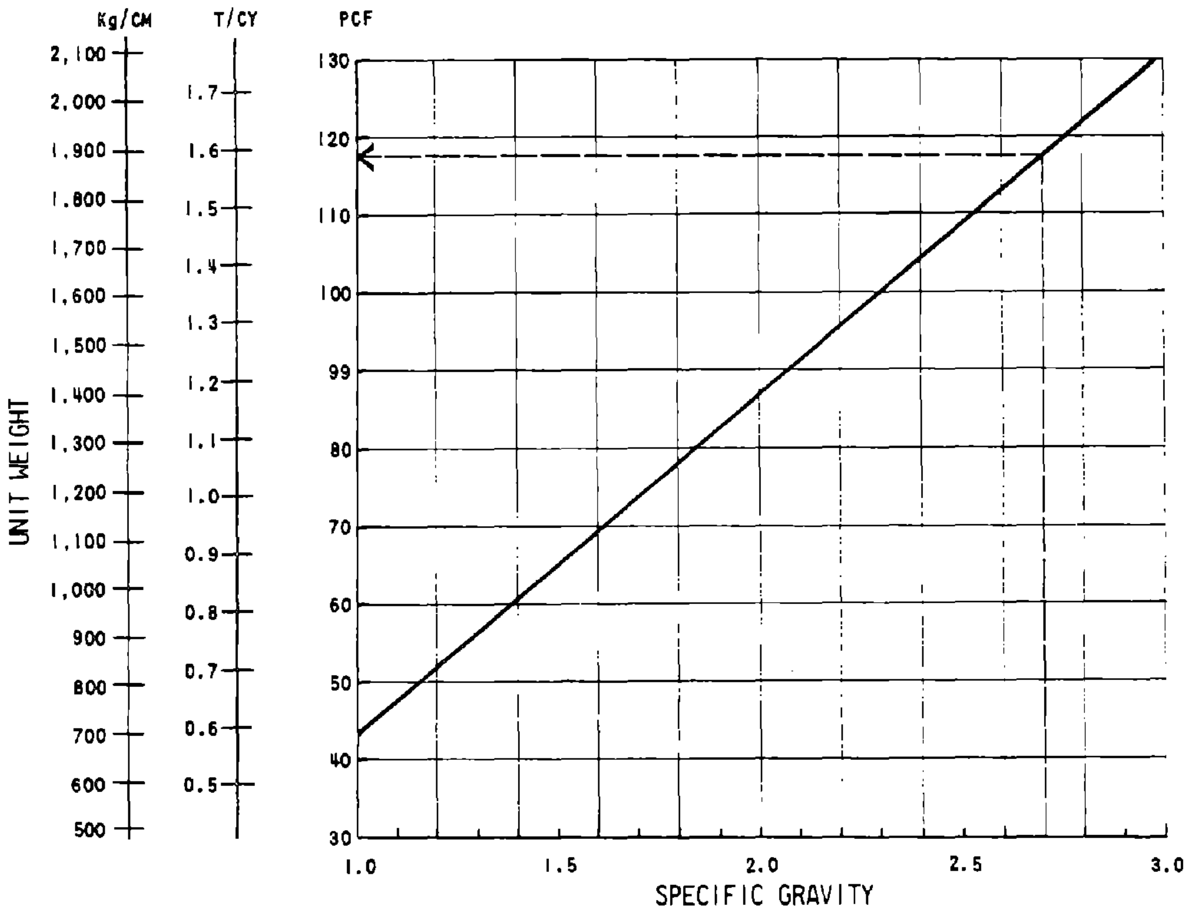
4 VERY SOFT TO SOFT CLAY, SILTY CLAY, ORGANIC SILT AND CLAY: CL, ML, OL, CH, MH, OH.

5 MEDIUM TO STIFF CLAY DEPOSITED IN CHUNKS AND PROTECTED FROM INFILTRATION: CL, CH.

FOR TYPE 5 MATERIAL, H IS REDUCED BY 4 FT, RESULTANT ACTS AT A HEIGHT OF (H-4)/3 ABOVE THE BASE

UNIT WEIGHT OF GABION FILL

(BASED ON A POROSITY OF 0.30)



SPECIFIC GRAVITY OF COMMON MATERIALS

BASALT	3.0
BRICK	2.0
CONCRETE (BROKEN)	2.4
GRANITE	2.7
LIMESTONE	2.5
SANDSTONE	2.2
TRAP ROCK	2.7

EXAMPLE

GIVEN : SPECIFIC GRAVITY : 2.7

FIND : UNIT WEIGHT IN a) PCF
b) T/CY
c) Kg/CM

SOLUTION

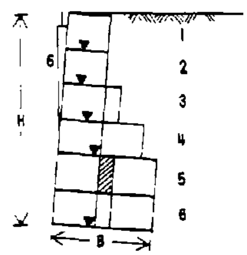
PROCEED VERTICALLY FROM S.G.: 2.7 TO INTERSECTION OF DIAGONAL LINE. THEN PROCEED HORIZONTALLY TO INTERSECTION OF VERTICAL LINE AND FIND:

a) UNIT WEIGHT : 118 PCF
b) UNIT WEIGHT : 1.59 T/CY
c) UNIT WEIGHT : 1,890 Kg/CM

GABION WALLS

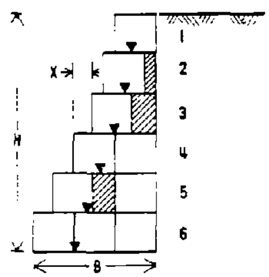
CASE I, β = 0°

a) FRONT FACE ON 1:6 BATTER



No OF COURSES	H	B	REMARKS
1	3'-3"	3'-3"	FRONT FACE MAY BE VERTICAL
2	6'-7"	3'-3"	
3	9'-10"	5'-0"	
4	13'-1"	6'-7"	
5	16'-5"	8'-3"	
6	19'-8"	8'-3"	HATCHED PORTION IN COURSE 5 DOES NOT REQUIRE GABION MESH

b) FRONT FACE STEPPED

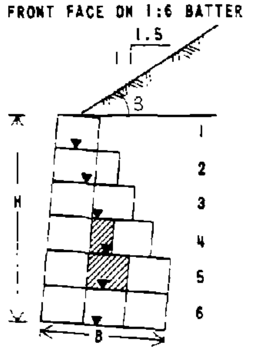


No OF COURSES	H	B	X	REMARKS
1	3'-3"	3'-3"		
2	6'-7"	4'-3"	12"	
3	9'-10"	5'-3"	12"	HATCHED PORTION OF COURSE 2 DOES NOT NEED GABION MESH.
4	13'-1"	6'-7"	16"	HATCHED PORTION OF COURSE 2 & 3 DOES NOT NEED GABION MESH.
5	16'-5"	8-3"	18"	USE COUNTERFORTS AT 9-9 IN COURSE 4. SEE NOTE FOR COURSE 4.
6	19'-8"	9'-10"	18"	SEE NOTES FOR COURSES 4 AND 5.

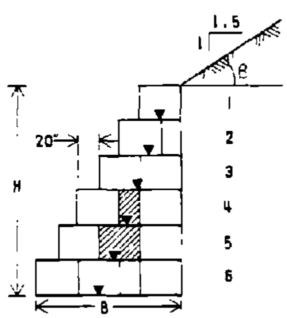
▼ LOCATION OF RESULTANT OF WEIGHT OF WALL AND EARTH PRESSURE

CASE II, β = 33°41'

a) FRONT FACE ON 1:6 BATTER



No OF COURSES	H	B	REMARKS
1	3'-3"	3'-3"	FRONT FACE MAY BE VERTICAL.
2	6'-7"	5'-0"	
3	9'-10"	6'-7"	
4	13'-1"	8-3"	
5	16'-5"	9'-10"	HATCHED PORTION OF COURSE 4 DOES NOT NEED GABION MESH.
6	19'-8"	9'-10"	HATCHED PORTIONS OF COURSES 4 & 5 DO NOT NEED GABION MESH.



b) FRONT FACE STEPPED

	H	B	REMARKS
1	3'-3"	3'-3"	
2	6'-7"	5'-0"	
3	9'-10"	6'-7"	
4	13'-1"	8'-3"	
5	16'-5"	9'-10"	HATCHED PORTION OF COURSE 4 DOES NOT NEED GABION MESH.
6	19'-8"	11'-6"	HATCHED PORTIONS OF COURSES 4 & 5 DO NOT NEED GABION MESH.

Appendix F
Specification for Tied Back Retaining Walls

Description

This work shall consist of furnishing and installing tied back retaining walls at designated locations in accordance with these specifications, and in reasonable conformity with the lines, grades, and dimensions shown on the drawings.

Materials

Structural Steel Piling. Structural steel piling shall be rolled steel sections of the weight and shape called for on the drawings, and it shall meet the requirements of AASHTO M-183.

Rock Bolts. Anchors and connectors shall be the size and type called for on the drawings, or an approved equivalent. The rock bolt ultimate design capacity shall be 75,000 pounds. The tie rod connection shall have a minimum yield strength of 36,000 psi.

Miscellaneous Metal

Bolts. Bolts, nuts, and washer material shall conform to ASTM A-325 for high strength bolts for structural steel joints, including suitable nuts and plain, hardened washers.

Dowels. Reinforcing bar used for doweling shall conform to requirements of AASHTO M-31 and be a grade 40, unless otherwise designated in the plans.

Forgings. Forgings shall conform to the requirements of ASTM A-237 for class A alloy steel forgings.

Miscellaneous Metals. Miscellaneous structural steel and connections required shall conform to the requirements of AASHTO M-183.

Treated Timber. Timbers shall be Pacific Coast Douglas fir species structural grade and conform to AASHTO M-168. Treatment will be in accordance with AASHTO M-133 and consist of pentachlorophenol. After treatment, timbers shall have a minimum retention of .6 pcf and .5 penetration.

Construction

Preparation of the Site. Preparation of the retaining wall foundation shall be as follows:

(1) Rough excavation for the site of the wall shall be made to elevation of the finished ground line at the face of the wall. Where the wall is to be set upon a solid rock foundation, a bench shall be prepared for the vertical pile section. The minimum width normal to the long axis of the wall shall be a minimum of 24 inches. The other dimension shall be as required.

(2) Existing culverts and retaining logs shall be removed as shown, prior to the installation of the wall.

Installing Anchorages and Wall Members

(1) Installing Bottom Anchorages. The anchor rod at the bottom of the vertical pile section shall be set in a predrilled hole. The rod shall penetrate a minimum of 18 inches into the rock.

(2) When the indicated wall height exceeds 8 feet or the fractured rock is encountered, an additional horizontal tieback shall be installed at the bottom of the wall, as shown on the drawings.

(3) The top connection to the pile shall be made as shown on the drawings.

(4) The anchorage for the top tieback shall consist of a drilled-in rock bolt, as shown. An alternate design will be used for common material if encountered, or as directed by the engineer.

Installing Lagging

(1) The bottom course of timber lagging shall be trimmed to bear uniformly on the ground's surface. The cut area shall be field-treated with three coats of the preservative specified in article 2.4. All holes bored after treatment shall be impregnated with hot creosote oil or other approved treatment method.

(2) To secure the cut ends, an additional section of piling shall be welded across the adjacent vertical members where the bottom course of lagging is stepped. As a second alternative on wall ends, the unsecured ends may be doweled to the rock as shown on the plans.

(3) Surfaces of subsequent courses shall be in uniform contact across the full width of the wall.

Backfilling. All portions of the structures shall be in true alignment and approved by the engineer before backfilling is started. The filling behind the wall shall be of approved material, placed in layers not to exceed 12 inches in thickness, and shall be mechanically tamped. All organic material and unsuitable soil shall be excluded from the backfill. Select material for backfill shall be provided from specified borrow sources. Compaction of the backfill shall be to the equivalent of 90 percent of maximum density as determined by AASHTO T-99, method C. As this method is not applicable in coarse materials, this will be determined visually by the engineer to be the point at which negligible consolidation of the layer occurs with additional compactive effort.

Special Project Specification 650–1
Anchored Sheet Pile Retaining Wall

Description

This work shall consist of furnishing and installing anchored sheet pile retaining walls at designated locations in accordance with these specifications, and in reasonable conformity with the lines, grades, and dimensions shown on the drawings.

Materials

Structural steel sheet piling. Structural steel sheet piling for retaining walls and anchor walls shall be LBF-1707 sheet piling, or an equivalent as approved by the project engineer. Sheet piling shall meet the requirements of ASTM A328.

Miscellaneous structural steel

(1) Steel tie rods, plates, and channels shall be of the sizes and dimensions as shown on the drawings, and they shall conform to the requirements of ASTM A6 and ASTM A36.

(2) Bolts, nuts, and washer material shall conform to ASTM A325 for high strength bolts for structural steel joints, including suitable nuts and plain, hardened washers.

(3) Miscellaneous structural steel and connections required shall conform to the requirements of AASHTO M-183.

PVC pipe. PVC pipe shall be schedule 40 pipe of the sizes and dimensions as shown on the drawings, and they shall conform to the requirements of ASTM 01785.

Construction

Excavation. Structural excavation for the wall shall be to the elevations shown on the drawings. Excavated material shall be stock-piled for use during backfilling, if material is suitable in the opinion of the project engineer. If unsuitable, excavated material shall be wasted at the locations designated on the drawings.

Driving of sheet piling. Steel sheet piling shall be furnished, handled, and driven to the minimum depths shown on the drawings. Sheet piling shall be driven with the interlocks on the outside of the retaining wall.

After driving, 1-inch-diameter weep holes shall be burned in each sheet pile used for the retaining wall just above the finish grade line on the outside of the wall.

Installation of wales and anchor rods. Wales, anchor rods, and structural connections shall be assembled and installed as shown on the drawings. Members shall be handled carefully, and any members which are structurally damaged shall be replaced. Channels shall be bolted back to back as shown prior to welding to sheet piling. Welds shall be made using shielded metal arc process with AWS A5.1, AWS A5.5, or E6OXX welding rods. If the specified welded connections cannot be properly made due to the alignment of the sheet piling, then the connection shall be modified as approved by the project engineer. Threaded ends of tie rods shall be damaged or nut shall be welded to tie rod after completion of backfilling and approval of project engineer. Splices and sleeve nuts as shown on the drawings may be used at locations as approved by the project engineer.

Backfilling. All portions of the structure shall be in true alignment and approved by the project engineer before backfilling is started. The backfilling behind the wall shall be of approved material, placed in horizontal layers between 9- and 18-inch depth and compacted. Select backfill shall be provided from structural excavation material, if suitable, or from specified borrow sources. Sheathing material shall meet the requirements of the special project specifications. All organic material and unsuitable soil shall be excluded from the backfill material. Compaction of sheathing material and select backfill shall be to a uniform density not less than 90 percent of maximum density, as determined by AASHTO T-99, method C. Backfill within 2 feet of the retaining wall or anchor wall shall be compacted with approved hand tampers or compactors. Alignment of retaining wall and anchor wall shall be maintained during backfilling.

Method of Measurement

Anchored sheet pile retaining walls and anchor walls shall be measured in linear feet of sheet piling complete in place and accepted, and includes furnishing, burning of weep holes, and driving or otherwise, installing the required lineal footage of sheet piling.

Tie rods shall be measured as the number of tie rods complete in place and accepted, and includes furnishing and installing (including welding) all structural steel and PVC pipe required per the drawings for connecting the retaining wall and the anchor wall.

The volume of structural excavation will not be measured, but it will be the number of cubic yards shown on the drawings. Select backfill will also not be measured, but it will be incidental to the structural excavation.

Basis of Payment The accepted quantities will be paid for at the contract unit price shown in the schedule of items.

Pay item	Pay unit
650 (01) Structural excavation	Cubic yard
650 (02) Furnishing and driving sheet piling	Linear foot
650 (03) Furnishing and installing tie rods	Each

Oregon Department of Transportation Specifications—Retaining Walls

Description and Scope

This work consists of constructing mechanically stabilized earth (MSE) retaining walls, metal bin walls, rock gabions, cast-in-place concrete retaining walls, or other approved retaining wall systems at locations shown or directed.

Construct geotextile retaining walls according to section 00350.

The retaining wall system to be used will be shown on the plans and specified in the special provisions. If alternate types or systems are allowed, select and submit an option from the list of alternate retaining wall types or systems in the special provisions.

All wall systems must meet the requirements in the policy document *Acceptance Criteria for Retaining Wall Systems* that is available from the project manager. Alternate retaining wall systems may be acceptable depending on specific project constraints. Submittals shall conform to 00395.03.

Definitions

Mechanically stabilized earth (MSE) walls. Retaining walls composed of nonextensible, metallic elements in combination with specified frictional soil to form a composite of reinforced soil volume. Precast concrete facing panels are used at the face to prevent erosion of the reinforced volume and to provide an acceptable appearance.

Metal bin walls. Prefabricated members with fittings and appurtenances for field assembly into a retaining wall composed of metal cells filled with approved stone embankment material.

Rock gabions. Assembled wire mesh baskets filled with specified rock, connected together, and placed with a riprap geotextile between the back face of the gabions and the backfill material.

> <u>Connecting wires</u>. Internal wires used to prevent gabions from bulging.
>
> <u>Tie or spiral binding wire</u>. Wire used to assemble and join the gabion units.
>
> <u>Fasteners</u>. Clips or rings used to assemble individual gabion units as an alternate to tie or spiral binding wire.

Variables

The amount and type of work required to construct a wall vary according to wall design selected. The contractor is responsible for making a careful study of the work required under each wall design and for determining the variables in quantities.

Variables could involve shoring, excavation, backfilling, excess material, staging work, corrosion protection, and other details of the work.

Select the wall type from the approved list of retaining walls in the special provisions.

Alternate Retaining Wall Systems Submittals

If alternate retaining wall systems are permitted in the special provisions, submit shop drawings, design calculations, and a field construction manual to the engineer for approval according to 00150.35. Submit alternate retaining wall designs before wall fabrication at least 30 calendar days before beginning retaining wall construction for the engineer's review according to 00150.35(b-4). Obtain the engineer's written approval before fabricating the walls.

The shop drawings and design calculations shall be prepared by or under the direction of a professional engineer licensed with the State of Oregon and shall bear his or her signature and seal. The contractor's submittal shall meet the following design criteria and include the following applicable details:

(1) **Plan and elevation sheets.** A plan and elevation sheet or sheets for each wall which includes:

Elevation view. Indicates the following: the elevation at the top of the wall, at all horizontal and vertical break points, and at least every 50 feet along the wall; elevations at the top of the leveling pads and/or footings; the distance along the face of the wall to all steps in the footings and/or leveling pads; the designation as to the size and type of panel or module; the length, size, and number of mesh or strips; the distance along the face of the wall to where changes in the length of the mesh or strips occur; and the location of the original and final ground line at both the heel and face of the wall.

Plan view. Indicates the offset from the construction centerline to the face of the wall at all changes in horizontal alignment, the limits of the widest module, mesh or strip, and the center line of any drainage structure or drainage pipe which is behind, passes under, or through the wall.

General notes. For design and construction of the alternate retaining wall option selected.

Curve data. All horizontal and vertical curve data affecting wall construction.

Quantity summary list. For all items of each wall, including incidental items.

Limits. Of granular backfill material and the extent of reinforced soil volume.

Structural and geometric details. All structural and geometric details, include reinforcing, bar-bending details.

Foundation and/or leveling pads. Include details in the steps for the footings or leveling pads and the actual maximum bearing pressures. Include pile information if piles are used.

Facing elements. Show all dimensions necessary to construct the element, all reinforcing steel in the element, and the location of reinforcement element attachment devices embedded in the facing. Show the type of concrete finish.

Details. Provide details for the following:

- Specified strip or mesh length, panel thickness, loading conditions, size of concrete leveling pad, and details for appurtenances.

- Construction of the wall around drainage facilities, overhead sign footings, abutment piles, or other structures.

- Connection to traffic barriers, coping, parapets, sound walls, fences, and attached lighting.

(2) **Design Criteria Calculations.** Design calculations shall do the following:

- Conform to the requirements for service load design, of the current AASHTO standard specifications for highway bridges, and interims.

- Include a detailed explanation of any symbols and computer programs used in the design of the walls. Design all walls to meet factors of safety for overturning, sliding, and pullout resistance of 2.0, 1.5, and 1.5, respectively.

- Show the maximum toe pressure at the base of the wall.

- Use the following values for MSE walls:

	Angle of Internal Friction	Unit Weight (lbs/ft^3)
Soil retained by wall	30°	120
Granular backfill in wall	34°	120
Soil below wall	30°	*

- Dependent on water table, unless specified in the special provisions.

- Limit the allowable bearing pressure of abutment spread footings constructed directly on mechanically stabilized earth structures to 2.5 tons per square foot.

- Provide tensile stresses in steel for MSE walls within the following limits: reinforcing strips and connections, (0.55 Fy) and, at welded connections of transverse grid members (0.47 Fy).

- Provide longitudinal and transverse grid members of the same size.

- Verify the adequacy of traffic barriers, sound walls, sign supports or luminaire supports and the supporting wall to resist the design loadings if the wall is detailed to provide such support.

- Provide a minimum service life of 75 years for all components of the wall.

(3) **Field construction manual**. Prepared by the manufacturer of the wall option selected providing step by step directions for constructing the retaining wall system.

Materials

General. Obtain all manufactured materials for the retaining wall option selected from the same company. Only one option of wall will be allowed on the project, unless different systems are called for by the plans, special provisions, or if they are approved.

Mechanically Stabilized Earth (MSE) Retaining Walls

(1) **General**. This specification covers the following systems: retained earth (system 1), reinforced earth (system 2), and Hilfiker (RSE) retaining walls (system 3).

Acceptance will be based on manufacturers' test results and certificate of compliance according to 00165.60, visual inspection, and other criteria in this section.

Purchase the facing elements, reinforcing mesh or strips, attachment devices, joint filler, and all other necessary components from any one of the following manufacturers:

System 1	Retained Earth Company c/o VSL Corporation 1077 Dell Avenue Campbell, CA 95008 408-866-5000
System 2	Reinforced Earth Company 22619 S.E. 64th Place, Suite 240 Issaquah, WA 98027 206-391-0111
System 3	Hilfiker Retaining Walls P.O. Drawer L 3900 Broadway Eureka, CA 95501 707-443-5091

Do not use materials from sources not listed without a written approval.

(2) **Wall facings**

General. Provide precast concrete panel wall facings, unless otherwise specified. Fabricate concrete panels according to section 00540 with the following exceptions and additions. Full-height facing panels will not be allowed unless specified.

Portland cement concrete. Use class 4000:1-1/2, 1, or 3/4 concrete. Do not use chemical admixtures other than for air entrainment.

Casting. The loop embeds, tie strips guides, or other galvanized devices shall not contact or be attached to the face panel reinforcement steel. Set tie strip guides and loop embeds on the rear face.

Use clear form oil of the same manufacturer throughout the casting operation.

Curing. Fully support and do not strip the forms from the units until the concrete reaches a minimum compressive strength of 1,000 psi.

Finish. Give the front face a general surface finish according to 00540.52, unless otherwise specified.

Tolerance. Manufacture all units within the following tolerances:

- **Panel dimensions**. The difference between diagonals shall not exceed 1/2 inch. All other dimensions shall be within 3/16 inch, and angular distortion with regard to the height of the panel shall not exceed 0.2 inch in 5 feet.

- **Tie strips and loop embeds**. Panel tie strip connection devices shall be within 1 inch and loop embeds within 3/16 inch of the plan location.

- **Panel Face**. Smooth-formed surfaces measured over a length of 5 feet shall not vary over 1/8 inch. Textured-finished surfaces measured over a length of 5 feet shall not vary over 5/16 inch.

- **Rear face**. Eliminate surface distortions over 1/4 inch.

Acceptance of Concrete Acceptance will be according to 00540.16, except as follows:

- Acceptance of concrete panels will be determined on the basis of production sublots. A production sublot will be represented by a single compressive strength sample and will consist of either 40 panels or a single day's production, whichever is less. Cast one set of cylinders for each production sublot.

- Acceptability of the precast unit will be determined by compressive strength tests results, check tests, and visual inspection. Panels may be placed in the wall if the 7-day initial strength exceeds 85 percent of the 28-day requirements. Final acceptance will be based on the 28-day test results.

Marking. Clearly scribe on the rear face of each panel the date of manufacture, the production sublot number, and the piecemark.

Handling, Storage, and Shipping. Handle, store, and ship all units in a manner that eliminates chipping, discoloration, cracks, fractures, and connecting device damage. Support stored panels on firm blocking.

Rejection. Any one of the following defects will be cause for rejection: imperfect molding, honeycombed or open texture concrete, improper installation of loop embeds or tie strips, broken or chipped concrete, extreme color variation on front face of panel, or nonspecification strength.

(3) **Soil reinforcing and attachment hardware**

Shop-fabricate true size and defective free-reinforcing and attachment hardware galvanized according to 02530.70, except where noted, as follows:

Repair damaged galvanized components before placing backfill material to provide a galvanized coating comparable to that provided by 02530.70.

Reinforcing strips. Use steel reinforcing strips of the required shape and dimensions according to AASHTO M 183 or an equal.

Reinforcing mesh. Furnish welded wire fabric according to 02510.40.

Tie strips. Provide tie strips from steel according to the requirements of ASTM A 570, grade 50 or an equivalent.

Loop embeds. Fabricate 1-inch loop embeds from cold-drawn steel wire according to AASHTO M 32. Weld loop embeds according to ASTM A 185. Galvanize loop embeds according to ASTM B 633 or an equivalent.

Fasteners. Use galvanized high strength bolts according to 02560.20.

Connectors. Fabricate connectors from cold-drawn steel wire according to AASHTO M 32.

(4) **Joint materials**

Provide joint materials to the dimensions and thicknesses shown on the plans or approved shop drawings.

Bearing pads. In horizontal joints between panels, provide either preformed EPDM rubber pads according to ASTM D 2000 4AA, 812 A13 B33 C12 F17, having a durometer hardness of 80±5, or neoprene elastomeric pads having a durometer hardness of 55±5, or other bearing material as recommended by the supplier, certified by the manufacturer, and approved by the engineer.

Joint cover. Use a wall geotextile according to 02320 over all joints at the backface of panels. Use an adhesive recommended by the supplier and manufacturer and approved by the engineer to attach the geotextile material to the rear of the facing panels.

Use a minimum 12-inch-wide geotextile with an overlap of at least 4 inches.

(5) **MSE granular backfill material**

Use MSE granular backfill material free from unsuitable materials and according to the following gradation as determined by AASHTO T 27:

Sieve size passing	Percentages
4"	100
1/4"	20–100
No. 40	0–60
No. 200	0–15

The plasticity index of the MSE granular backfill material passing the no. 40 sieve shall not exceed 6 when tested according to AASHTO T 90.

The MSE granular backfill material shall also conform to the following electrochemical requirements:

Property	Limits	Test procedures
PH	4.5 (min.), 9.5 (max.)	
Resistivity	5,000 ohm/cm (min.)	ASTM G 57
Organic content	0	AASHTO T 267

MSE granular backfill material with resistivities of less than 5,000 ohm/cm but greater than 2,000 ohm/cm may be accepted if they meet the following additional requirements:

Property	Limits	Test procedures
Chlorides	100 PPM (max.)	ASTM D 512 or ASTM D 4327
Sulfates	200 PPM (max.)	ASTM D 516 or ASTM D 4327

(6) **Footings, copings, and leveling pads**

Use reinforcement steel according to section 00530. Use class 3300:1-1/2, 1, or 3/4 concrete according to section 00540.

Metal Bin Walls

(1) **Base metal, galvanizing, and gauges**. Design all members, fittings, and appurtenances as integral units or parts of the whole assembly.

(2) **Fabrication**. Galvanized steel used in fabricating the members according to AASHTO M 218. Use galvanized bolts, nuts, and miscellaneous hardware of sizes and shapes designed specifically for use with the members furnished.

Fabricate the members from specified base metal of the gauges shown on the designs and plans. In the absence of given gauges or dimensions for any member, fitting or appurtenance, use metal gauge or dimensions that fully develop the strength of the members with given gauges and dimensions used in structural combination.

Fabricate so members of the same nominal size are fully interchangeable. Fabricate and punch the members so no drilling, punching, or drifting to correct defects in manufacture will be required during field assembly. Any members with improperly punched holes will be rejected. Replace with a member with properly punched holes.

(3) **Backfill**. Provide 4"-0 stone embankment material for bin walls according to 00330.16.

Rock Gabions

(1) **Baskets and geotextile**. Provide rock gabion baskets according to section 02340. Use riprap geotextile according to section 02320.

Provide wire mesh material, which is free of any breaks in the wire, at weld points, or other deficiencies.

(2) **Rock gabion fill material**. Provide a well-graded 4- to 10-inch stone fill material meeting the requirements of 02330.10(b).

Cast-In-Place Concrete Retaining Walls

Provide materials for cast-in-place concrete retaining walls according to sections 00510, 00530, and 00540.

Construction

General. Perform structure excavation according to section 00510 to the limits and stages shown. All retaining walls, regardless of design, shall conform to the applicable top of the wall profile shown on the plans. Provide footings as shown or approved, whichever is lower in elevation.

MSE Retaining Walls

(1) **Foundation preparation**. Grade the foundation for the structure level for a width equal to the length of the bottom reinforcement, or as shown on the approved shop drawings. Compact the foundation to a minimum 95 percent of the relative maximum density with a smooth wheel vibratory roller, except where constructed on rock according to 00330.42. Remove any unsuitable foundation soils and replace with MSE granular backfill material as directed.

(2) **Leveling pad.** Construct a cast-in-place unreinforced concrete leveling pad at each panel foundation level of the type shown. Cure the pad for a minimum of 12 hours before placing the wall panels.

(3) **MSE wall erection**. Obtain instructions and recommendations, provide for a company field representative from the manufacturer of the wall option selected to be present at the start of MSE retaining wall construction, and be available as needed during the erection of the wall to assist the fabricator, contractor, or engineer. Follow instructions and recommendations of the representative if approved by the engineer.

Construct adjacent general embankment layers at the same time the MSE layers are constructed.

To erect the wall face panels, handle them with the lifting devices connected to the upper edge of the panel, and them place so the final position is vertical or battered as shown on the approved shop drawing. As granular backfill placement proceeds, place the panels in successive horizontal courses in the sequence shown. As granular backfill material is placed behind the panels, maintain the panels in position by means of temporary wedges or bracing according to the wall suppliers' recommendations.

(4) **MSE wall tolerances**. During construction of wall and placement of the panels, maintain the following tolerances:

(a) Vertical tolerances and tangent horizontal alignment tolerances along the wall line for facing panels shall not exceed 3/4 inch when measured with a 10-foot straightedge.

(b) Maximum allowable offset in any panel joint shall be 3/4 inch.

(c) Horizontal, vertical, and sloped joint openings between panels shall be uniform, no larger than 1-1/4 inch, and no smaller than 1/2 inch.

Check the plumbness and tolerances of each panel course before erection of the next panel course.

(5) **Reinforcement elements**. Before placing the reinforcement elements, place and compact the fill according to 00395.41(a) and (f). Place the reinforcement elements normal to the face of the wall unless otherwise shown or directed.

(6) **MSE granular backfill material**. Closely follow erection of each course of panels with placement of the MSE granular backfill material. Avoid any damage or misalignment of the facing panels,

reinforcement elements, or fasteners as the backfill is placed. Remove any wall materials damaged during backfill placement and replace them at the contractor's expense. Correct misaligned or distorted wall facing panels as directed, due to backfill placed outside the limits specified in 00395.41(d), at the contractor's expense.

Construct the backfill according to 00330.42, 00330.43, and the following requirements:

(a) Compact MSE granular backfill material to 95 percent of the relative maximum density.

(b) Achieve 95 percent of relative maximum density within 3 feet of the back face of the MSE retaining wall by using a lightweight mechanical tamper, roller, or a vibratory system.

(c) For applications where spread footings for bridges or other structural loads are founded in the MSE granular backfill material, compact the top 5 feet below the footing elevation to 100 percent of the relative maximum density.

At the end of each day, if rain is anticipated, slope the MSE granular backfill away from the MSE retaining wall face to direct surface runoff away from the wall. Do not allow surface runoff from adjacent areas to enter the MSE retaining wall construction site.

Metal Bin Walls

Construct galvanized metal bin type retaining walls at the locations, to the lines, and at the grades and dimensions shown or directed.

Obtain instructions and provide for a company field representative from the manufacturer of the bin wall materials to be present at the start of the metal bin wall construction and be available as needed during erection of the wall to assist the contractor and engineer. Follow instructions and recommendations of the representative if approved by the engineer.

Concurrently with the assembly of the bins, backfill with 4"-0 stone embankment material within and around the bins of the assembled wall to the limits shown on the plans. Keep the backfill around the outside approximately level with the inside fill. Place the backfill material in layers not more than 6 inches thick. Exercise care to completely fill the depressions of stringers and spacers and compact without displacing them from line and batter. Construct the backfill with mechanical tampers according to 00330.42 and 00330.43 subject to the above.

Rock Gabions

(1) **General**. Construct all rock gabions on suitable foundation materials to the lines and grades shown or as directed. Excavate or backfill according to section 00510. Subexcavate any

unsuitable foundation materials and backfill with suitable material as directed.

If required, furnish and place riprap geotextile according to section 00350 and the following: minimum overlap shall be 12 inches, and place riprap geotextile against the back of the gabion wall before placing backfill material.

Select and use the same style of mesh for the gabion panels (bases, ends, sides, diaphragms, and lids), the same method of joining the edges of a single gabion unit, and the same method of tying successive gabion units together throughout each structure.

If the constructed rock gabion wall is deficient in height more than 5 percent from the design height, add additional rock gabion baskets as directed to attain the design height at the contractor's expense.

(2) **Assembly**. Assemble each style of gabion by rotating the panels into position and joining the vertical edges with tie- or spiral-binding wire.

If welded wire panels are tied, pass the tie wire through each mesh opening along the vertical edges joint and secure with a half hitch-locked loop.

If twisted wire panels are tied, join the salvage vertical edges with tie wire at 4-inch nominal spacing, with alternating single- and double- locked loops.

Leave no openings along the edges or at the corners of the tied or spiral bound gabions of either mesh style greater than 4.75 inches (line dimension). Crimp the edges of spiral binding wire to secure the spiral in place.

(3) **Placement**. Set the empty gabions in place and connect each gabion to the adjacent gabion along the top and vertical edges with tie or spiral binding wire. Connect each layer of gabions to the underlying layer along the front, back, and sides in the same manner specified for the assembly of baskets. If allowed by the special provisions, fasteners meeting the requirement of 02340.00 may be used to tie assembled gabions together. Common wall construction will not be allowed.

Before filling each gabion with rock, remove all kinks and folds in the wire fabric and properly align all baskets. Remove all temporary clips and fasteners. The assembled gabion baskets may be placed in tension before filling.

(4) **Filling**. Carefully place the rock by hand or machine to ensure proper alignment, avoid bulges, and assure that there is a minimum of voids. All exposed rock surfaces shall have a smooth, neat appearance and no sharp edges projecting through the wire mesh.

Place the rock in layers to allow the placement of internal connecting wires in each outside cell of the structure or when ordered at the following intervals:

- None required for 12-inch high baskets.
- At the 9-inch level for 18-inch high baskets.
- At 1/3 points for 3-foot-high baskets.

Completely fill the basket so the lid will bear on the rock when it is closed. Secure the lid to the sides, ends, and diaphragms in the same manner specified for joining the vertical edges.

(5) **Repairs**. During construction, repair and secure any breakage of the wire mesh that results in mesh or joint openings larger than 4.75 inches (line dimension). Make repairs using a 13.5 gauge-galvanized tie wire as directed.

Repair any damage to PVC wire coating in a manner that provides the same degree of corrosion resistance as the undamaged wire according to the manufacturer's recommended repair procedures and as approved.

Cast-In-Place Concrete Retaining Walls

Construct cast-in-place concrete retaining walls according to sections 00510, 00530, and 00540.

Measurement for Excavation

Unless otherwise shown, no measurement for excavation will be made.

MSE Retaining Walls

The pay quantity for MSE retaining walls will be measured on the square foot basis (to the nearest square foot) of the actual surface area of the walls plus footings.

The pay quantity for coping will be measured on the linear foot basis (to the nearest linear foot) from end to end of coping.

Metal Bin Walls

Metal bin walls will be accepted for payment on a lump sum basis. No measurement of quantities will be made.

No separate measurement will be made for specified backfill used in constructing the walls because this work is considered incidental to the wall construction.

Rock Gabions

Rock gabions will be measured on the cubic yard basis by using the neat line dimensions of the baskets, as indicated on the plans.

No separate measurement will be made for riprap geotextile for gabions.

Cast-In-Place Concrete Retaining Walls

Cast-in-place concrete retaining walls will be measured according to sections 00530 and 00540.

Payment

General. The accepted quantities will be paid for at the contract price per unit of measurement for each of the following pay items:

Pay item	Unit of measurement
(a) MSE retaining walls	Square foot
(b) _____ coping	Linear foot
(c) Metal bin wall	Lump sum
(d) Rock gabions	Cubic yard

If retaining walls in items (a) or (c) are constructed at more than one location, each retaining wall will be paid for separately and will be identified by adding the description "Station ____," with the retaining wall location station inserted in the blank.

Item (a) includes all MSE retaining wall, options in the section on mechanically stabilized earth (MSE).

In item (b) the type of coping will be inserted in the blank, with a separate item for each type. Reinforcing steel and concrete used in this work is considered incidental with no separate or additional payment being made.

Item (c) includes furnishing and placing stone embankment material both inside and around the outer sides of the bin wall.

Item (d) includes furnishing and placing the gabion baskets, stone fill material, and anchorages, if required. Riprap geotextile used for gabions will be considered incidental with no separate or additional payment being made.

Payment will be payment in full for all work specified including all materials, equipment, tools, labor, and incidentals necessary to complete the work.

If shown, excavation for retaining walls will be paid for according to section 00510, otherwise excavation will be considered incidental with no separate or additional payment being made.

Geotextile retaining walls will be paid for according to 00350.90(d).

Cast-in-place concrete retaining walls will be paid for according to sections 00530 and 00540.

Shotcrete Slope Stabilization

Description and scope. This work consists of constructing pneumatically applied shotcrete stabilization blankets onto slope surfaces at locations shown or as directed.

Definitions, Standards, and Requirements

Requirements. Design the shotcrete mix and be responsible for the quality of shotcrete used in the work.

Shotcrete. Either dry-mix or wet-mix material composed of portland cement, fine and coarse aggregate, water, and reinforced with either welded wire fabric or steel fibers.

Standards. Construct shotcrete according to these specifications and applicable sections of the latest edition of the American Concrete Institute's "Guide to Shotcrete" (ACI 506).

Materials

General. Materials shall meet the following requirements:

Bar reinforcement	02510.10
Cement (type I or II)	02010.10
Chemical admixtures	02040
Coarse aggregate	02690.20
Curing materials	02050.10
Fine aggregate	02690.10
Grout	02080.20
PVC pipe	02410.70
Water	02020
Welded wire fabric	02510.40

Prepackaged Product

Premixed and prepackaged concrete products, with or without steel fibers, manufactured as a shotcrete product may be used for on-site mixed shotcrete if the materials meet this specification and if they are approved.

Washington Department of Transportation Specifications—Mechanically Stabilized Earth Wall (Preapproved)

General Requirements

Description

(1) This work consists of constructing the preapproved, mechanically stabilized earth walls in accordance with the plans, the standard specifications, and these special provisions, and in conformity with the lines, grades, design, and dimensions shown in the plans or as established by the engineer.

(2) The mechanically stabilized earth walls for this project shall be constructed of one of the following four wall types: reinforced earth (a registered trademark of the Reinforced Earth Company), retained earth (a registered trademark of the VSL Corporation), Hilfiker retaining walls, or Keystone. The allowable types for each wall are listed in the special provision Alternative Bids.

Quality Assurance

(1) The completed wall shall meet the following tolerances:

- Vertical tolerances (plumbness) and horizontal tolerances (alignment) shall not exceed 3/4 inch when they are measured along a 10-foot straightedge.
- The maximum allowable offset in any panel joint shall be 3/4 inch.
- The overall vertical tolerances of the wall (plumbness from top to bottom) shall not exceed 1/2 inch per 10 feet of wall height.

(2) The mechanically stabilized earth wall manufacturer shall provide a qualified and experienced representative at the job site, at the start of wall construction, and as needed to resolve wall construction problems as directed by the engineer. Recommendations made by the representative and approved by the engineer shall be followed by the contractor.

Submittals

(1) The contractor, or the supplier as his agent, shall furnish the engineer a certificate of compliance stating that the mechanically stabilized earth wall materials comply with the applicable sections of these specifications.

(2) A copy of all test results performed by the contractor or his supplier, which are necessary to assure compliance with the specifications, shall also be furnished to the engineer.

(3) Before fabrication, the contractor shall submit a field construction manual to the engineer for approval for the mechanically stabilized earth walls. This submittal shall be made at least 90 days prior to the beginning of wall construction and shall include a field construction manual prepared by the wall manufacturer.

(4) Design calculations and shop drawings: The contractor, or the supplier as his agent, shall submit detailed design calculations and shop drawings to the engineer for approval at least 90 days prior to the start of wall construction. The submittal shall include detailed design calculations and all details, dimensions, quantities and cross-sections necessary to construct the wall. The calculations shall include a detailed explanation of any symbols and computer programs used in the wall design.

The design calculations shall be based on the following specifications:

(a) The factor of safety for overturning and sliding are 2.0 and 1.5, respectively.

(b) The angle of internal friction of the soil and the maximum allowable toe pressure is as follows:

Wall	Allowable Toe Pressure	Soil Angle Of Internal Friction
3,5,6,10,11,14,16	4.0 ksf	36 degrees
4	5.0 ksf	36 degrees
8	14.0 ksf	36 degrees
14,16	6.0 ksf	36 degrees
17	4.0 ksf	36 degrees

(c) The slope of the backfill shown in the plans.

(d) If the wall is adjacent to a highway, a 2-foot surcharge shall be used in the design.

(e) If the plans detail a New Jersey-type traffic barrier on top of the wall, the barrier and wall shall be capable of resisting a 10,000-pound horizontal load applied at the top of the barrier.

A minimum of six sets of shop plans shall be fully detailed and submitted on 24- by 36-inch sheets and shall include, but not be limited to, the following items:

- A plan and elevation sheet or sheets for each wall, containing the following information:

  - An elevation view of the wall that shall indicate the elevation at the top of the wall, at all horizontal and vertical break points, and at least every 50 feet along the wall, elevations at the top of leveling pads and footings, the distance along the face of the wall to all steps in the footings and leveling pads, the designation as to the type of panel or module, the length, size and number of mesh or strips, the distance along the face of the wall to where changes in length of the mesh or strips occur, the location of the original and final ground line.

  - A plan view of the wall that shall indicate the offset from the construction center line to the face of the wall at all changes in horizontal alignment, the limit of the widest module, mesh or strip, and the center line of any drainage structure or drainage pipe which is behind passes under or through the wall.

  - Any general notes required for design and construction of the wall.

  - All horizontal and vertical curve data affecting wall construction.

  - A listing of the summary of quantities provided on the elevation sheet of each wall for all items including incidental items.

  - A cross-section showing the limits of construction in-fill sections, limits, and extent of select granular backfill material placed above original ground.

  - Limits and extent of reinforced soil volume.

- All details including reinforcing bar-bending details. Bar-bending details shall be in accordance with ACI standards.

- All details for foundations, leveling pads, steps in the footings or leveling pads, and allowable and actual maximum bearing pressures.

- All modules and facing elements shall be detailed. The details shall show all dimensions necessary to construct the element, all reinforcing steel in the element, and the location of reinforcement element attachment devices embedded in the facing.

- All details for construction of the wall around drainage facilities, overhead sign footings, and abutment piles shall be clearly shown.

- All details for connections to traffic barriers, coping, parapets, noise walls, and attached lighting shall be shown.

- All details for the New Jersey-type traffic barrier attached to the top of the wall.

- The plans shall be prepared and signed by a professional engineer, licensed in the State of Washington.

Materials

(1) **General**. The contractor shall make his own arrangements to purchase the concrete facing panels, reinforcing strips or mesh, attachment devices, joint filler, and all necessary incidentals from one of the following four manufacturers:

The Reinforced Earth Company
22619 S.E. 64th Place, Suite 240
Issaquah, WA 98227
206-391-0111

VSL Corporation
1077 Dell Avenue
Campbell, CA 95008
408-866-5000

Hilfiker Retaining Walls
P.O. Box 2012
Eureka, CA 95502-2012
707-443-5093
707-443-2891 (FAX)

Keystone Pacific Northwest
10445 S.W. Canyon Rd., Suite 111
Beaverton, OR 97005
1-800-733-7470

Concrete facing panels for reinforced earth, VSL, and Hilfiker walls

(1) Materials

(a) Concrete for face panels shall be in accordance with section 6-02.3(27) and this special provision.

(2) Testing and inspection

(a) Acceptability of the panels will be determined by compressive strength tests and visual inspection.

(b) The panels shall be considered acceptable regardless of curing age when compressive test results indicate that the compressive strength will conform to the 28-day requirements and when the visual inspection is satisfactorily completed.

(c) Panels will be considered acceptable for placement in the wall when 7-day initial strengths exceed 85 percent of the 28-day requirements.

(3) Casting

(a) Tie attachment devices shall be set in place to the dimensions and tolerances shown in the plans prior to casting.

(4) Curing

(a) The panels shall be cured for a sufficient length of time so that the concrete will develop the specific compressive strength.

(5) Removal of forms

(a) The forms shall remain in place for a sufficient length of time so they can be removed without damage to the panels.

(6) Finish

(a) The concrete surface for the front face shall have the finish shown in the plans, and for the rear face, an unformed finish.

(b) The rear face of the panel shall be roughly screeded to eliminate open pockets of aggregate and surface distortions in excess of 1/4 inch.

(7) Tolerances

(a) All panels shall be manufactured within the following tolerances:

- All dimensions ± 3/16 inch.
- Squareness, as determined by the difference between the two diagonals, shall not exceed 1/2 inch.
- Surface defects on smooth-formed surfaces measured on a length of 5 feet shall not exceed 1/8 inch. Surface defects

on textured-finished surfaces measured on a length of 5 feet shall not exceed 5/16 inch.

(8) Marking

(a) The date of manufacture, the production lot number and the piece-mark, shall be clearly marked on the rear face of each panel.

(9) Handling, storage and shipping

(a) All panels shall be handled, stored, and shipped properly to eliminate the danger of chipping, cracks, fractures, and excessive bending stresses.

(b) Panels in storage shall be supported on firm blocking located immediately adjacent to tie strips to avoid bending the tie strips.

Reinforcing strips

(1) Reinforcing strips shall be shop-fabricated from hot-rolled steel according to the requirements of AASHTO M 223 Gr. 65 or to an approved equivalent.

(2) Reinforcing strips shall be hot dip-galvanized according to AASHTO M 111.

Reinforcing mesh

(1) The reinforcing mesh shall be shop-fabricated from cold-drawn steel wire according to the minimum requirements of AASHTO M 32 and shall be welded into finished mesh fabric according to AASHTO M 55.

(2) Reinforcing mesh shall be hot dip-galvanized according to M 111. Damage to the galvanizing from forming the buttonheads shall be repaired with state formula A-9-73 galvanizing repair paint per section 9-08.2.

Tie strips

(1) Tie strips shall be shop-fabricated from hot-rolled steel conforming to the requirements of ASTM A570 grade 50 or an approved equivalent.

(2) Tie strips shall be hot dip-galvanized according to AASHTO M 111.

Fasteners—reinforced earth wall

(1) The 1/2-inch-diameter bolts and nuts shall be high strength, hexagonal cap screws, according to AASHTO M 164, galvanized per AASHTO M 232.

Fasteners—retained earth wall

(1) The 1-inch-diameter coil embed shall be fabricated from cold-drawn steel wire conforming to AISI C1035 and shall be galvanized in accordance with ASTM B 633.

(2) The 1-inch-diameter coil bolt shall have 2 inches of thread. It shall be cast of 80-55-06 ductile iron conforming to ASTM A 536, galvanized per ASTM B 633.

(3) Fasteners—Hilfiker, HQ.

(4) Fasteners—Keystone, HQ.

Joint materials—reinforced earth wall

(1) Rubber bearing pads shall be a type and grade as recommended by the Reinforced Earth Company.

(2) Vertical joint filler between panels, when specified in the plans, shall be flexible, open cell polyester foam strips, grade UU-34, 2- by 2-inch, recommended by the Reinforced Earth Company.

(3) Filter fabric joint cover for both horizontal and vertical joints, when specified in the plans, shall be a previous woven polypropylene filter fabric recommended by the Reinforced Earth Company. Adhesive used to attach the fabric material to the rear of the panel shall be as recommended by the Reinforced Earth Company.

Joint materials—retained earth wall

(1) The material to be attached to the rear side of the facing panel covering the inclined and horizontal joints between panels shall be monofilament filter fabric as recommended by the VSL Corporation. Adhesive used to attach the fabric to the panel shall be as recommended by the VSL Corporation.

Concrete leveling pad

(1) Concrete for the leveling pad shall be class 4000 concrete according to section 6-02.

Backfill material

(1) All backfill material used in the mechanically stabilized earth compaction zone shall be free-draining, free from organic or deleterious material, and shall conform to the gradations for gravel borrow as specified in section 9-03.14.

(2) The material shall be substantially free of shale or other particles which are soft and poor in durability. The material shall have magnesium sulfate soundness loss of less than 30 percent after four cycles.

(3) The material shall meet the following corrosive requirements:

Resistivity: greater than 3,000 ohm/cm
PH: 5 to 10
Chlorides: less than 200 mg/kg
Sulfates: less than 1,000 mg/kg
Sulphides: less than 300 mg/kg

Keystone geogrid wall materials

Approval and acceptance of tensar geogrid:

Samples will be randomly taken by the engineer at the job site to confirm that the geogrid meets the property values specified.

Approval will be based on testing of samples from each lot. For the purposes of this specification, a "lot" shall be defined as all geogrid rolls within the consignment (i.e., all rolls sent to the project site) which were produced by the same manufacturer, have the same product name, and have the same manufacturer's certificate of compliance arrive at the Headquarters Materials Laboratory in Tumwater. A maximum of 28 calendar days will be required for this testing. If the results of the testing show that a geogrid lot, as defined, does not meet the properties required in table 1 (geogrid property requirements), the roll or rolls which were sampled will be rejected. Two additional rolls from the lot previously tested will then be randomly selected by the engineer for sampling and retesting. If the retesting shows that either or (both rolls) does not meet the required properties, the entire lot will be rejected. All geogrid having defects, deterioration, or damage, as determined by the engineer, will also be rejected. All rejected geogrid shall be replaced at no cost to the state.

Tensar geogrid material requirements:

The wall shall be constructed using Tensar geogrid product UX1400 and UX1500 as identified in the plans. The geogrid shall conform to the properties as indicated in table 1.

Table 1.—Geogrid property requirements.

Property	Test method[1]	Tensar UX1400	Tensar UX1500
Peak Tensile Strength (lb/ft)	WSDOT test Method No. 925	3,700 lb/ft (machine direction)	5,800 lb/ft (machine direction)
Junction Integrity (rib strength percent)	GRI Test method GG2-87	80 percent	80 percent

[1] All geogrid facing units shall be constructed to the dimension and shape as detailed in the plans and shall meet all the materials, manufacturing, and physical requirements of ASTM C 90, except for the following:

[2] This test method involves: six specimens from each laboratory sample that shall be tested. Each specimen shall be 8 inches wide with a gage length between grips of 10 inches. Each specimen shall be tested at a constant rate of extension of 10 percent per minute, at a temperature of 20° C, ± 2° C.

Concrete facing units:

The concrete facing units shall be constructed to the dimension and shape as detailed in the plans and shall meet all the materials, manufacturing, and physical requirements of ASTM C 90, except for the following:

Compressive strength minimum	3,000 psi at 28 days
Water absorption maximum	6 percent
Unit weight minimum	95 pounds per square foot of wall face

Facing units shall be interlocked with noncorrosive fiberglass pins as shown in the plans. The pins shall consist of polyester resin rods with fiberglass reinforcement. Pins shall have a minimum flexural strength of 128,000 psi according to ASTM D 790. The pins shall have a minimum diameter of 0.5 inch and a minimum length of 9.25 inches.

Material for the wall face leveling pad shall consist of either gravel backfill for foundations, class A, or concrete-placed as shown in the plans.

Construction Requirements

Wall excavation

Excavation shall be according to the requirements of section 2-09 and in close conformity to the limits and construction stages shown in the plans.

Foundation preparation

(1) The foundation for the structure shall be graded level for a width equal to or exceeding the length of reinforcing as shown in the approved shop plans.

(2) Prior to wall construction, the foundation, if not in rock, shall be compacted as directed by the engineer.

(3) Any foundation soils found to be unsuitable shall be removed and replaced, as provided for under section 2-09.3(1)C.

(4) At each panel foundation level, an unreinforced concrete leveling pad shall be provided as shown in the plans. The leveling pad shall be cured a minimum of 12 hours before placement of wall panels.

Wall erection

(1) The panels shall be placed vertically with the aid of a light crane. For erection, panels are handled with a lifting device set into the upper edge of the panels.

(2) Panels should be placed in successive horizontal lifts in the sequence shown in the plans as backfill placement proceeds.

(3) External bracing is required for the initial lift.

(4) As backfill material is placed behind the panels, the panels shall be maintained in a vertical position with temporary wooden wedges placed in the joint at the junction of the two adjacent panels on the external side of the wall.

(5) Reinforcing shall be placed normal to the face of the wall, unless otherwise shown in the plans or directed by the engineer. Prior to reinforcing placement, backfill shall be compacted.

Backfill placement

(1) Backfill placement shall closely follow the erection of each course of panels. Backfill shall be placed in such a manner as to avoid any damage or disturbance to the wall materials or misalignment of the panels.

(2) Any wall materials which become damaged or disturbed during backfill placement shall be either removed and replaced at the contractor's expense or repaired and corrected as directed by the engineer.

(3) Any misalignment or distortion of the panels due to placement of backfill outside the limits of this specification shall be corrected as directed by the engineer.

(4) The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer of material:

   (a) The moisture content of all backfill material shall meet the requirements of section 2-03.3(14)C, method C.

   (b) Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniformly acceptable throughout the entire lift.

   (c) The optimum moisture content shall be determined in accordance with section 2-03.3(14)D.

(5) Backfill shall be compacted to 95 percent of the maximum density as specified under compacting earth embankments, method C, in section 2-03.3(14)C, except as modified herein:

   (a) The maximum lift thickness after compaction shall not exceed 10 inches.

   (b) The contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

   (c) Compaction within 3'-0" of the back of the wall facing panels shall be achieved using light mechanical tampers approved by the engineer and shall be done in a manner to cause no damage or distortion to the wall facing elements.

(6) At the end of each day's operation, the contractor shall shape the last level of backfill to permit runoff of rainwater away from the wall face. In addition, the contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Specific construction requirements for Keystone wall

Shipment and storage of geogrid

During the periods of shipment and storage, the geogrid shall be kept dry at all times and shall be stored off the ground. Under no circumstances during shipment or storage shall the materials be exposed to sunlight or to other light form which contains ultraviolet rays for more than 5 calendar days.

The wall face leveling pad shall be compacted using method C if gravel backfill for foundations class A is used. The surface of the leveling pad shall be level and hard. The leveling pad shall be prepared in a manner that will ensure complete contact between each wall facing unit in the lowest course of facing units and the leveling pad.

The surface of the wall backfill on which each geogrid layer is placed shall be graded to a smooth uniform condition free from ruts, potholes, and protruding objects. The geogrid shall be spread immediately ahead of the covering operation.

Wall construction shall begin at the lowest portion of the excavation and each layer shall be placed horizontally as shown in the plans. Each layer shall be compacted entirely before the next layer is started.

Splices for the geogrid shall consist of butting adjacent reinforcement strips together using hog rings or other methods approved by the engineer to prevent the reinforcement strip from separating during installation and backfilling. Geogrid splices parallel to the wall face will not be allowed. The geogrid shall be stretched out in the direction perpendicular to the wall face to ensure that no slack exists in the geogrid prior to backfilling. Backfill shall be spread from the wall face outward to ensure that the geogrid layer remains taut. Geogrid splices shall be offset from one another so that the splices for these various wall elements do not line up with one another. If horizontal alignment curves in the wall face require the geogrid layers at the back of the geogrid reinforcement layer to be overlapped more than 2 feet, the overlapped geogrid shall be splayed vertically so that a minimum of 3 inches of soil will be placed between the two geogrid reinforcement strips.

Under no circumstances shall the geogrid be dragged through mud or over sharp objects which could damage the geogrid. The fill material shall be placed on the geogrid in such a manner that a minimum of 6 inches of material will be between the vehicle or equipment tires or tracks and the geogrid at all times. Turning of vehicles of the first lift above the geogrid will not be permitted. End-dumping fill directly on the geogrid will not be permitted.

Should the geogrid be damaged or the overlaps disturbed, the backfill around the damaged or displaced area shall be removed and the damaged strip of geogrid replaced by the contractor at no cost to the State.

Facing units may be saw-cut as required using standard masonry tools. Sawn, half-width blocks shall not be used in the base course.

Two fiberglass connecting pins shall be installed in each concrete facing unit in a manner that will ensure the pins will protrude a minimum of 1 inch into each facing unit.

All voids within and around the concrete facing units shall be filled with gravel borrow fill tamped in place. Excess fill material shall be swept from the surface of the concrete facing blocks.

The geogrid reinforcement shall be hooked over each fiberglass pin and pulled taut prior to backfill placement over the grid. The facing units placed on top of the grid layer shall be pulled forward away from the wall backfill zone against the fiberglass pins.

Wall facing units shall be turned into the embankment with a convex return end at the ends of each course where the change in wall elevation is greater than 8 inches. A minimum of three units shall be installed below grade at these ends. End returns are not required for elevation changes of 8 inches or less.

The cap course of the wall shall be bonded to the adjacent lower course with an approved cement base, waterproof anchoring cement.

Measurement

Mechanically Stabilized Earth Wall

(1) The quantity of the following items to be paid for on this project shall be the quantity shown in the bid proposal, unless changes are made according to section 1-04.4, which affected this quantity. The quantity shown is the average of the wall types that apply (see special provision Alternative Bids). The quantity shown in the bid proposal will be adjusted by the amount of the change and will be paid for as specified in section 1-04.4

Mechanically stabilized earth wall
Mechanically stabilized earth wall no. 4
Mechanically stabilized earth wall no. 15

The quantities in the bid proposal are listed only for the convenience of the contractor to determine the volume of work involved, and are not guaranteed to be accurate. The prospective bidders shall verify these quantities before submitting a bid. No adjustments other than for approved changes will be made in the quantity even though the actual quantities required may deviate from those listed.

(2) The backfill material will be measured as specified in section 2-03.3.

Geotextile Retaining Wall

Description

The contractor shall construct geotextile retaining walls according to the details shown in the plans, these special provisions, or as directed by the engineer.

Submittals

The contractor shall submit to the engineer a minimum of 14 calendar days prior to beginning construction of each wall, detailed plans for each wall, and other information. As a minimum, the submittals shall include the following information:

(1) Detailed wall plans showing the actual lengths proposed for the geotextile reinforcing layers, and the locations of each geotextile product proposed for use in each of the geotextile reinforcing layers.

(2) The contractor's proposed wall construction method, including proposed forming systems, types of equipment to be used, and proposed erection sequence.

(3) Geotextile certificate of compliance and samples for the purpose of source approval as required elsewhere in these special provisions.

(4) Details of geotextile wall corner construction as required elsewhere in these special provisions.

(5) If the wall is permanent, the shotcrete mix design with compressive strength test results, and the method proposed for shotcrete wall face finishing and curing.

Approval of the contractor's proposed wall construction details and methods shall not relieve the contractor of his or her responsibility to construct the walls in accordance with the requirements of these specifications.

Materials

Geotextiles and thread for sewing

The material shall be woven or nonwoven geotextile consisting only of long chain polymeric filaments or yarns formed into a stable network so that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 95 percent by weight of the long chain polymers shall be polyolephins, polyesters, or polyamides. The material shall be free of defects and tears. The geotextile shall conform to the properties as indicated in tables 1 and 2. The geotextile shall be free from any treatment or coating which may adversely alter its physical properties after installation.

Thread used shall be high strength polypropylene, polyester, or Kevlar thread. Nylon threads will not be allowed. The thread used must also be resistant to ultraviolet radiation if the sewn seam is exposed at the wall face.

Table 1.—Minimum properties required for geotextile used in geotextile retaining walls.

Geotextile property	Test method[2]	Geotextile[1] property requirements
Water permeability	WSDOT test method 924: water permeability of minimum geotextiles by permittivity	0.005 cm/sec minimum
AOS	WSDOT test method 922: apparent maximum opening size of geotextiles	.84 mm maximum (#20 sieve)
Grab tensile strength, minimum in machine and x-machine direction	WSDOT test method 916: breaking load and elongation of geotextiles (grab method)	200 lb minimum
Burst strength	WSDOT test method 920: diaphragm bursting strength of geotextiles	300 psi minimum
Puncture resistance	WSDOT test method 921: puncture strength of geotextiles	80 lb minimum
Tear strength, minimum in machine and x-machine direction	WSDOT test method 919: trapezoid testing strength of geotextiles	60 lb minimum
Ultraviolet (UV) radiation stability (strength retained percentage)	ASTM D 4355-84, after 500 hours in weatherometer	70% minimum
Seam breaking strength[3]	WSDOT test method 918 and WSDOT test method 916 (grab test)	160 lb minimum

[1]All geotextile properties are minimum average roll values (i.e., the average test results for any sampled roll in a lot shall meet or exceed the values shown in the table).

[2]WSDOT test methods 916, 917, 919, and 924 conform to ASTM geotextile test procedures, except for geotextile sampling and specimen conditioning. Copies of all WSDOT geotextile test methods are available at the WSDOT Headquarters Materials Laboratory in Tumwater.

[3]Applies only to seams perpendicular to the wall face.

Table 2.—Wide strip tensile strength required for the geotextile used in geotextile retaining walls.

			Minimum tensile strength[1]	
			Geotextile polymer type	
Wall location	Vertical reinforcement layer spacing	Distance from top of wall	Polyester	Polypropylene/ polyethylene
$$1$$	$$	$$	$$	$$

Note: These geotextile strengths are minimum average roll values (i.e., the average test results for any sampled roll in a lot shall meet or exceed the minimum values shown in the table).

[1] WSDOT test method 917: method for wide width tensile strength. This strength requirement only applies in the geotextile direction perpendicular to the wall face. WSDOT test method 917 conforms to ASTM D 4595, except for geotextile sampling and specimen conditioning. Copies of all WSDOT geotextile test methods are available at the WSDOT Headquarters Materials Laboratory in Tumwater.

Shotcrete wall facing. Shotcrete shall be the application of one or more layers of pneumatically placed concrete to soil, geosynthetic, concrete, or steel surfaces.

Plain shotcrete is defined as a Portland cement concrete mix containing admixtures to provide quick set, high early strength, and satisfactory adhesion, which is conveyed through a hose and pneumatically projected at a high velocity onto a surface.

The wet mix process involves thoroughly mixing all ingredients, except accelerating admixtures but including the mixing water, introducing the mixture into the delivery equipment and delivering it by positive displacement or compressed air to the nozzle. The mixed shotcrete is then air-jetted from the nozzle at a high velocity onto the surface in the same manner as for the dry mix process. The accelerator for the wet mix is added to the shotcrete mixture in such a way that the quantity can be properly regulated and the material uniformly dispersed throughout the shotcrete when it is placed.

Portland cement shall be type II, as specified in section 9-01.2(1). Air entrainment shall be 6 percent (plus or minus 1-1/2 percent) to comply with the requirements of section 9-23.6.

Aggregate for shotcrete shall meet the following gradation requirements:

Sieve size	Percent passing by weight
1/2 inch	100
3/8 inch	90–100
No. 4	70–85
No. 8	50–70
No. 16	35–55
No. 30	20–35
No. 50	8–20
No. 100	2–10
No. 200	0–2.5

Water for mixing and curing shall be clean and free of substances which may be harmful to concrete or steel. Water shall also be free of elements which would cause staining.

Reinforcement shall be as shown in the plans and shall comply with the requirements of section 9-07.

Proportioning concrete. Shotcrete shall be proportioned to produce a 4,000 psi compressive strength at 28 days. The shotcrete mix design and method of placement proposed for use at the job site shall be submitted to the engineer by the contractor at least 14 calendar days prior to beginning shotcrete placement. The contractor shall also include evidence within this submittal that the proposed shotcrete mix design and method of placement will produce the required compressive strength at 28 days. The contractor must receive notification from the engineer that the proposed mix design and method of placement is acceptable before shotcrete placement can begin.

No admixture shall be used without the permission of the engineer. If admixtures are used to entrain air, to reduce a water-cement ratio, to retard or accelerate setting time, or to accelerate the development of strength, the admixtures shall be used at a rate specified by the manufacturer and approved by the engineer.

Shotcrete testing. The contractor shall make shotcrete test panels for the evaluation of shotcrete quality, strength, and aesthetics. The preproduction test panels, and the nozzlemen prequalification, for approval and the production test panels, shall be prepared by the contractor. Any cores obtained for the purpose of shotcrete strength testing shall have the following minimum dimensions: (1) the core diameter shall be at least 3 times the maximum aggregate size, but not less than 2 inches, and (2) the core height shall be 1.5 times the core diameter, but not less than 3 inches.

The core will be obtained and tested according to AASHTO T 24. Cores removed from the panel shall be immediately wrapped in wet burlap and sealed in a plastic bag. Cores shall be clearly marked to identify from where the cores were taken and whether they are for preproduction or for production testing. If the cores are for production testing, the section of the wall represented by the cores shall be clearly marked on the cores. Cores shall be delivered to the engineer within 2 hours of coring. The remainder of the panel(s) shall become the property of the contractor.

(1) **Preproduction testing.** The contractor shall prepare at least one 36- by 36-inch panel for each mix design to evaluate and test shotcrete quality and strength. The contractor shall make one additional 48- by 48-inch qualification panel for evaluation and approval of the contractor's proposed method for shotcrete, installation, finishing, and curing. Both the 36-inch and the 48-inch panels shall be constructed using the same methods and initial curing used to construct the shotcrete wall, except that the 36-inch panel shall not include wire reinforcement. The 36-inch panel shall be constructed to the minimum thickness necessary to obtain the core samples of the required dimensions. The 48-inch panel shall be constructed to the same thickness as proposed for the wall face. Production shotcrete work shall not begin until satisfactory test results are obtained and the panels are approved by the engineer.

(2) **Production testing.** The contractor shall make at least one 36- by 36-inch panel for each section of wall shot, or as many as directed by the engineer. A section is defined as one day's placement. The production panels shall be constructed using the same methods and initial curing used to construct the shotcrete wall, but without wire reinforcement. The panels shall be constructed to the minimum thickness necessary to obtain core samples of the required dimensions. If the production shotcrete is found to be unsuitable based on the results of the test panels, the section(s) of the wall represented by the test panel(s) shall be repaired or replaced to the satisfaction of the engineer at no cost to the contracting agency.

Shotcrete coloration for facing alternate C

If facing alternate C is required, the contractor shall provide shotcrete coloration for finishing the sculptured shotcrete to match the color of the natural surroundings. Approval of the final appearance of the coloration will be based on the preproduction test panel. The approval of the long-term properties of the coloration material shall be based on a manufacturer's certification, which verifies the following to be true about the product:

(1) Resistance to alkalies in accordance with ASTM D 543.

(2) Demonstrates no change in coloration after 1,000 hours of testing, according to ASTM D 822.

(3) Does not oxidize when tested according to ASTM D 822.

(4) Demonstrates resistance to gasoline and mineral spirits when tested according to ASTM D 543.

The certification shall also provide the product name, proposed mix design and application method, and evidence of at least one project where the product, using the proposed mix and application method, was applied and provided at least 5 years or more of acceptable durability color permanency.

Wall backfill material

All backfill material used in the reinforced soil zone of the geotextile wall shall be free-draining, free of organic or otherwise deleterious material, and shall conform to the gradations for the backfill material specified in the plans. The material shall be substantially free of shale or other soft particles poor in durability. The material shall have magnesium sulfate soundness loss of less than 30 percent after four cycles.

The backfill material shall meet the following chemical requirements for permanent walls:

Property or chemical	**Test method**	**Allowable quantity**
Resistivity	AASHTO T288-911	Greater than 3,000 ohm/cm
pH	AASHTO T289-911	Polypropylene/polyethylene: 5–10 Polyester: 5–8
Chlorides	AASHTO T291-911	Less than 100 mg/kg
sulfates	AASHTO T290-911	Less than 200 mg/kg

For temporary walls, the only backfill chemical property requirements which apply are the following: soil pH shall be between 3 and 11, and soil resistivity shall be greater than 1,000 ohm/cm.

Geotextile Approval and Acceptance

Source approval. The contractor shall submit to the engineer the following information regarding each geotextile proposed for use: the manufacturer's name and current address, the full product name, and geotextile polymer type(s).

If the manufacturer of the proposed geotextile(s) has not previously submitted a geotextile for initial source approval for a geotextile wall and obtained approval, a sample of each proposed geotextile shall be submitted for approval by the Headquarters Materials Laboratory in

Tumwater. After the sample and required information for each geotextile type have arrived at the Materials Laboratory, a maximum of 14 calendar days will be required for this testing. Source approval will be based on conformance to the applicable values from tables 1 and 2. Each sample shall have minimum dimensions of 1.5 yards by the full roll width of the geotextile. A minimum of 6 square yards of geotextile shall be submitted to the engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll.

The geotextile samples shall be cut from the geotextile roll with scissors, a sharp knife, or with another instrument that produces a smooth geotextile edge and would not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile nor the inner wrap of the core.

Acceptance of samples. Samples will be randomly taken by the engineer at the job site to confirm that the geotextile meets the property values specified.

Approval will be based on the testing of samples from each lot. For the purposes of this specification a "lot" shall be defined as all geotextile rolls within the consignment (i.e., all rolls sent to the project site) which were produced by the same manufacturer and have the same product name. After the samples and manufacturer's certificate of compliance have arrived at the Headquarters Materials Laboratory in Tumwater, a maximum of 14 calendar days will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required in tables 1 and 2, the sampled roll or rolls will be rejected. Two additional rolls from the lot previously tested will then be selected at random by the engineer for sampling and retesting. If the retesting shows that either or both rolls do not meet the required properties, the entire lot will be rejected. All geotextile with defects, deterioration, or damage, as determined by the engineer, will also be rejected. All rejected geotextile shall be replaced at no cost to the State.

Acceptance will be based on the manufacturer's certificate of compliance without sampling if the geotextile samples previously tested for the purpose of source approval came from the same geotextile lot as defined which is proposed for use at the project site, provided that the number of samples submitted and tested meet the requirements of WSDOT test method 914 "Practice for Sampling of Geotextiles for Testing."

The contractor shall provide a manufacturer's certificate of compliance to the engineer, whether or not samples are submitted for testing. The certificate includes the following information about each geotextile roll to be used: the manufacturer's name and current address, the full product name, geotextile roll number, geotextile polymer type, and certified test results.

Approval of seams. If the geotextile seams are to be sewn in the field, the contractor shall provide a section of sewn seam that can be sampled by the engineer before the geotextile is installed. The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seam sewn for sampling must be at least 2 yards in length. If the seams are sewn in the factory, the engineer will obtain samples of the factory seam at random from any of the rolls to be used. The seam assembly description shall be submitted by the contractor to the engineer and will be included with the seam sample obtained for testing. This description shall include the seam type, seam allowance, stitch type, sewing thread tex ticket number(s) and type(s), stitch density, and stitch gage.

Construction Requirements

Shipment and storage of geotextile. During periods of shipment and storage, the geotextile shall be kept dry at all times and shall be stored off the ground. Under no circumstances, either during shipment or storage, shall the materials be exposed to sunlight or other forms of light having ultraviolet rays, for more than 5 calendar days. Each roll shall be labeled or tagged to provide product identification sufficient for field identification purposes. In addition to these requirements, the identification, storage, and handling of the geotextile shall conform to ASTM D 4873.

Wall construction. The base of the wall shall be graded to a smooth, uniform condition free of ruts, potholes, and protruding objects, such as rocks or sticks. The geotextile shall be spread immediately ahead of the covering operation.

Wall construction shall begin at the lowest portion of the excavation, and each layer shall be placed horizontally, as shown in the plans. Each layer shall be entirely completed before the next layer is started. Geotextile splices transverse to the wall face will be allowed, provided that the minimum overlap is 2 feet or that the splice is sewn together. Geotextile splices parallel to the wall face will not be allowed. The geotextile shall be stretched out in the direction perpendicular to the wall face to ensure that no slack or wrinkles exist in the geotextile prior to backfilling.

Under no circumstances shall the geotextile be dragged through mud or over sharp objects which could damage the geotextile. The fill material shall be placed on the geotextile in such a manner that a minimum of 6 inches of material will be between the vehicle or equipment tires or tracks and the geotextile at all times. Particles within the backfill material greater than 3 inches in size shall be removed. The turning of vehicles on the first lift above the geotextile will not be permitted. End-dumping the fill directly on the geotextile will not be permitted.

Should the geotextile be torn or punctured or the overlaps or sewn joints disturbed (i.e., there is visible geotextile damage, subgrade pumping, intrusion, or distortion) the backfill around the damaged or displaced area shall be removed. The damaged geotextile section shall be replaced by the contractor with a new section of geotextile at no cost to the State.

If geotextile seams are to be sewn in the field or at the factory, the seams shall have two parallel rows of stitching. The two rows of stitching shall be 1/2 inch apart, with a tolerance of ±1/4 inch, and shall not cross, except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance (i.e., the minimum distance from the geotextile edge to the stitch line nearest to that edge) shall be 1-1/2 inches if a flat or prayer seam, type SSa-2, is used. The minimum seam allowable for all other seam types shall be 1 inch. The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the engineer. The seam strength will be tested and shall meet the requirements stated in this special provision.

A temporary form system shall be used to prevent sagging of the geotextile facing elements during construction. A typical example of a temporary form system and sequence of wall construction required when using this form are shown in the plans.

Pegs, pins, or the manufacturer's recommended method, with the forming system, shall be used when needed to hold the geotextile in place until the specified cover material is placed.

The wall backfill shall be placed and compacted according to the wall construction sequence shown in the plans. The minimum compacted backfill lift thickness of the first lift above each geotextile layer shall be 6 inches. The maximum compacted lift thickness anywhere within the wall shall be 8 inches or half of the geotextile layer spacing, whichever is less.

Each layer shall be compacted to 95 percent of the maximum density. The water content of the wall backfill shall not deviate above the optimum water content by more than 3 percent. Sheep's foot rollers, other rollers with protrusions, and full-size vibratory rollers will not be allowed. Small vibratory rollers will only be used with the approval of the engineer. Compaction within 3 feet of the wall face shall be achieved using light, mechanical tampers approved by the engineer and shall be done in a manner to cause no damage or distortion to the wall facing elements or reinforcing layers.

If corners must be constructed in the geotextile wall due to abrupt changes in the alignment of the wall face as shown in the plans, the method used to construct the geotextile wall corner(s) shall be submitted to the engineer for approval at least 14 calendar days prior to beginning construction of the wall. The corner must provide a positive connection between the sections of the wall on each side of the corner such that the wall backfill material cannot spill out through the corner at any time during the design life of the wall. Furthermore, the corner must be constructed in such a manner that the wall can be constructed with the full geotextile embedment lengths shown in the plans in the vicinity of the corner in both directions.

The method of ending a geotextile wall layer at the top of the wall where changes in wall top elevation occur shall also be submitted for approval with the wall corner details submittal. The end of each layer at the top of the wall must be constructed in a manner which prevents wall backfill material from spilling out the face of the wall throughout the life of the wall. If the profile of the top of the wall changes at a rate of 1:1 or steeper, this change in top of wall profile shall be considered a corner. Also, wall angle points with an interior angle of less than 150 degrees shall be considered a corner.

The base of the excavation shall be completed to within ± 3 inches of the staked elevations, unless directed by the engineer. The external wall dimensions shall be placed to within ± 2 inches of that staked on the ground. Each layer and overlap distance shall be completed to within ±1 inch of that shown in the plans.

The maximum deviation of the wall face from the batter shown in the plans shall not be greater than 3 inches for permanent walls, and 5 inches for temporary walls. The face batter measurement shall be made at the midpoint of each wall layer. Each wall layer depth shall be completed to within ±1 inch of that shown in the plans.

If the wall is to be a permanent structure, the entire wall face shall be coated with a reinforced shotcrete facing, as detailed in the plans and described in this special provision.

Alignment Control. Noncorroding alignment wires and thickness control pins shall be provided to establish thickness and plane surface. Alignment wires shall be installed at corners and offsets not established by formwork. The contractor shall ensure that the alignment wires are tight, true to line, and placed to allow further tightening. Alignment wires shall be removed after wall construction is complete.

Placement of Shotcrete Wall Facing

Qualifications of contractor's personnel. All nozzlemen on this project shall have had at least 1 year of experience in the application of shotcrete. Each nozzleman will be qualified, by the engineer, to place shotcrete on this project after successfully completing one test panel for each shooting position which will be encountered, and the results of the 7-day strength tests are known. A member of the personnel shall not place shotcrete without these specific qualifications.

The contractor shall notify the engineer no less than 2 days prior to the shooting of a qualification panel. The mix design for the shotcrete shall be the same as that slated for the wall being shot.

Qualification will be based on a visual inspection of the shotcrete density, void structure, and finished appearance along with a minimum 7-day compressive strength of 2,500 psi determined from the average test results from two cores taken from each panel.

If shotcrete finish alternate B or C is specified by the plans, all shotcrete crew members shall have completed at least three projects in the last 5 years where such finishing, sculpturing and texturing, of shotcrete was performed. Evidence of this experience, in addition to approval of the shotcrete finish qualification panel, will form the basis for qualification of nozzlemen to perform the work as specified by shotcrete finish alternate B or C. Shotcreting shall not begin until this qualification process is completed.

Placing wire reinforcement. The reinforcement of the shotcrete shall be placed as shown in the plans. The wire reinforcement shall be securely fastened to the no. 3 epoxy coated rebar so that it will be 1 to 1-1/2 inches from the wall face at all locations. Wire reinforcement shall be lapped 1-1/2 squares in all directions.

Shotcrete construction requirements. A clean, dry supply of compressed air sufficient for maintaining adequate nozzle velocity for all parts for the work and for simultaneous operation of a blow pipe for cleaning away rebound shall be maintained at all times. Thickness, method of support, air pressure, and rate of placement of shotcrete shall be controlled to prevent sagging or sloughing of freshly applied shotcrete.

The shotcrete shall be applied from the lower part of the area upwards so that rebound does not accumulate on the portion of the surface that still has to be covered. Surfaces to be shot shall be damp, and without free-standing water. No shotcrete shall be placed on dry, dusty, or frosty surfaces. The nozzles shall be held at a distance and angle approximately perpendicular to the working face so that rebound will be minimal, and the compaction will be maximized. Shotcrete shall emerge from the nozzle in a steady uninterrupted flow. When, for any reason, the flow becomes intermittent, the nozzle shall be diverted from the work until a steady flow resumes.

Surface defects shall be repaired as soon as possible after the initial placement of the shotcrete. All shotcrete that lacks uniformity, exhibits segregation, honeycombing, or lamination, or containing any dry patches, slugs, voids, or sand pockets shall be removed and replaced with fresh shotcrete by the contractor to the satisfaction of the engineer at no additional cost to the contracting agency.

Construction joints in the shotcrete shall be uniformly tapered over a minimum distance of twice the thickness of the shotcrete layer, and the surface of the joints shall be cleaned and thoroughly wetted before any adjacent shotcreting is performed. Shotcrete shall be placed in a manner that provides a shotcrete finish with a uniform texture and color across the finished construction joint.

The shotcrete shall be cured by applying a clear curing compound as specified in section 9-23. The curing compound shall be applied immediately after the final gunning. The air in contact with shotcrete surfaces shall be maintained at temperatures above 40° F for a minimum of 7 days. Curing compounds shall not be used on any surfaces against which additional shotcrete or other cement-like finishing materials are to be bonded, unless positive measures, such as sandblasting, are taken to completely remove curing compounds prior to the applications of such additional materials.

If field inspection or testing as directed by the engineer indicates that any shotcrete produced by the contractor fails to meet the requirements of this special provision, the contractor shall immediately modify procedures, equipment, or system as necessary and as approved by the engineer to produce specification material. All substandard shotcrete already placed shall be repaired by the contractor to the satisfaction of the engineer at no additional cost to the contracting agency. Such repairs may include the removal and replacement of all affected materials.

Shotcrete finishing. The shotcrete face shall be finished using the alternate aesthetic treatment indicated in the plans. The alternates are the following:

> **Alternate A**: After the surface has taken its initial set (crumbling slightly when cut), the surface shall be broom-finished to secure a uniform surface texture.
>
> **Alternate B**: Place the shotcrete a fraction beyond the alignment wires and forms. Allow it to stiffen to the point where the surface will not pull or crack when screened with a rod or trowel. Excess material shall then be trimmed, sliced, or scraped to the true lines and grade. Alignment wires shall then be removed. The surface shall receive a steel trowel finish, leaving a smooth uniform texture and color. Once the shotcrete has cured, a pigmented sealer shall be applied to the shotcrete face according

to the special provisions. The shotcrete surface shall be completed to within a tolerance of ±1/4 inch of true line and grade.

Alternate C: The shotcrete shall be hand-sculptured, colored, and textured to simulate the relief, jointing, and texture of the natural backdrop surrounding the wall. The ends and base of the wall shall transition in appearance as appropriate to more nearly match the color and texture of the adjoining roadway fill slopes. This may be achieved by broadcasting fine and coarse aggregates, rocks, and other native materials into the final surface of the shotcrete while it is still wet, allowing sufficient embedment into the shotcrete to become a permanent part of the surface.

Measurement. Geotextile retaining wall will be measured by the square foot of face of the completed wall. The shotcrete wall facing will be measured by the square foot of completed shotcrete wall facing.

Payment. The unit contract prices per square foot for "geotextile retaining wall" and "shotcrete wall facing," per ton for "gravel borrow including haul," and per cubic yard for "structure excavation class A" shall be full pay to complete the work in accordance with these specifications, including compaction of the backfill material and the temporary forming system.

Hilfiker Welded Wire Wall Technical Specifications

General

Description. This work shall consist of welded wire walls constructed accordance with these specifications and in reasonably close conformity with the lines, grades, design, and dimensions shown on the plans or established by the engineer.

Materials

The contractor shall make his own arrangements to purchase all welded wire wall materials, including wire mesh reinforcement mats, backing materials, and all necessary incidentals from Hilfiker Retaining Walls, P.O. Box 2012, Eureka, CA 95502-2012, telephone 707-443-5093.

Wire reinforcement and cap mesh. Wire mesh for facing shall be formed by a 90-degree bend of the soil wire reinforcement mesh and shall have a prebent tie to connect to the soil reinforcing mesh above. The reinforcing mesh shall be shop-fabricated of cold-drawn steel wire conforming to the minimum requirements of ASTM A-82 and shall be welded into the finished mesh fabric according to ASTM A-185. Fabric for the welded wire wall shall be per project specifications, either brite basic (nongalvanized), commercial galvanized (0.4 oz/SF), or hot dip galvanized (2.0 oz./SF, ASTM A-123). Any damage done to the mesh galvanization prior to installation shall be repaired in an acceptable manner.

Backing materials

Backing mats: Where required, as shown on the plans, steel backing mat shall be W1.7 x W1.7 (minimum) welded wire fabric meeting ASTM A-185 and galvanized according to either (a), (b), or (c) in the previous paragraph.

Backfill not conforming to this specification shall not be used without a written consent of the engineer.

The contractor shall furnish the engineer with a certificate of compliance certifying that the select granular backfill material complies with this section of the specifications. A copy of all test results performed by the contractor, which are necessary to assure compliance with the specifications, shall be furnished to the engineer.

The frequency of sampling of select granular backfill necessary to assure gradation shall be directed by the engineer.

Construction Requirements

Wall Excavation

Unclassified excavation shall be according to the requirements of general specifications and in reasonably close conformity with the limits and construction stages shown on the plans.

Foundation

The foundation for the structure shall be graded level for a width equal to or exceeding the length of the reinforcement mat, or as shown on the plans. Prior to the wall construction, the foundation, if not in rock, shall be compacted, as directed by the engineer. Any foundation soils found to be unsuitable shall be removed and replaced, as directed by the engineer.

Wall Erection

Wire mesh reinforcement mats, and applicable facing materials, shall be placed in successive horizontal lifts in the sequence shown on the plans as backfill placement proceeds. Vertical tolerance (plumbness) and horizontal alignment tolerance shall not exceed 2 inches when measured at the junction of the wire facing the soil reinforcement along a 10-foot straight edge.

The overall vertical tolerance of the wall (top and bottom) shall not exceed 1 inch per 10 feet of wall height, unless the wall design requires a battered facing. For battered facing structures, the overall tolerance from the theoretical battered locations shall not exceed 1 inch per 10 feet of battered wall height.

The quantity to be paid for shall be measured on the basis of wall face area shown on the plans.

Measurement and payment for excavation and backfill performed during welded wire wall construction will be according to the applicable sections of the contract specifications.

Wall Erection

The unit of measurement for wall erection will be the square foot of wall surface area complete and in place. The quantity to be paid for will be the actual quantity erected in place at the site. Payment shall include compensation for all labor and materials required to prepare the wall foundation. Place the reinforcement mats and position the backing mats and screens as shown on the plans.

Hilfiker retaining walls are covered by one or more of the following patents:

3,631,682	4,068,482	4,329,089	
3,922,864	4,117,686	4,324,508	Other
243,697	4,051,570	4,343,572	Patents
243,613	4,266,890	4,391,557	Pending
4,154,554	4,260,296	4,505,621	

Tensar Specifications

These guidelines have been developed to aid in the preparation of construction and material specifications for a specific project. These guidelines should be modified to:

- Incorporate specific modular concrete facing unit criteria.
- Incorporate any special project requirements.
- To delete any unnecessary requirements.
- To provide format and wording consistent with other project specifications.
- To provide consistency with construction drawings.

These guideline specifications include guidelines for the physical and mechanical properties of modular concrete block units and geogrid reinforcements that are of primary importance in ensuring satisfactory long-term performance of these retaining walls.

Material Specifications for Tenswal System

General

Description. This work shall consist of furnishing and constructing a Tenswal retaining wall system according to these specifications and in reasonably close conformity with the lines, grades, and dimensions shown on the plans or established by the engineer.

This specification is intended to cover the Tenswal system.

Work included:

(1) Furnishing Tensar structural geogrid reinforcement as shown on the construction drawings.

(2) Furnishing modular concrete facing units as shown on the construction drawings.

(3) Storing, cutting, and placing Tensar structural geogrid reinforcement as specified in this guide and as shown on the construction drawings.

(4) Excavation, placement, and compaction of the wall fill and backfill material as specified in this guide and as shown on the construction drawings.

Related Work. Site preparation, section 02100, and earthwork, section 02200.

Reference documents

(1) Geosynthetic Research Institute

GG1-87	*Standard Test Method for Geogrid Rib Tensile Strength*
GG2-87	*Standard Test Method for Geogrid Junction Strength*

(2) American Association of State Highway and Transportation Officials

T-99-90	*Moisture-Density Relations of Soils Using a 5-1/2 Pound Rammer in a 12-inch Drop*

(3) American Society for Testing and Materials Standards

C 33-90	*Specification for Concrete Aggregates*
C 140-90	*Methods of Sampling and Testing Concrete Masonry Units*
C 150-89	*Specification for Portland Cement*
C 331-89	*Specification for Lightweight Aggregates for Concrete Masonry Units*
C 595-89	*Specification for Blended Hydraulic Cements*
C 618-91	*Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete*
C 989-91	*Specification for Ground Granulated Blast Furnace Slag Cement*

(4) Where specifications and reference documents conflict, the engineer shall make a final determination of the applicable document.

Materials

Definitions

(1) Structural geogrid: A Tensar structural geogrid formed by a regular network of integrally connected tensile elements with apertures of sufficient size to allow interlocking with surrounding soil, rock, or earth, and function primarily as reinforcement.

(2) Modular concrete facing units: hollow and solid modular concrete wall units, machine-made from Portland cement, water, and mineral aggregates with or without the inclusion of other materials.

(3) Unit fill: Granular soil which is within and immediately behind the modular concrete facing units.

(4) Reinforced backfill: Compacted soil which is within the reinforced soil volume as outlined on the plans.

Structural geogrids

(1) The geogrids shall be a regular grid structure of select high density polyethylene or polypropylene resin produced by the Tensar Corporation.

(2) The minimum allowable junction strength of the geogrid, as per G.R.I.-GG2, shall be equal to or greater than 80 percent of the ultimate strength of the geogrid, as per G.R.I.-GG1.

(3) The geogrid connection to the modular concrete facing units shall be capable of carrying 100 percent of the maximum design tensile load of the geogrid at no more than 3/4 inch total deformation.

(4) The maximum design tensile load of the geogrid shall be less than or equal to 50 percent of the tested ultimate strength of the connection between the geogrid and the modular concrete facing unit.

(5) The manufacturer shall provide certification of the ultimate strength and junction strength as per GG2 of the specified product (with accompanying test results if requested by the engineer).

(6) The manufacturer shall provide the certification that the ultimate strength of the geogrid as per GG1 is equal to or greater than the ultimate strength called for on the drawings.

(7) The manufacturer shall furnish the engineer with a written certification that all purchased resin used to produce the structural geogrid is virgin resin.

Modular concrete facing units

(1) **Materials**

(a) Cement-like materials: Materials shall conform to the following applicable specifications:

- Portland Cement—Specification C 150.

- Modified Portland cement—Portland cement conforming to specification C 150, modified as follows: limestone-calcium carbonate, with a minimum 85 percent ($CaCO_3$) content, may be added to the cement, provided these requirements of specification C 150 as modified are met: the limitation on insoluble residue (1.5 percent), limitation on air content of mortar (volume percent, 22 percent maximum), and limitation on loss of ignition (7 percent).

- Blended cements—specification C 595.

- Pozzolans—specification C 618.

- Blast furnace slag cement—specification C 989.

Note: Sulphate resistant cement should be used in the manufacture of units to be used in areas where the soil has high sulphate content, such as in arid regions of the western United States.

(b) Aggregates: Aggregates shall conform to the following specifications:

- Normal weight aggregates—Specification C 33.

- Lightweight aggregates—Specification C 331.

(c) Other constituents: Air-entraining agents, coloring pigments, integral water repellents, finely ground silica, and other constituents shall be previously established as suitable for use in concrete modular retaining wall units and shall conform to applicable ASTM standards, or shall be shown by test or experience to be not detrimental to the durability of the concrete modular retaining wall units or any material customarily used in masonry construction.

(2) **Physical requirements**

(a) Concrete wall units shall have a minimum compressive strength after 28 days of 3,000 psi. The concrete units shall have the required freeze/thaw protection with a maximum absorption rate of 6 pounds per cubic foot in southern climates, and 8 pounds per cubic foot in northern climates.

(3) **Manufacture tolerance**

(a) Modular unit dimensions shall not differ more than ±1/8 inch (3.2 mm) from the manufacturer's published dimension.

(4) **Finish and appearance**

(a) All units shall be sound and free of cracks or other defects that would interfere with the proper placing of the unit or significantly impair the strength or permanence of the construction. Minor cracks incidental to the usual method of manufacture, or minor chipping resulting from shipment and delivery, are not grounds for rejection.

(b) The exposed surfaces of units shall be free of chips, cracks, or other imperfections when viewed from a distance of 10 feet under diffused lighting.

(5) **Sampling and testing**

(a) The purchaser or authorized representative shall be accorded proper facilities to inspect and sample units from lots ready for delivery.

(b) Sample and test units for compressive strength and absorption according to the applicable provisions of ASTM method C140-90. Compressive strength test specimens shall conform to the saw-cut coupon provisions of section 5.2.4 of ASTM C140-90 with the following exceptions: coupons shall have a minimum thickness of 1-1/2 inches (38.1 mm).

(6) **Rejection**

(a) If the shipment fails to conform to the specified requirements, new specimens shall be selected by the purchaser from the retained lot at the expense of the manufacturer. If the second set of specimens fails to conform to the test requirements, the entire lot shall be rejected.

(7) Acceptable modular concrete facing unit manufacturers shall be approved by Tensar Earth Technologies, Inc.

Delivery, storage, and handling

(1) **Structural Geogrid**

(a) The contractor shall check the geogrid upon delivery to ensure that the proper material has been received.

(b) Geogrids shall be stored above –20°F (–29°C).

(c) The contractor shall prevent excessive mud, wet cement, epoxy, and similar materials from coming in contact with and affixing to the geogrid material.

(d) Rolled geogrid material may be laid flat or stood on its end for storage.

(2) **Modular concrete facing units**

(a) The contractor shall check the units upon delivery to ensure that proper materials have been received.

(b) The contractor shall prevent excessive mud, wet cement, epoxy, and similar materials from coming in contact with and affixing to the units.

(c) The contractor shall protect the units from damage (i.e., cracks, chips, spalls). Damaged units shall be evaluated for usage in the wall according to ASTM C-90-75 (1981 rev.) and ASTM C-145-75 (1981 rev.).

Fill and foundation materials

(1) Foundation base material: Material for leveling pad shall consist of compacted sand, gravel, and/or unreinforced concrete as shown on the construction drawings.

(2) Unit fill: Fill for units shall consist of well-graded 3/4 inch minus crushed stone or granular fill with the gradation listed below.

Sieve size	Percent passing unit fill
2 inches	100
3/4 inch	100
No. 4	0–60
No. 40	0–50
No. 200	0–5

(a) A minimum of 12 inches of drainage fill must extend behind the wall.

(b) Provide a drainage zone behind the wall units to within 1 foot of the final grade. Cap backfill with 1 foot of impervious material.

(3) Reinforced backfill: Backfill requirements for the Tenswal system:

Sieve size	Percent passing reinforced backfill
2 inches	100–75
3/4 inch	100–75
No. 4	100–20
No. 40	0–60
No. 200	0–40

The maximum aggregate size shall be limited to 3/4 inch, unless field tests have been or will be performed to evaluate the potential strength reductions to the geogrid due to damage during construction. The plasticity index (PI) is less than or equal to 20 and a liquid limit is less than or equal to 30.

(a) Material shall be site–excavated material where the requirements previously mentioned can be met. Where additional fill is required, or site-excavated soils are not suitable, the contractor shall submit a sample to the engineer, who will determine if it is acceptable.

Execution

Construction

(1) The contractor shall excavate to the lines and grades shown on the construction drawings. The contractor shall be careful not to disturb the base beyond the lines shown.

Subgrade preparation

(1) Subgrade shall be excavated as required for the placement of leveling pad as shown on the construction drawings, or as directed by the engineer.

(2) The subgrade shall be examined by the engineer to ensure that the actual foundation conditions meet or exceed the assumed design assumptions. Subgrade conditions not meeting the required strength shall be removed and replaced with acceptable material.

(3) Over-excavated areas shall be replaced with compacted backfill material to the lines and grade shown on the construction drawings.

(4) As a minimum, soil shall be proof-rolled before construction proceeds.

Modular concrete facing unit installation

(1) The first course of concrete retaining wall units shall be placed on top of and in full contact with the levelling pad. The units shall be checked for proper elevation and alignment.

(2) Units shall be placed side by side for the full length of the wall. Proper alignment may be achieved with the aid of a string line or offset from the baseline.

(3) Connecting pins (if required) shall be installed and the voids in and/or around the unit filled with tamped unit fill.

(4) All excess material shall be swept from the top of units prior to installing the next course. Each course shall be completely filled prior to proceeding to the next course.

(5) Units shall be laid to create the minimum radius possible, or as otherwise shown on the construction drawings. Units shall be installed so that only the front face of the units shall be visible.

Geogrid installation

(1) Geogrid shall be oriented with the highest strength axis perpendicular to the wall alignment.

(2) Geogrid reinforcement shall be placed at the elevation(s) and to the extent(s) shown on the construction drawings, or as directed by the engineer.

(3) The geogrid soil reinforcement shall be laid horizontally on the compacted backfill. Place the next course of wall units over geogrid. The geogrid shall be pulled taut and anchored prior to backfill placement on the geogrid.

(4) Geogrid reinforcements shall be continuous throughout their embedment length(s). Spliced connections between shorter pieces of geogrid will not be allowed unless approved by the engineer prior to construction.

Reinforced backfill placement

(1) Reinforced backfill shall be placed, spread, and compacted in a manner that minimizes the development of slack in the geogrid.

(2) Reinforced backfill shall be placed and compacted in lifts not to exceed 6 inches where hand-compaction is used, or 10 inches where heavy compaction equipment is used.

(3) Reinforced backfill shall be compacted to 90 percent of the maximum density, as determined by AASHTO T-99. The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer and shall be within 2 percentage points (dry) of optimum.

(4) Only lightweight hand-operated compaction equipment shall be allowed within 3 feet of the facing units.

(5) Tracked construction equipment shall not be operated directly upon the geogrid reinforcement. A minimum fill thickness of 6 inches is required prior to operation of tracked vehicles over the geogrid. Tracked vehicle turning should be kept to a minimum to prevent tracks from displacing the fill and damaging the geogrid.

(6) Rubber-tired equipment may pass over the geogrid reinforcement at slow speeds less than 10 MPH. Sudden breaking and sharp turning shall be avoided.

(7) At the end of each day's operation, the contractor shall slope the last lift of reinforced backfill away from the wall facing to rapidly direct the runoff away from the wall face. In addition, the contractor shall not allow the surface runoff from adjacent areas to enter the wall construction site.

Measurement and Payment

(1) Payment shall be considered full compensation for all labor, materials, and equipment to install Tensar structural geogrids, modular concrete retaining wall units, wall fill, reinforced backfill, leveling pads, and clean up.

(2) Depending on existing topography, quantities may vary from that shown on construction drawings. Changes to the total quantity of materials will be at the contract unit price bid.

(3) The Tenswal system, a geogrid reinforced soil retaining wall system with concrete facing wall units, shall be paid for on a square foot of wall surface basis at the contract unit price. Measurement of the square foot of wall shall include all materials supplied and installed by the contractor. Measurement and payment shall include all necessary materials, labor, dewatering, supervision, equipment, engineering, design submittals, and incidentals to complete the retaining wall.

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