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16. Abstract

This document presents information on the analysis, design, and construction of soil nail walls in highway applications. The main objective is to provide practitioners in this field with sound and simple methods and guidelines that will allow them to analyze, design, and construct safe and economical structures. This document updates information contained in FHWA-SA-96-069R (Byrne et al., 1998). The focus is on soil nailing techniques that are commonly used in the U.S. practice. The contents of this document include: an introduction, a chapter on applications and feasibility, descriptions and guidelines for field and laboratory testing in soil nailing applications, descriptions of the common U.S. practice, analysis and design of soil nail walls, chapters on contracting approach and technical specifications and design examples. Because of the popularity of the Allowable Stress Design (ASD) method [also known as Service Load Design (SLD)] among practitioners, the methods presented in this document are based on the ASD method.

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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**When You Know**	**Multiply By**	**To Find**	**Symbol**
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
ml	millimeters	0.034	fluid ounces	fl oz
m^3	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
l	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
TEMPERATURE				
ºC	Celsius	1.8 C + 32	Fahrenheit	ºF
WEIGHT DENSITY				
kN/m^3	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / square inch	psi
kPa	kilopascals	20.9	poundforce / square foot	psf

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PREFACE

The purpose of this Geotechnical Engineering Circular (GEC) is to facilitate the planning, design, and construction of soil nail walls used in highway engineering applications. The intended audience for this document is geotechnical, structural, and highway design and construction specialists involved in the design, construction, and inspection of soil nail wall systems. This GEC aims to serve as the FHWA reference document for highway projects involving soil nail walls.

The technique of soil nailing has been increasing its popularity among contractors because it offers an effective and cost-effective retaining system for a variety of ground conditions. The technique has been used extensively in Europe for the last 30 years. In the Unites States, the past 10 years have seen a continued interest in its applications.

This document draws the basics of the description and design methods from earlier FHWA publications in this field, in particular, the predecessor manual entitled "Manual for Design and Construction Monitoring of Soil Nail Walls," Report FHWA-DP-96-69R (Byrne et al., 1998). Valuable information was obtained from the publication entitled "Ground Nailing Demonstration Project 103, Guideline Manual and Workshop" (Chassie, R.G., 1994). The primary references used in the preparation of this GEC include the following FHWA references:

- Elias, V. and Juran, I. (1991). "Soil Nailing for Stabilization of Highway Slopes and Excavations," Publication FHWA-RD-89-198, Federal Highway Administration, Washington D.C.

- FHWA (1993a). "FHWA International Scanning Tour for Geotechnology, September–October 1992—Soil Nailing Summary Report," Publication SA-96-072, Federal Highway Administration, Washington, D.C.

- FHWA (1993b). "French National Research Project Clouterre, 1991-Recommandations Clouterre 1991," (English Translation) Soil Nailing Recommendations, Publication FHWA-SA-93-026, Federal Highway Administration, Washington, D.C.

- Porterfield, J. A., Cotton, D. M., and Byrne, R. J. (1994). "Soil Nailing Field Inspectors Manual, Project Demonstration 103," Publication No. FWHA-SA-93-068, Federal Highway Administration, Washington, D.C.

- Chassie, R.G. (1994). "FHWA Ground Nailing Demonstration Project, Guideline Manual and Workshop," Federal Highway Administration, Washington, D.C.

- Byrne, R.J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G. (1998). "Manual for Design and Construction Monitoring of Soil Nail Walls," Report FHWA-DP-96-69R, Federal Highway Administration, Washington, D.C.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AL	Alignment load
ASD	Allowable stress design method
ASTM	American Society of Testing Materials
B	Buoyancy load effect
CALTRANS	California Department of Transportation
CF	Centrifugal Forces on Bridge Deck
CIP	Cast-in-place reinforced concrete
CPT	Cone penetration test
CU	Consolidated undrained triaxial compression
D	Dead load effect
DMT	Flat plate dilatometer test
E	Earth pressure effect
EA	Entrained air
EQ	Earthquake effect
GEC	Geotechnical engineering circular
HDPE	High-Density polyethylene
I	Live load impact effect
ICE	Ice pressure effect
L	Live load effect
LF	Longitudinal forces on bridge deck effect
LRFD	Load and resistance factor design method
M-O	Mononobe-Okabe method
MSE	Mechanically stabilized earth
OD	Outer diameter
PMT	Pressuremeter test
PTI	Post-Tensioning Institute
PVC	Polyvinyl chloride
QA/QC	Quality Assurance/Quality Control
R+ S+ T	Rib shortening, shrinkage, and temperature effect
RMR	Rock Mass Rating System
ROW	Right-of-way
RQD	Rock Quality Designation
RSR	Rock Structure Rating
SF	Stream current pressure effect
SI	International System of Units
SLD	Service Load method
SMS	Slope Rock Mass Rating System
SM	Silty sand

SPT	Standard penetration test
SW	Well-graded sand
TYP	Typical
UC	Unconfined compressive strength
URCS	Unified Rock Classification System
USCS	Unified Soil Classification System
UU	Unconsolidated-undrained triaxial compression
VST	Field vane shear test
W	Wind loads on structure effect
WL	Wind loads on live load effect
WWM	Welded wire mesh

LIST OF SYMBOLS

a_{hm}	Horizontal reinforcement cross sectional area per unit width at midspan
a_{hn}	Horizontal reinforcement cross sectional area per unit width at nail head
a_{ij}	Reinforcement cross section area per unit width in “i” direction and “j” location
a_{vm}	Vertical reinforcement cross sectional area per unit width at midspan
a_{vn}	Vertical reinforcement cross-sectional area per unit width at nail head
$a_{S\ MIN}$	Mesh minimum reinforcement cross sectional area per unit width
$a_{S\ MAX}$	Mesh maximum reinforcement cross sectional area per unit width
A	Normalized peak ground acceleration coefficient
A	Tangent of angle to calculate orientation of failure plane behind wall
A_E	Effective cross-sectional area of threaded bolts (anchors)
A_H	Cross-sectional area of stud head
A_{hw}	Total cross-sectional area of waler bars in horizontal direction
A_{vw}	Total cross-sectional area of waler bars in vertical direction
A_m	Normalized horizontal acceleration at center of block
A_{SH}	Cross-sectional area of the headed-stud shaft
A_S	Cross-sectional area of waler bars
A_t	Nail bar cross-sectional area
B	Tangent of angle to calculate orientation of failure plane behind wall
B’	Width of influence in an excavation
B_e	Width of excavation
B_L	Length of horizontal failure surface along base
BSRF	Bond strength reduction factor
c_m	Mobilized cohesion
c*	Normalized cohesion = $c/\gamma H$

c'	Soil effective cohesion
c_b	Soil cohesion strength along base of failure surface
C	Soil dependent coefficient to estimate wall displacements in Table 5.2
C_{1F}	Correction factor for norm. max. nail force for D_{DH} other than 100 mm (4 in.)
C_{1L}	Correction factor for normalized nail lengths for D_{DH} other than 100 mm (4 in.)
C_{2F}	Correction factor for normalized maximum nail force for c* other than 0.02
C_{2L}	Correction factor for normalized nail lengths for c* other than 0.02
C_{3L}	Correction factor for normalized nail lengths for FS_G other than 1.35
C_B	Correction factor in SPT for borehole diameter
C_E	Correction factor in SPT for energy correction efficiency
C_F	Factor to consider non-uniform soil pressures behind facing
C_N	Normalization factor in SPT for effective overburden pressure
C_P	Correction factor to account for contribution of the soil support
C_R	Correction factor in SPT for rod length
C_{RT}	Reduction coefficient for maximum nail length during nail tests
C_S	Correction factor in SPT for sampling method
C_{ST}	Compression strut force in soil nail strut concept
C_u	Uniformity coefficient
d	Half-thickness of facing
d_e	Seismically induced wall lateral movement
dT	Differential of the tensile force (T) along the nail bar
D_B	Depth to strong deposit from bottom of excavation
D	Parameter accounting for geometric features in the Mononobe-Okabe equation
D'_C	Equivalent conical failure surface diameter at the center of facing
D_C	Conical failure surface diameter on back of facing
D_{DEF}	Horizontal distance behind wall where ground deformation may take place
D_{DH}	Average or effective diameter of the drillhole
D_E	Effective diameter of threaded bolt (anchor) core
D_H	Diameter of the stud head
D_r	Relative density
D_S	Diameter of the headed-stud shaft
DTL	Design load in nail test
E	Young's modulus of steel
f_c'	Concrete compressive strength
f_y	Nail bar yield strength
F_{in}	Equivalent inertia force acting at the center of gravity of block

$F_{in\ h}$	Horizontal component of force F_{in}
$F_{in\ v}$	Vertical component of inertia force F_{in}
F_I	Inertia force of lower block I
F_{II}	Inertia force of lower block II
FS_{FP}	Factor of safety against facing punching shear failure
FS_{FF}	Factor of safety against facing flexure failure
FS_G	Factor of safety against global failure
FS_H	Factor of safety against heave failure at bottom of excavation
FS_{HT}	Factor of safety against headed-stud tensile failure
FS_P	Factor of safety against pullout failure
FS_{SL}	Factor of safety for sliding stability
FS_T	Factor of safety against soil nail tensile failure
FS_{Tproof}	Factor of safety against tensile failure during proof production tests
FS_{Tver}	Factor of safety against tensile failure during verification tests
g	Acceleration of gravity
h	Thickness of facing
h_C	Effective depth of conical surface
H	Wall height
H_1	Effective height over which the earth pressure acts
H_{eq}	Equivalent wall height
i	Nail inclination
i	Subscript indicating the load type (in Table 5.4)
k	Hydraulic conductivity
k_h	Horizontal seismic coefficient
k_o	At-rest earth lateral pressure coefficient
k_v	Vertical seismic coefficient
K_A	Active earth pressure coefficient
K_{AE}	Total (static and dynamic) active pressure coefficient
L_e	Excavation or wall length
L	Total nail length
L_{BP}	Bearing plate length
$L_{BPT\ max}$	Maximum bonded soil nail length during proof tests
L_{BPT}	Bonded soil nail length during proof tests
$L_{BVT\ max}$	Maximum bonded soil nail length during verification tests
L_{BVT}	Bonded Soil nail length during verification tests
LI	Liquidity index
LL	Liquid limit
L_p	Pullout length or nail length behind the failure surface
L_S	Headed-stud length
L_F	Length of failure plane
L_{SN}	Length of compression strut nail

$L_{TOT\ NU}$	Total nail length in the non-uniform length pattern
$L_{TOT\ U}$	Total nail length in the uniform-length pattern
M	Flexure moment along nail bar
n	Subscript indicating load group number
n_t	Number of threads per unit length
N	Load group
N	Total number of nails in a given wall section
N	Uncorrected SPT blowcount
N_c	Bearing capacity factor
N_F	Normal force on failure surface
N_H	Number of headed-studs (usually 4)
N_K	Cone factor
N_{meas}	Measured SPT blowcounts
N_{60}	Corrected N-value for an energy efficiency of 60 percent
$(N_1)_{60}$	Normalized and corrected SPT blowcount
OCR	Overconsolidation ratio
pH	Potential of hydrogen
P	Maximum applied test load during verification
P_A	Active lateral earth force
P_a	Normal atmospheric pressure
P_{AE}	Total active thrust
PGA	Peak ground acceleration
PI	Plasticity index
P_L	Limit pressure in the PMT
q	Mobilized shear stress acting around the perimeter of the nail-soil interface
q_s	Distributed surface loading
q(x)	Mobilized bond strength as a function of the coordinate x
q_{ALL}	Allowable bond strength
q_u	Ultimate bond strength
Q	Mobilized pullout per unit length
Q_T	Surcharge load
Q_{ALL}	Allowable pullout capacity per unit length
Q_D	Permanent vertical load
Q_{in}	Load type (in Table 5.4)
Q_u	Pullout capacity per unit length
r_i	Ratio of length L_i of nail “i” to length L_1 of nail 1
R_c	Cohesive component of shear force S_F
R_F	Generic facing capacity
R_F	Shear force due to punching in facing
R_{FF}	Facing flexure capacity

R_{FP}	Facing punching shear capacity
R_{HT}	Nail head capacity against tensile failure of the headed studs
R_P	Nail Pullout Resistance
$R_{P\ ALL}$	Allowable Nail Pullout Resistance
R_T	Nail bar Tensile capacity
$R_{T\ ALL}$	Allowable nail tensile capacity
R_ϕ	Frictional component of shear force S_F
S	Site coefficient
S_F	Shear force on failure surface
S_H	Soil nail horizontal spacing
S_{HS}	Headed-stud spacing
S_{max}	Maximum soil nail spacing
S_u	Undrained shear strength of fine-grained soils
S_V	Soil nail vertical spacing
S_{V0}	Vertical distance from top of wall to first row of soil nails
S_{VN}	Vertical distance from deepest row of soil nails to bottom of excavation
t_H	Headed-stud head thickness
$t_{max\text{-}s}$	Maximum normalized design tensile force of all nails
t_P	Bearing plate thickness
T	Tensile force along the nail bar
T(x)	Tensile force along the nail bar as a function of the coordinate x
$T_{1..N}$	Tensile force in soil nails 1…N
T_{avg}	Average nail load calculated by SNAIL
$T_{avg\text{-}s}$	Average nail load calculated by SNAIL for $FS_G = 1$
T_{EQ}	Equivalent nail force
T_{max}	Maximum tensile force along the nail bar
$T_{max\text{-}s}$	Maximum nail load calculated for $FS_G = 1$
T_o	Nail bar tensile force at facing
T_T	Tensile force in soil nail for strut nail concept
UL	Unbonded nail length during verification or proof tests
V	Shear force along nail bar
V_F	Punching shear force acting through facing section
w_n	Soil in situ moisture content
W	Weight of block considered for stability
W_I	Weight of lower block I
W_{II}	Weight of upper block II
W/C	Water-cement ratio by weight in a shotcrete mix
x	Distance along nail length
α	Wall face batter angle (from vertical)
β	Backslope angle
β_E	Factor for earth loads in AASHTO-specified loads

β_{eq}	Equivalent backslope angle
β_{in}	Load combination coefficient for load type “i” and load group “n”
γ	Total unit weight of soil mass
γ_n	Load factor for a load group “n”
γ_w	Unit weight of water
γ_{sat}	Saturated soil unit weight
γ_d	Soil dry unit weight
δ	Wall-soil interface friction angle
δ	Lateral deflection of facing due to earth load
δ_h	Horizontal wall displacement at the top of the wall
δ_v	Vertical wall displacements at the top of the wall
ΔH	Slope rise up to bench
ΔH	Equivalent overburden
ΔK_{AE}	Coefficient of dynamic increment
ΔP_{AE}	Dynamic active thrust increment
ΔL	Total measured nail head movement during verification or proof tests
ΔL_{min}	Minimum acceptable nail head movement during verification or proof tests
ϕ'	Effective angle of internal friction
ϕ'_b	Effective angle of internal friction of the base of failure surface
ϕ_m	Mobilized friction angle
ϕ'_{rem}	Remolded friction angle
μ	Normalized allowable pullout resistance
θ	Inclination of wall face from horizontal
θ_1	Inclination of tensional force in the strut nail concept
θ_2	Inclination of compression force in the strut nail concept
ρ	Reinforcement ratio
ρ^*	Angle in the calculation of the orientation of failure plane
ρ_{ij}	Reinforcement ratio of reinforcement per unit width in “i” direction (vertical or horizontal) and at location “j” (nail head or midspan)
ρ_{hm}	Horizontal reinforcement ratio at midspan
ρ_{hn}	Horizontal reinforcement ratio at the nail head
ρ_{vm}	Vertical reinforcement ratio at midspan
ρ_{vn}	Vertical reinforcement ratio at the nail head
ρ_{tot}	Total reinforcement ratio
ρ_{max}	Maximum reinforcement ratio for facing flexural capacity
ρ_{min}	Minimum reinforcement ratio for facing flexural capacity
σ'_c	Effective consolidation pressure in a triaxial test
σ'_p	Effective preconsolidation pressure

σ'_{vo}	In situ effective vertical or overburden pressure
ΣD	Sum of driving forces in sliding failure mode
ΣL_a	Total nail length for case (a) Figure 6.3
Σ Normal forces	Sum of normal forces on failure plane
ΣR	Sum of horizontal resisting forces
Σ Tangent forces	Sum of tangent forces on failure plane
ω	Angle relating horizontal and vertical seismic coefficients
ξ	Angle of orientation of failure plane pseudo-static slope stability analysis
ψ	Inclination of failure plane

CHAPTER 1 INTRODUCTION

1.1 PURPOSE

This document provides state-of-the-practice information for the selection, analysis, design, and construction of soil nail walls in highway applications using the Allowable Stress Design (ASD) procedure. The intended audience for this document includes geotechnical, structural, and highway design and construction specialists involved in soil nail wall systems. The primary goal of this document is to provide the practitioner sufficient information to facilitate the safe and cost-effective use of permanent soil nail walls for a variety of transportation-related projects.

This document presents historical background information and a description of soil nail wall systems, step-by-step design procedures, simplified design charts, and general construction specifications. The document concludes with a detailed soil nail wall design example. This document provides sufficient information to confidently design soil nail walls in a wide range of ground conditions. Limitations related to the use of these systems in marginal ground conditions are also provided. Information provided herein is not intended to represent a prescriptive methodology; rather the information should be used in conjunction with good engineering judgment for specific projects.

1.2 BACKGROUND

1.2.1 Origins of Soil Nail Walls

The origins of soil nailing can be traced to a support system for underground excavations in rock referred to as the New Austrian Tunneling Method (Rabcewicz, 1964a, 1964b, 1965). This tunneling method consists of the installation of passive (i.e., not prestressed as for ground anchors) steel reinforcement in the rock (e.g., rockbolts) followed by the application of reinforced shotcrete. This concept of combining passive steel reinforcement and shotcrete has also been applied to the stabilization of rock slopes since the early 1960s (e.g., Lang, 1961). This ground-support technique relies on the mobilization of the tensile strength of the steel reinforcement at relatively small deformations in the surrounding ground. This support is enhanced by the continuity of the shotcrete. The combination of passive reinforcement and shotcrete when applied to soil, in lieu of rock, is termed soil nailing.

One of the first applications of soil nailing was in 1972 for a railroad widening project near Versailles, France, where an 18-m (59-ft) high cut-slope in sand was stabilized using soil nails (Rabejac and Toudic, 1974). Because the method was cost-effective and the construction faster than other conventional support methods, an increase in the use of soil nailing took place in France and other areas in Europe. In Germany, the first use of a soil nail wall was in 1975 (Stocker et al., 1979). The first major research program on soil nail walls was undertaken in Germany from 1975 through 1981 by the University of Karlsruhe and the construction company Bauer. This investigation program involved full-scale testing of experimental walls with a variety of configurations and the development of analysis procedures to be used in design (Gässler and Gudehus, 1981; Schlosser and Unterreiner, 1991). In France, the Clouterre research program, involving private and public participants, was initiated in 1986. This research effort consisted of

full-scale testing, monitoring of in-service structures, and numerical simulations (Schlosser, 1983; Clouterre, 1991).

1.2.2 History of Use and Development in the United States

Pioneering applications of soil nail walls in North America were for temporary excavation support in Vancouver, B.C, Washington, D.C. and Mexico City, Mexico in the late 1960s and early 1970s. One of the first published applications of soil nailing in the United States was the support of the 13.7-m (45-ft) deep foundation excavation in dense silty lacustrine sands for the expansion of the Good Samaritan Hospital in Portland, Oregon in 1976 (Byrne et al; 1998). The construction of the retaining system was reportedly conducted in nearly half the time and at about 85 percent of the cost of conventional excavation support systems. In 1984, a prototype soil nail wall 12-m (40-ft) high was built near Cumberland Gap, Kentucky, as part of a demonstration project funded by the U.S. Department of Transportation Federal Highway Administration (FHWA) (Nicholson, 1986). In 1989, the Oregon Department of Transportation built an 8-m (24-ft) high wall as the first application of a soil nail wall used in a bridge abutment cut wall (end-slope removal). In 1988, a 12.2-m (40-ft) high, 2-tiered wall was built along Interstate Highway 78, near Allentown, Pennsylvania. Each wall tier was 6.1-m (20-ft) high with a 3 m (10 ft) horizontal offset and was built in colluvium and highly weathered rock. Other examples of early uses of soil nail walls include those built along Interstate 10 in San Bernadino, California; Interstate 90 near Seattle, Washington; and along George Washington Parkway (Interstate 495) in Virginia (Byrne et al., 1998).

The use of soil nail walls has substantially increased in the United States during the last decade because it has been demonstrated that soil nail walls are technically feasible and, in many cases, a cost-effective alternative to conventional retaining walls used in top-to-bottom excavations in temporary and permanent applications. Design engineers are becoming increasingly familiar with soil nailing technology. Most soil nail walls constructed in the United States are still used for temporary retaining structures, however, the use of soil nail walls as a permanent structure has increased substantially in the last five years.

The more widespread use of soil nail walls today is due in large part to the efforts of FHWA through the Office of Research and Development. The first FHWA document on soil nailing was issued through FHWA's Office of Research and Development (Elias and Juran, 1991). The objective of this first document was to disseminate information to U.S. highway agencies and practitioners on the use of this technique as a retaining system in highway projects. In 1992, FHWA sponsored two-week long technical tours by various U.S. experts to France, Germany, and England (FHWA, 1993a). The objectives of the tour were to: (1) learn the then current European state-of-the-practice in soil nail wall technology; (2) update the available information on the mechanisms of soil nail wall performance, design approach, and computer programs; and (3) gather up-to-date construction specifications, corrosion protection details, and information and appropriate contracting practices. These efforts provided the basis for establishing subsequent research and development activities in the United States. In 1993, FHWA sponsored an English translation of the French practice summary on soil nailing (FHWA, 1993b). In 1994, a Soil Nailing Inspector's Manual was issued (Porterfield et al., 1994). Also in 1994, FHWA launched Demonstration Project 103 (Demo 103) to disseminate further the use of soil nail walls among state highway agencies.

From 1996 to 1998, FHWA and local agencies sponsored workshops around the country to provide guidance for the practice of soil nailing. Documents developed for Demo 103 served as a preliminary design guide and subsequently evolved as a design manual (Byrne et al., 1998). FHWA also funded various research projects in academic institutions.

1.3 DOCUMENT ORGANIZATION

This document provides an update to the FHWA design manual (Byrne et al., 1998) and incorporates recent trends in design methods, construction contracting, and construction monitoring. The design principles presented in this document are based on the ASD procedure. This document also presents new simplified charts that can be used in the preliminary design phase of a project, and discusses advantages and limitations of two computer programs, SNAIL and GOLDNAIL, developed for the analysis and design of soil nail walls.

The remainder of this document is organized as follows:

- *Chapter 2 – Applications and Feasibility Evaluations* – This chapter provides an overview of soil nailing and describes the various soil nail wall system components. This chapter also presents examples of applications and discusses favorable and unfavorable ground conditions for soil nailing, as well as criteria for feasibility evaluations.

- *Chapter 3 – Site Investigation, Laboratory Testing, and Recommendations* – This chapter describes basic field and laboratory testing techniques used to evaluate subsurface characteristics and stratigraphy for soil nail wall applications. In addition, this chapter presents recommendations of soil properties to be used in the analysis and design of soil nail walls.

- *Chapter 4 – Construction Materials and Methods* – This chapter provides information on major components of soil nail walls and information on construction sequence, equipment, and methods.

- *Chapter 5 – Analysis of Soil Nail Walls* – This chapter describes the failure modes considered in soil nail wall design, external and internal stability concepts, and soil nail wall deformation behavior. The features of the computer programs GOLDNAIL and SNAIL are also presented.

- *Chapter 6 – Design of Soil Nail Walls* – This chapter presents step-by-step design procedures, preliminary dimensioning charts, and other design concepts. A simplified step-by-step design example is included.

- *Chapter 7 – Contracting Approach and Specifications* – This chapter presents guidelines for specifications based on both the method and performance-based contracting approaches.

- *Chapter 8 – Construction Inspection and Performance Monitoring* – This chapter presents procedures for the inspection of construction materials and activities, field testing, and long-term monitoring of soil nail walls.

- *Chapter 9 – References* – This chapter provides a listing of the cited references.

- *Appendix A – Reinforcement Characteristics* – This appendix presents common threaded bar and welded wire mesh characteristics used in shotcrete facings.

- *Appendix B – Charts for Preliminary Design* – This appendix presents dimensionless design charts developed using the computer program SNAIL for various backslope angles, wall batter angles, and soil parameters.

- *Appendix C – Corrosion Protection* – This appendix discusses basic concepts of corrosion potential associated with soil nail walls, presents typical corrosion protection methods used in soil nail walls, and provides criteria to select appropriate corrosion protection methods.

- *Appendix D – Design Example* – This appendix presents a detailed, annotated design example demonstrating the methods presented in the document.

- *Appendix E – Construction Specifications for Soil Nail Walls* – This appendix provides specification templates for materials and construction methods typically used in soil nail walls.

- *Appendix F – User's Guide For SNAIL, Version 3.*09 – This appendix presents directions to implement the use of the program SNAIL in the analysis of soil nail walls.

CHAPTER 2 APPLICATIONS AND FEASIBILITY EVALUATIONS

2.1 INTRODUCTION

Soil nailing consists of the passive reinforcement (i.e., no post-tensioning) of existing ground by installing closely spaced steel bars (i.e., nails), which are subsequently encased in grout. As construction proceeds from the top to bottom, shotcrete or concrete is also applied on the excavation face to provide continuity. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to other retaining wall systems. For certain conditions, soil nailing offers a viable alternative from the viewpoint of technical feasibility, construction costs, and construction duration when compared to ground anchor walls, which is another popular top-to bottom retaining system. This chapter introduces some basic aspects of soil nailing, presents typical highway applications, and discusses criteria to be used in evaluating the feasibility of soil nail walls.

While the terms “soil nail wall” and “soil nailing” are broadly applied to soil systems, the technique is also applicable to excavations in soil-like materials (e.g., soft rock or weathered rock). In this document, the term “soil nail wall” is used regardless of the supported material.

This document addresses soil nails that are installed with a near horizontal orientation (i.e., inclination of 10 to 20 degrees below horizontal) and are primarily subjected to tensile stresses. Such soil nail systems are used to stabilize natural slopes and excavations. An alternative application of passive reinforcement in soil is sometimes used to stabilize landslides. In this case, the reinforcement (sometimes also called “nails”) is installed almost vertically and perpendicular to the base of the slide. In this alternative application, nails are also passive, installed in a closely spaced pattern approximately perpendicular to the nearly horizontal sliding surface, and subjected predominantly to shear forces arising from the landslide movement. However, this application of soil nails as a means to stabilize landslides is not often used in the current U.S. practice and is therefore not discussed in this document.

This document discusses the use of soil nail walls for both temporary and permanent structures. A structure can be characterized as temporary or permanent by its service life or intended duration of use. A structure with a service life of 18 months or less qualifies as temporary; a structure with a longer service life qualifies as permanent. If a structure is initially intended as temporary (e.g., temporary support of an excavation) but substantial construction delays may be expected such that the excavation will remain open much longer than 18 months, the structure should be characterized as permanent.

2.2 BASIC ELEMENTS OF A SOIL NAIL WALL

The most common practice for soil nailing in the United States consists of drilled soil nails, in which a steel bar is placed in a pre-drilled hole and then grouted (Figure 2.1). Figure 2.1 shows a cross-section of a typical soil nail wall where the following components are illustrated:

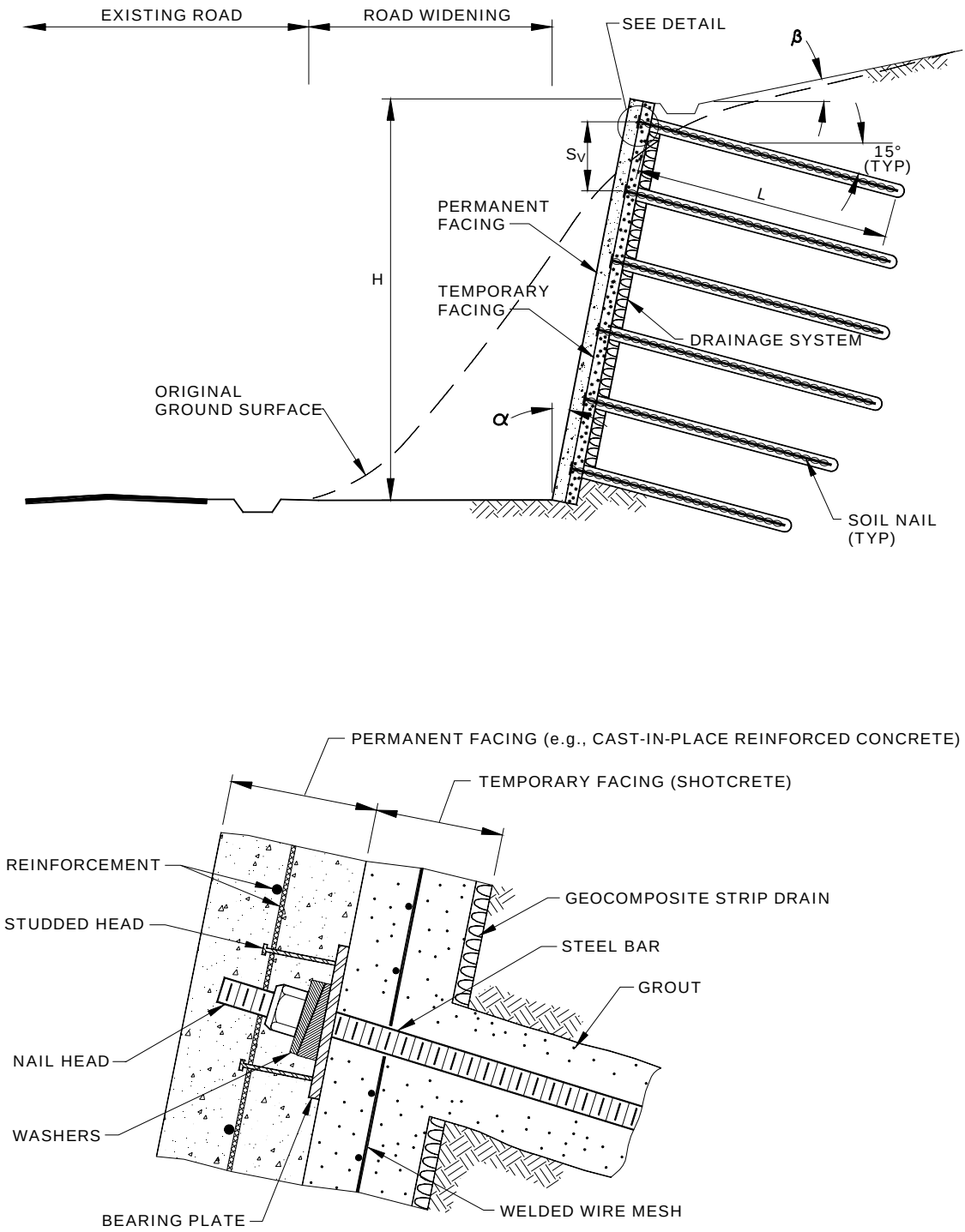


Figure 2.1: Typical Cross-Section of a Soil Nail Wall.

1. Steel reinforcing bars – The solid steel reinforcing bars are the main component of the soil nail wall system. These elements are placed in pre-drilled drillholes and grouted in place. Tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation activities.

2. Grout – Grout is placed in the pre-drilled borehole after the nail is placed. The grout serves the primary function of transferring stress from the ground to the nail. The grout also provides a level of corrosion protection to the soil nail.

3. Nail head – The nail head is the threaded end of the soil nail that protrudes from the wall facing.

4. Hex nut, washer, and bearing plate – These components attach to the nail head and are used to connect the soil nail to the facing.

5. Temporary and permanent facing – The facing provides structural connectivity. The temporary facing serves as the bearing surface for the bearing plate and support the exposed soil. This facing is placed on the unsupported excavation prior to advancement of the excavation grades. The permanent facing is placed over the temporary facing after the soil nails are installed and the hex nut has been tightened.

6. Geocomposite strip drainage – The geocomposite strip drainage systemmedia is placed prior to application of the temporary facing to allow collection and transmission of seepage water that may migrate to the temporary facing.

7. Additional corrosion protection (not shown) in Figure 2.

Complete descriptions of the soil nail wall components commonly used in the typical U.S. practice and shown in Figure 2.1 are presented in more depth in Chapter 4. Methods to provide corrosion protection are discussed in Chapter 4 and Appendix C.

2.3 CONSTRUCTION SEQUENCE

The typical sequence of construction for a soil nail wall using solid steel nail bars is described below and shown schematically in Figure 2.2.

Step 1. *Excavation.* Initial excavation is carried out to a depth for which the face of the excavation has the ability to remain unsupported for a short period of time, typically on the order of 24 to 48 hours. The depth of the excavation lift is usually between 1 and 2 m (3 and 6 ft) and reaches slightly below the elevation where nails will be installed. The width of the excavated platform or bench must be sufficient to provide access to the installation equipment.

Step 2. *Drilling Nail Holes*. Drillholes are drilled to a specified length, diameter, inclination, and horizontal spacing from this excavated platform.

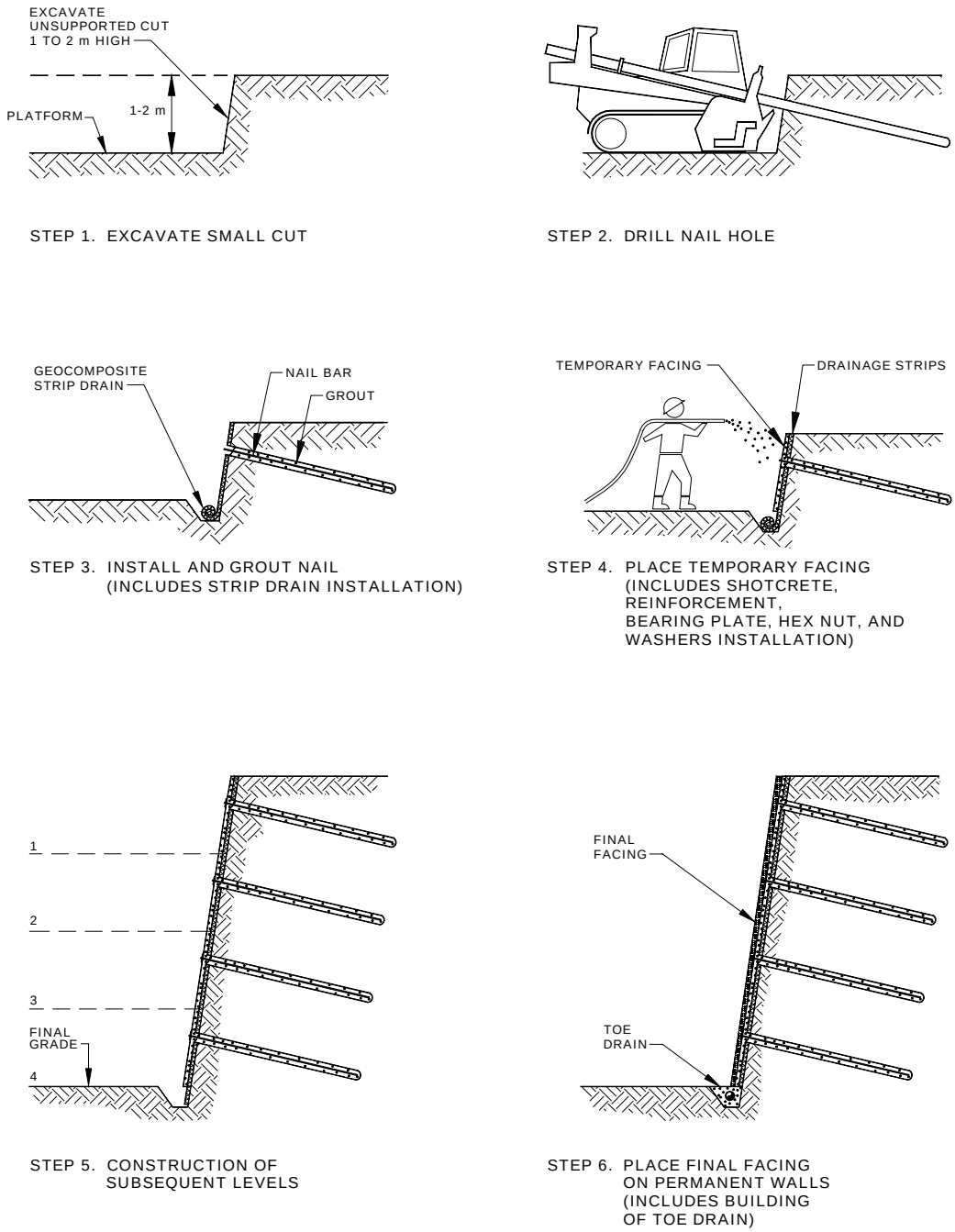
Step 3. *Nail Installation and Grouting*. Nail bars are placed in the pre-drilled hole. The bars are most commonly solid, although hollow steel nails can be also usedhave seen increased usage. Centralizers are placed around the nails prior to insertion to help maintain alignment within the hole and allow sufficient protective grout coverage over the nail bar. A grout pipe (tremie) is also inserted in the drillhole at this time. When corrosion protection requirements are high, corrugated plastic sheathing can also be

used to provide an additional level of corrosion protection. The drillhole is then filled with cement grout through the tremie pipe. The grout is commonly placed under gravity or low pressure. If hollow self-drilling bars are used (only as temporary structures), the drilling and grouting take place in one operation. Prior to Step 4 (facing placement), geocomposite drainage strips are installed on the excavation face approximately midway between each set of adjacent nails. The drainage strips are then unrolled to the next wall lift. The drainage strips extend to the bottom of the excavation where collected water is conveyed via a toe drain away from the soil nail wall.

Step 4. *Construction of Temporary Shotcrete Facing.* A temporary facing system is then constructed to support the open-cut soil section before the next lift of soil is excavated. The most typical temporary facing consists of a lightly reinforced shotcrete layer commonly 100 mm (4 in.) thick. The reinforcement typically consists of welded wire mesh (WWM), which is placed at approximately the middle of the facing thickness (see lower part of Figure 2.1). The length of the WWM must be such that it allows at least one full mesh cell to overlap with subsequent WWM panels. Following appropriate curing time for the temporary facing, a steel bearing plate is placed over the nail head protruding from the drillhole. The bar is then lightly pressed into the first layer of fresh shotcrete. A hex nut and washers are subsequently installed to secure the nail head against the bearing plate. The hex nut is tightened to a required minimum torque after the temporary facing has sufficiently cured. This usually requires a minimum of 24 hours. If required, testing of the installed nails to measure deflections (for comparison to a pre-specified criterion) and proof load capacities may be performed prior to proceeding with the next excavation lift. Before proceeding with subsequent excavation lifts, the shotcrete must have cured for at least 72 hours or have attained at least the specified 3-day compressive strength [typically 10.5 MPa (1,500 psi).

Step 5. *Construction of Subsequent Levels.* Steps 1 through 4 are repeated for the remaining excavation lifts. At each excavation lift, the vertical drainage strip is unrolled downward to the subsequent lift. A new panel of WWM is then placed overlapping at least one full mesh cell. The temporary shotcrete is continued with a cold joint with the previous shotcrete lift. At the bottom of the excavation, the drainage strip is tied to a collecting toe drain.

Step 6. *Construction of a Final, Permanent Facing.* After the bottom of the excavation is reached and nails are installed and load tested, a final facing may be constructed. Final facing may consist of cast-in-place (CIP) reinforced concrete, reinforced shotcrete, or prefabricated panels. The reinforcement of permanent facing is conventional concrete bars or WWM. When CIP concrete and shotcrete are used for the permanent facing, horizontal joints between excavation lifts are avoided to the maximum extent possible.



Modified after Porterfield et al. (1994).

Figure 2.2: Typical Soil Nail Wall Construction Sequence.

Variations of the steps described above may be necessary to accommodate additional preparation tasks or supplementary activities for specific project conditions. For example, shotcrete may be applied at each lift immediately after excavation and prior to nail hole drilling and installation,

particularly where face stability is a concern. A detailed description of the major activities related to the installation of soil nails is presented in Chapter 4.

2.4 APPLICATIONS OF SOIL NAIL WALLS

2.4.1 Introduction

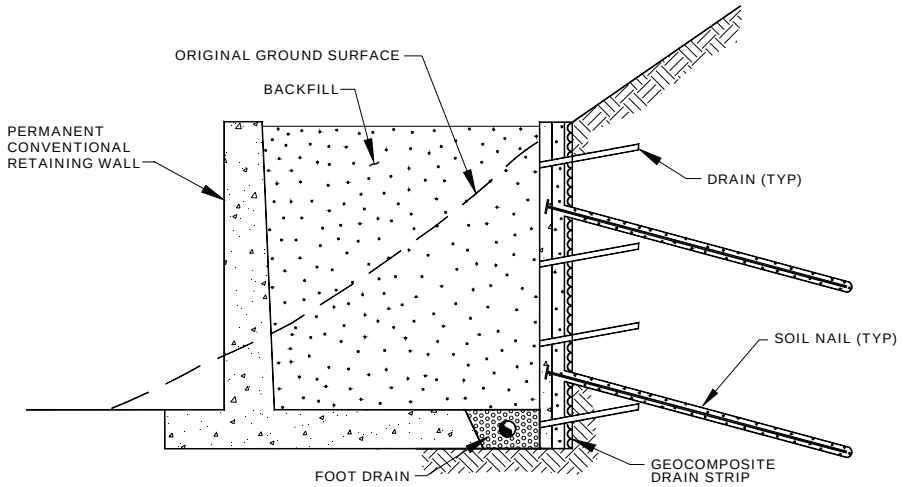
Soil nail walls are particularly well suited to excavation applications for ground conditions that require vertical or near-vertical cuts. They have been used successfully in highway cuts; end slope removal under existing bridge abutments during underpass widening; for the repair, stabilization, and reconstruction of existing retaining structures; and tunnel portals. Soil nail walls have been shown to be particularly well suited in the following temporary or permanent applications:

- roadway cut excavations;
- road widening under an existing bridge end;
- repair and reconstruction of existing retaining structures; and
- temporary or permanent excavations in an urban environment.

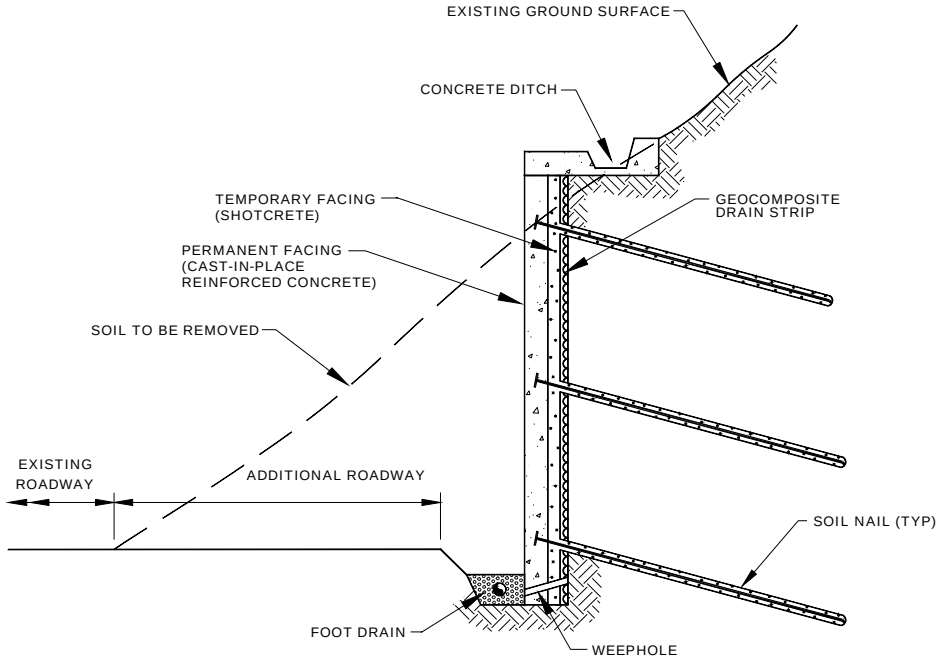
A discussion of important advantages and considerations related to the use of soil nail walls in some of the applications listed above is presented in the following three sections.

2.4.2 Retaining Structure in Cuts

Soil nail walls can be considered as retaining structures for any permanent or temporary vertical or near-vertical cut construction, as they add stabilizing resistance in situations where other retaining structures (e.g., anchor walls) are commonly used and where ground conditions are suitable. The relatively wide range of available facing systems allows for various aesthetic requirements to be addressed. In this application, soil nailing is attractive because it tends to minimize excavation, requires reasonable right-of-way (ROW) and clearing limits, and hence, minimizes environmental impacts within the transportation corridor. Soil nail walls are particularly applicable for uphill widening projects that must be constructed either within an existing ROW or in steep terrain. Figure 2.3 shows examples of the use of soil nail walls in temporary and permanent cut applications.



SOIL NAIL WALL FOR TEMPORARY SHORING



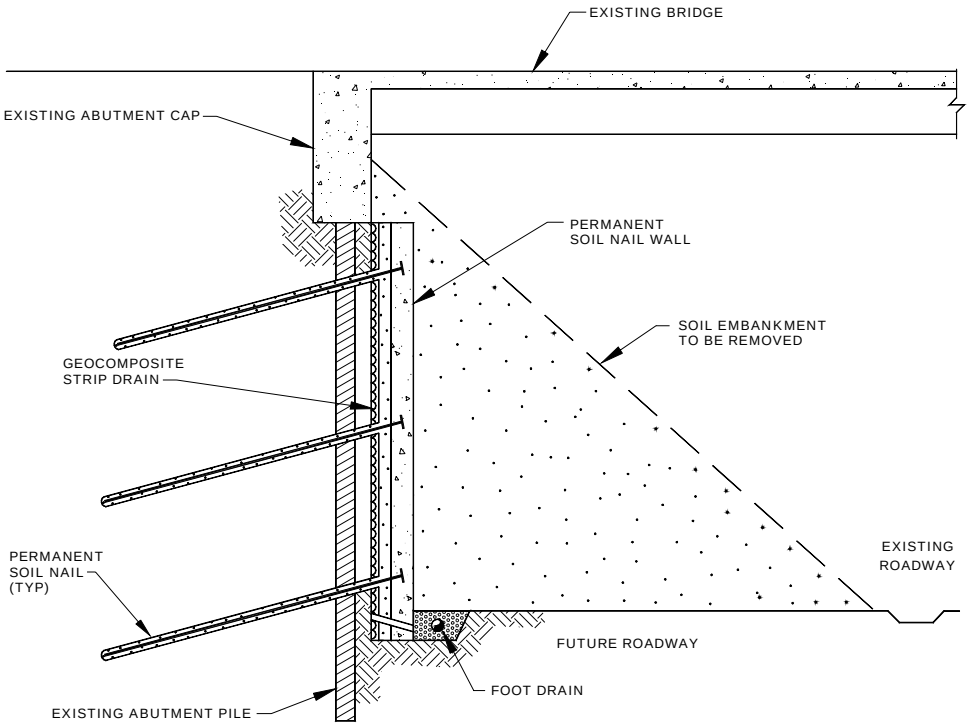
PERMANENT SOIL NAIL WALL IN
HIGHWAY WIDENING AND TRAFFIC LANE ADDITIONS

Source: Porterfield et al. (1994).

Figure 2.3: Soil Nail Walls for Temporary and Permanent Cut Slopes.

2.4.3 Retaining Structure Under Existing Bridge Abutments

Soil nail walls can be advantageous for underpass widening by removal of an existing bridge abutment end slope (see Figure 2.4) when compared to conventional ground anchor supported walls. Soil nail walls can be installed at comparable costs; however, the installation of soil nail walls does not require that bridge traffic be interrupted. If a ground anchor supported wall is used, soldier beams would have to be installed through the bridge deck because of limited overhead space under the bridge prior to excavating the end slope abutment. This operation results in the disruption of overpass traffic and accrues additional costs associated with lane closures and the procurement of large steel beams. Conversely, steel reinforcing bars used as soil nails are readily available. One disadvantage of the use of soil nail walls for end slope removal projects is that because the first level of soil nails is typically placed within 1 to 2 m (3 to 6 ft) from the top of the slope and because the nails are sloped downward, it is possible that the bridge girders will interfere with soil nail installation. This problem can usually be avoided by positioning the soil nails horizontally to be within the clear space between bridge girders.



Source: Porterfield et al. (1994).

Figure 2.4: Road Widening Under Existing Bridge.

2.4.4 Repair and Reconstruction of Existing Retaining Structures

Soil nails can be used to stabilize and/or strengthen existing, failing or distressed retaining structures. The soil nails are installed directly through the retaining structure. In these applications, which represent a departure from the original concept of soil nailing, the ground deformations required to mobilize the reinforcing resistance are not derived from removal of lateral support during excavation but from ongoing movements associated with the distressed structure. Relevant applications include:

- masonry or reinforced concrete retaining walls that exhibit structural deterioration or excessive deformations, often related to loose or weak backfill, or poor foundation performance; and
- mechanically stabilized earth (MSE) walls or crib walls that have deteriorated because of reinforcement corrosion or poor quality backfill.

2.5 FEASIBILITY EVALUATION OF SOIL NAIL WALLS

2.5.1 General

The feasibility evaluation of a soil nail wall should encompass technical and economical considerations and include: (1) an evaluation of the prevailing ground conditions; (2) an assessment of the advantages and disadvantages of a soil nail wall for the particular application being considered; (3) comparison with alternative systems (e.g., ground anchor wall systems); and (4) evaluation of costs. The following sections present a discussion of these aspects of the feasibility evaluation.

2.5.2 Evaluation of Ground Conditions for Soil Nail Walls

2.5.2.1 Introduction

Soil nail walls can be used for a wide range of soil types and conditions. Project experience has shown that certain favorable ground conditions make soil nailing cost effective over other techniques. Conversely, certain soil conditions can be considered marginal for soil nailing applications and may make the use of soil nails too costly when compared with other techniques. Soil nail walls can generally be constructed without complications in a mixed stratigraphy, as long as the individual layers of the soil profile consist of suitable materials. The following two sections present the soil conditions that are considered most and least suitable for soil nail walls. Intermediate soil conditions, for which the feasibility of soil nailing is not readily apparent, are also described.

2.5.2.2 Favorable Soil Conditions for Soil Nailing

Soil nail walls have been constructed successfully in various types of soils. Construction difficulties and long-term complications can generally be avoided when specific favorable soil conditions prevail. Soil nailing has proven economically attractive and technically feasible when:

- the soil in which the excavation is constructed is able to stand unsupported in a 1- to 2-m (3- to 6-ft) high vertical or nearly vertical cut for one to two days;

- all soil nails within a cross section are located above the groundwater table; and

- if the soil nails are below the groundwater table, the groundwater does not adversely affect the face of the excavation, the bond strength of the interface between the grout and the surrounding ground, or the long-term integrity of the soil nails (e.g., the chemical characteristics of the ground do not promote corrosion).

Although not an absolute requirement, it is advantageous that the ground conditions allow drillholes to be advanced without the use of drill casings and for the drillhole to be unsupported for a few hours until the nail bars are installed and the drillhole is grouted. Alternatively, soil nails have been installed with success using the hollow-stem drilling method in fully and temporarily cased drillholes. It is important to note that the selection of the drilling method is typically left to the discretion of the soil nail installation contractor.

Soil conditions are presumed to be favorable for the construction of soil nail walls when results from field tests indicate competent soils. The Standard Penetration Test (SPT, see next chapter) provides the SPT value, N, which can be used to preliminarily identify favorable soil conditions.

Based on the general criteria for favorable conditions noted above, the following ground types are generally considered well suited for soil nailing applications.

- *Stiff to hard fine-grained soils*. Fine-grained (or cohesive) soils may include stiff to hard clays, clayey silts, silty clays, sandy clays, sandy silts, and combinations thereof. Fine-grained soils can be tentatively classified as stiff if they have SPT N-values of at least 9 blows/300 mm (blows/ft). However, the consistency characterization of fine-grained soils should not rely solely on SPT N-values. Instead, the consistency (and thereby shear strength) characterization should be supplemented with other field and/or laboratory testing. To minimize potential long-term lateral displacements of the soil nail wall, fine-grained soils should be of relatively low plasticity [i.e., in general, plasticity index (PI) < 15].

- *Dense to very dense granular soils with some apparent cohesion.* These soils include sand and gravel with SPT N-values larger than 30 (Terzaghi et al., 1996), and with some fines (typically no more than about 10 to 15 percent of fines) or with weak natural cementation that provide cohesion. Capillary forces in moist fine sands may also provide an apparent cohesion. In general, the apparent cohesion for these soils should be greater than 5 kPa (100 psf) to assure reasonable stand-up times. To avoid excessive breakage of capillary forces and thereby significant reduction of this apparent cohesion, the movement of water toward the excavation face needs to be minimized including by redirecting surface water away from the excavation face.

- *Weathered rock with no weakness planes.* Weathered rock may provide a suitable supporting material for soil nails as long as weakness planes occurring in unfavorable orientations are not prevalent (e.g., weakness planes dipping into the excavation). It is also

desirable that the degree of weathering be approximately uniform throughout the rock so that only one drilling and installation method will be required. Conversely, a highly variable degree of rock weathering at a site may require changes in drilling equipment and/or installation techniques and thereby cause a costly and prolonged soil nail installation.

- *Glacial soils.* Glacial outwash and glacial till materials are typically suitable for soil nailing applications as these soils are typically dense, well-graded granular materials with a limited amount of fines.

2.5.2.3 Unfavorable or Difficult Soil Conditions for Soil Nailing

Examples of unfavorable soil types and ground conditions are provided below:

- *Dry, poorly graded cohesionless soils.* When poorly graded cohesionless soils are completely dry, contain no fines, or do not exhibit any natural cementation, apparent cohesion is not available. Therefore, the required vertical or nearly vertical cuts are difficult to achieve.

- *Soils with high groundwater*. Perched groundwater occurring behind the proposed soil nail wall will require significant drainage, which is necessary to stabilize the mass of soil in this location. Additionally, large amounts of groundwater can cause drillholes (particularly in loose granular soils) to collapse easily, thus requiring a costly soil nail installation. Excessive groundwater seeping out to the excavation face may cause significant difficulties for shotcrete application.

- *Soils with cobbles and boulders*. A large proportion of cobbles and boulders present in the soil may cause excessive difficulties for drilling and may lead to significant construction costs and delays. When only a few boulders and cobbles are present, modifying the drilling orientation from place to place may minimize or eliminate most of the difficult drilling. However, this approach has practical limitations when too many boulders are present.

- *Soft to very soft fine-grained soils.* These soils typically have SPT N-values less than 4 and are unfavorable for soil nailing because they develop relatively low bond strengths at the nail-grout-soil interface, thereby requiring unreasonably long nail lengths to develop the required resistance. Long-term deformations (creep) of the soils may be a concern for highly plastic clays. Concerns for creep deformations are generally less critical for temporary applications. As with any retaining system constructed in a top-down manner, the potential for instability at the bottom of the excavation is high in soft fine-grained soils. Additionally, high-plasticity soils may be expansive and may induce additional localized pressure on the facing due to swelling.

- *Organic soils*. Some organic soils such as organic silts, organic clays and peat typically exhibit very low shear strengths and thereby low bond strengths, which causes uneconomical nail lengths. While some organic soils can exhibit acceptable shear strengths, other organic soils like fibrous peat may be highly heterogeneous and highly

anisotropic. In this case, while the soil shear strength can be reasonable along some orientations, it may be significantly low along other orientations. These unfavorable orientations may have a detrimental impact on the wall stability and very long soil nails will be required. In addition, organic soils tend to be more corrosive than inorganic soils.

- *Highly corrosive soil (cinder, slag) or groundwater.* These conditions may lead to the need of providing expensive corrosion protection. These conditions are obviously more disadvantageous for permanent applications of soil nail walls.

- *Weathered rock with unfavorable weakness planes and karst.* Weathered rock with prevalent unfavorable weakness planes such as joints, fractures, shears, faults, bedding, schistosity, or cleavage may affect the drillhole stability and make grouting difficult. In addition, the presence of these discontinuities may cause the formation of potentially unstable blocks in the retained mass behind the wall during excavation. The marginal stability of blocks may rapidly deteriorate due to various factors, such as gouge in the joints, uplift and lateral hydrostatic pressures, and seepage forces. The stabilization of individual blocks may be necessary and can make this solution uneconomical when compared to conventional soil nails. In addition, grouting in rock with very large open joints or voids will be very difficult and/or expensive due to excessive grout loss. Grouting in karstic formations is not appropriate.

- *Loess.* When it is dry, loess may exhibit acceptable strengths that would allow economical installation of soil nails. However, when sizable amounts of water ingress behind the proposed soil nail wall, the structure of the loess may collapse and a significant loss of soil strength may take place. Therefore, the collapse potential upon wetting of these soils must be evaluated. Appropriate measures to avoid excess water migration to the soil nail area must be provided in loess exhibiting significant collapse potential. Additionally, considerably low soil shear strengths may arise for the wetted condition. In these cases, unusually long soil nail lengths may result in using conventional methods of nail installation. Regrouting (an atypical and more costly step) has been used to increase bond strengths in loess.

In addition to the difficulties described above, other aspects related to soil conditions must be considered when assessing the feasibility of soil nail walls:

- The prolonged exposure to ambient freezing temperatures may cause frost action in saturated, granular soils and silt; as a result, increased pressures will be applied to the temporary and permanent facings.

- Repeated freeze-and-thaw cycles taking place in the soil retained by the soil nail wall may reduce the bond strength at the soil nail grout-ground interface and the adhesion between the shotcrete and the soil. To minimize these detrimental effects, a suitable protection against frost penetration and an appropriate shotcrete mix must be provided.

- Granular soils that are very loose ($N \leq 4$) and loose ($4 < N \leq 10$) may undergo excessive settlement due to vibrations caused by construction equipment and traffic.

- Loose and very loose saturated granular soil can be susceptible to liquefaction in seismically exposed regions. Several ground modification techniques (typically with significant associated costs) may be utilized to densify granular soils and thereby minimize these damaging effects.

Despite the difficulties associated with unfavorable soil conditions described above, soil nail walls may still be built. It should be recognized that these wall systems would typically be more expensive to design and construct when compared to conventional walls in a more suitable soil. It is likely that significant extra effort and cost is needed in the design and construction of soil nail walls in these marginal conditions and that more strict long-term performance requirements might be necessary to allow soil nailing in such challenging conditions.

2.5.2.4 Intermediate Soil Conditions for Soil Nailing

There exists some soil conditions that are intermediate to the two conditions described previously. Although, the engineering properties are less favorable than those described in Section 2.5.2.2, soil nail walls have been installed successfully and cost-efficiently in certain intermediate soil conditions. Examples of intermediate soil conditions are presented below:

- *Engineered fill.* Soil nails can be installed in engineered fill if it is a mixture of well-graded granular material (approximately 90 percent of the mix or more) and fine-grained soil with low plasticity (typically, PI < 15).

- *Residual soils.* Residual soils (i.e., those soils created from the in-place weathering of the parent rock material) may be an acceptable material for soil nailing. Similarly, lateritic soil, a highly weathered tropical soil, may be acceptable. For these types of soil, specific consideration should be given to the soil spatial variability and its ability to drain.

2.5.3 Advantages of Soil Nail Walls

Soil nail walls exhibit numerous advantages when compared to ground anchors and alternative top-down construction techniques. Some of these advantages are described below:

Construction

- requires smaller ROW than ground anchors as soil nails are typically shorter;

- less disruptive to traffic and causes less environmental impact compared to other construction techniques;

- provides a less congested bottom of excavation, particularly when compared to braced excavations;

- there is no need to embed any structural element below the bottom of excavation as with soldier beams used in ground anchor walls;

- installation of soil nail walls is relatively rapid and uses typically less construction materials than ground anchor walls;

- easy adjustments of nail inclination and location can be made when obstructions (e.g., cobbles or boulders, piles or underground utilities) are encountered; on the other hand, the horizontal position of ground anchors is more difficult to modify almost making adjustments in the field costly;

- because significantly more soil nails are used than ground anchors, adjustments to the design layout of the soil nails are more easily accomplished in the field without compromising the level of safety;

- overhead construction requirements are smaller than those for ground anchor walls because soil nail walls do not require the installation of soldier beams; this is particularly important when construction occurs under a bridge;

- soil nailing is advantageous at sites with remote access because smaller equipment is generally needed; and

- the number of contractors with qualifications and experience in soil nail wall construction is increasing each year.

Performance

- soil nail walls are relatively flexible and can accommodate relatively large total and differential settlements;

- measured total deflections of soil nail walls are usually within tolerable limits; and

- soil nail walls have performed well during seismic events owing to overall system flexibility.

Cost

- soil nail walls are more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used;

- soil nail walls are typically equivalent in cost or more cost-effective than ground anchor walls when conventional soil nailing construction procedures are used; and

- shotcrete facing is typically less costly than the structural facing required for other wall systems.

2.5.4 Disadvantages of Soil Nail Walls

Some of the potential disadvantages of soil nail walls are listed below:

- soil nail walls may not be appropriate for applications where very strict deformation control is required for structures and utilities located behind the proposed wall, as the system requires some soil deformation to mobilize resistance; post tensioning of soil nails can overcome this shortcoming in most cases, but this step increases the project cost;

- the occurrence of utilities may place restrictions on the location, inclination, and length of soil nails in the upper rows;

- soil nail walls are not well-suited where large amounts of groundwater seeps into the excavation because of the requirement to maintain a temporary unsupported excavation face;

- permanent soil nail walls require permanent, underground easements; and

- construction of soil nail walls requires specialized and experienced contractors.

2.5.5 Comparison with Ground Anchor Wall Systems

To gain further insight into the soil nail wall concept, it is useful to compare the main features of a soil nail wall with those of a ground anchor wall, which is a commonly used top-to-bottom system for retaining wall construction. Detailed information on ground anchor walls can be found in Sabatini et al. (1999).

- Installation Equipment: Ground anchor walls require a two-step process for construction. First, vertical elements (typically soldier beams) are driven or pre-drilled to depths below the level of the final excavation using relatively large equipment. Second, ground anchors are installed and then post-tensioned to transfer loads behind the potential failure using equipment different than that used for anchor installation. Construction using different equipment may be a sizeable portion of both the project cost and schedule in ground anchor wall projects. Soldier beams and dedicated pre-tensioning equipment are not used in soil nail walls.

- Nail/Anchor Density: Under similar project conditions, the number of required soil nails per wall unit area is larger than the number of ground anchors per wall unit area. The use of more reinforcing elements in a soil nail wall adds a degree of redundancy that can contribute to the stability of a soil nail wall. Consequently, the failure of one reinforcing element will have a smaller effect on the stability of a soil nail wall than that of a ground anchor wall. Typically, only five percent of production soil nails are load tested, whereas all ground anchors are tested for acceptance.

- Load on Wall Facing: The density of soil nails implies that the facing in soil nail walls supports a smaller portion of the soil pressure due to a smaller tributary area compared to the facing in ground anchor walls, which supports a much greater soil pressure. This difference is more due to the dissimilar design approaches in the two systems rather than differences in the controlling load transfer mechanisms.

- Load Transfer: Soil nail transfer load along the entire length of the nails, whereas ground anchors are designed to transfer load only in the anchor zone behind the potential failure surface.

- Load Distribution: The resisting force provided by soil nails is variable along its entire length. In a ground anchor, one portion of the anchor length is unbonded while the

remaining portion is bonded. The load in a ground anchor is approximately constant in the unbonded length and variable in the bonded zone.

- Load Mobilization: Ground anchors develop tensile capacity through post-tensioning after anchor installation. Soil nails are passive and only develop tensile capacity by deformation of the surrounding soil.

- Stability of Excavation Bottom: In ground anchor walls, soldier beams are embedded to elevations below the bottom of the excavation. The shear resistance derived from the embedded portion of the soldier beams provides additional stability of the bottom of the excavation in ground anchor walls. This favorable effect is absent in soil nail systems.

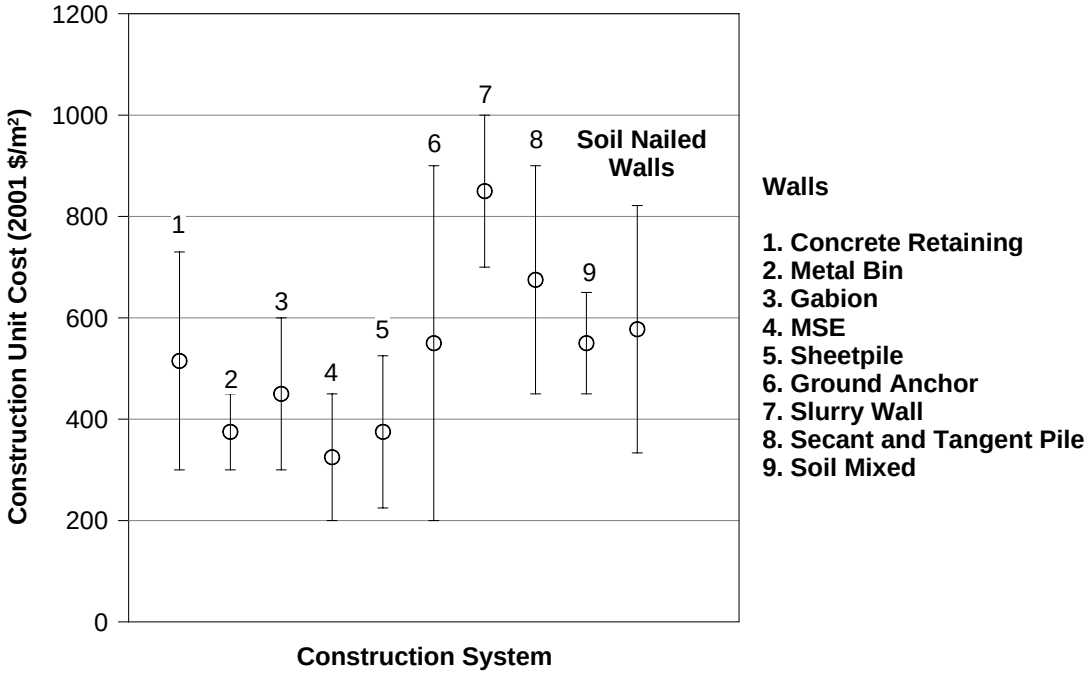
- Deflection: Field measurements in ground anchor walls indicate that the maximum wall lateral deflection is generally at midheight of the wall. In soil nail walls, the maximum lateral deflection takes place at the top of the wall. Also, maximum wall deformations are generally greater in soil nail walls than in ground anchor walls.

2.5.6 Construction Cost Evaluation

Costs for soil nail walls are a function of several factors, including ground conditions, site accessibility, wall size, facing type, corrosion protection requirements, temporary or permanent application, availability of contractors specialized in soil nailing and shotcrete, and regional conditions (e.g., seismic and frost susceptibility, etc.).

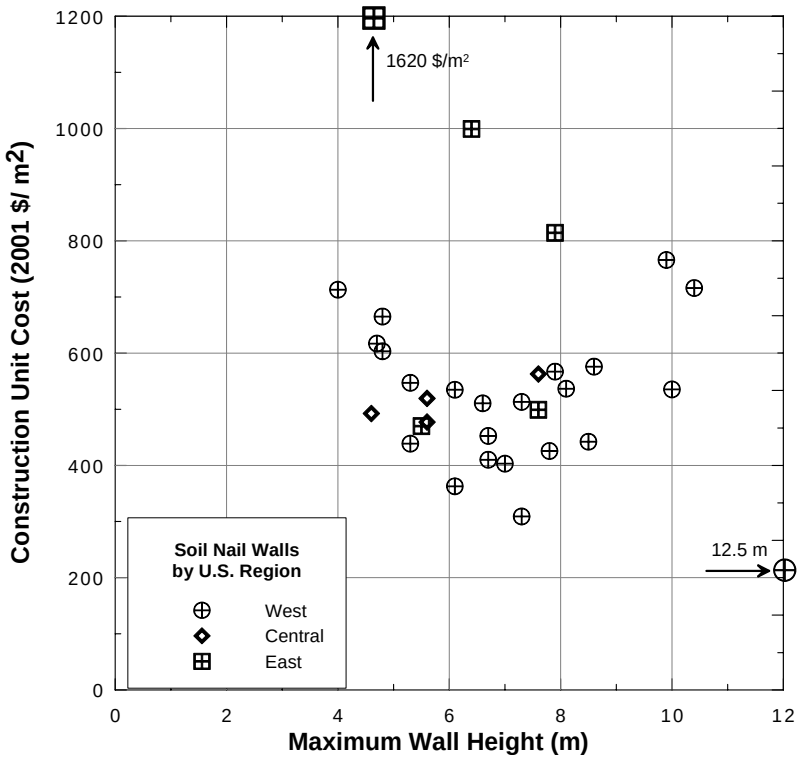
Typical cost ranges for other commonly used retaining structures are compared with those of soil nail walls in Figure 2.5. Experience on U.S. highway projects indicates that permanent soil nail walls can provide a 10 to 30 percent cost savings compared with similar retaining methods involving soil excavation. A major cost item for permanent walls is the facing. The placing of CIP or precast facings over an initial 100-mm (4-in.) thick shotcrete facing may comprise about 40 to 50 percent of the total cost of the wall.

Representative soil nail wall projects with a conventional shotcrete temporary facing and a permanent CIP facing were selected for the cost analysis presented in Figure 2.6. Cases where significant departures from the conventional shotcrete/CIP facing scheme or projects that experienced considerable difficulty during construction were discarded from the dataset presented herein. Figure 2.6 presents the results of the cost analysis for various soil nail wall projects and provides the variation of the unit cost of soil nailing construction (in 2001 dollars) as a function of the wall height. These data indicate that the unit cost is more or less uniform for all wall heights, with a tendency to be most economical for wall heights between 6 and 8 m (18 to 24 ft). The selected data do not seem to indicate a marked difference in cost by U.S. region.



Data from Sabatini et al. (1997).

Figure 2.5: Comparison of Construction Cost Data for Various Systems.



Data from Byrne et al. (1998).

Figure 2.6: Construction Cost Data for Soil Nail Walls.

CHAPTER 3 SITE INVESTIGATION, LABORATORY TESTING, AND RECOMMENDATIONS

3.1 INTRODUCTION

The purpose of this chapter is to describe site investigation and laboratory testing techniques and to provide recommendations on the soil parameters used for the design of soil nail walls. Recommendations for bond strength for nails installed in various soil types and construction conditions are also presented in this chapter.

Site investigation and laboratory testing are two of the most critical aspects in any geotechnical engineering project and have a direct effect on the technical feasibility and cost-effectiveness of a soil nail wall project. In general, the extent and level of detail of the site investigation and laboratory testing program depend on the nature and magnitude of the overall project, topography, geology, environmental limitations, type of application, consequences of a potential failure, project service life, and other project constraints.

Typical engineering properties provided by site investigation and laboratory testing for the design of soil nail walls include soil classification, unit weight, shear strength, and compressibility. Site investigation and laboratory testing can also provide information to establish the corrosion potential of the soil, which is a critical aspect of the long-term integrity of steel reinforcement in permanent soil nail walls. General information regarding site investigation and laboratory testing of soils and rocks is provided in this section. More comprehensive discussion and description is provided in the recent FHWA Geotechnical Engineering Circular No. 5 (GEC 5) titled "*Evaluation of Soil and Rock Properties*" (Sabatini et al., 2002). The reader is encouraged to refer to GEC 5 for a detailed explanation of investigation tools and laboratory testing techniques.

The next sections describe the typical main activities related to the site investigation that can be applied in a soil nail project.

3.2 REVIEW OF EXISTING GEOTECHNICAL INFORMATION

Prior to performing a site investigation, a review of existing regional, site, and subsurface information is a valuable first step to any geotechnical project. Sources of information may include:

- topographic maps, site plans, geologic maps, air photos, construction plans, and surveys;
- geological data, such as lithology of nearby rock outcrops, landforms, erosion patterns, subsidence, and landslides;
- existing geotechnical reports prepared for the site or nearby areas;
- data on the performance of existing engineered structures (including soil nail walls or comparable systems) in the area;

- data on bodies of water or groundwater levels near the project site; and
- data on seismic aspects, such as ground motion, liquefaction potential, and site amplification.

While historic information may be helpful, care must be exercised in assessing the quality, validity, and applicability of this information in a new project. The designer must proceed cautiously when extrapolating the subsurface soil conditions encountered in the area but at some distance from the project site.

3.3 SITE RECONNAISSANCE

Field reconnaissance consists of visual inspection of the site and collection of pertinent data. The objectives of a field reconnaissance are to observe and collect information on site features that may have an effect on the design, construction, and maintenance (if necessary) of the proposed wall. In general, the following information needs to be collected during the field reconnaissance:

- site accessibility;
- traffic conditions and control during investigation and construction;
- overhead space limitations;
- drainage and erosion patterns;
- nature of above-ground structures;
- identification of underground utilities;
- response of nearby cuts, slopes, and excavations; and
- evidence of surface settlement.

The following additional information, although generally applicable to most wall types, is especially pertinent when considering soil nail walls as an alternative:

- evidence of corrosion in nearby structures;
- evidence of areas with soil and groundwater conditions that are unfavorable for soil nail walls; and
- ROW and easements required for the nail installation.

After a thorough review of the existing information and a site reconnaissance are completed, the designer can identify gaps of information, delineate additional project requirements, and define the necessary subsurface investigation in consideration of existing information and new requirements. Some of the site investigation activities are described in the following section.

3.4 SUBSURFACE INVESTIGATION

3.4.1 General

The objectives of a subsurface investigation are to ascertain local subsurface conditions and delineate the lateral and spatial distribution of these conditions at a project site. Subsurface investigations typically consist of: (1) in situ testing of soil/rock properties, (2) retrieval of representative samples of soil/rock for visual classification and/or laboratory testing, (3) characterization of the stratigraphy, and (4) identification and observation of groundwater location. The soil/rock properties of interest to be determined from in situ and laboratory testing include classification, index parameters, strength, compressibility, and corrosion potential. In addition, determining the location and nature of the groundwater is important in soil nail wall projects because these systems are difficult to construct and more costly when the groundwater is high.

Subsurface investigations should be performed in accordance with the American Association of State Highway and Transportation Officials (AASHTO) "Subsurface Investigation Manual" (AASHTO, 1988), the American Society of Testing Materials (ASTM) standards (ASTM, 2002), or the standards of the local agency, if available. Additional information on subsurface investigation programs can be obtained from AASHTO (1988), Arman et al. (1997) and Sabatini et al. (2002). The following sections highlight the main aspects of subsurface investigations commonly used in soil nail wall projects.

3.4.2 Field Testing and Sampling

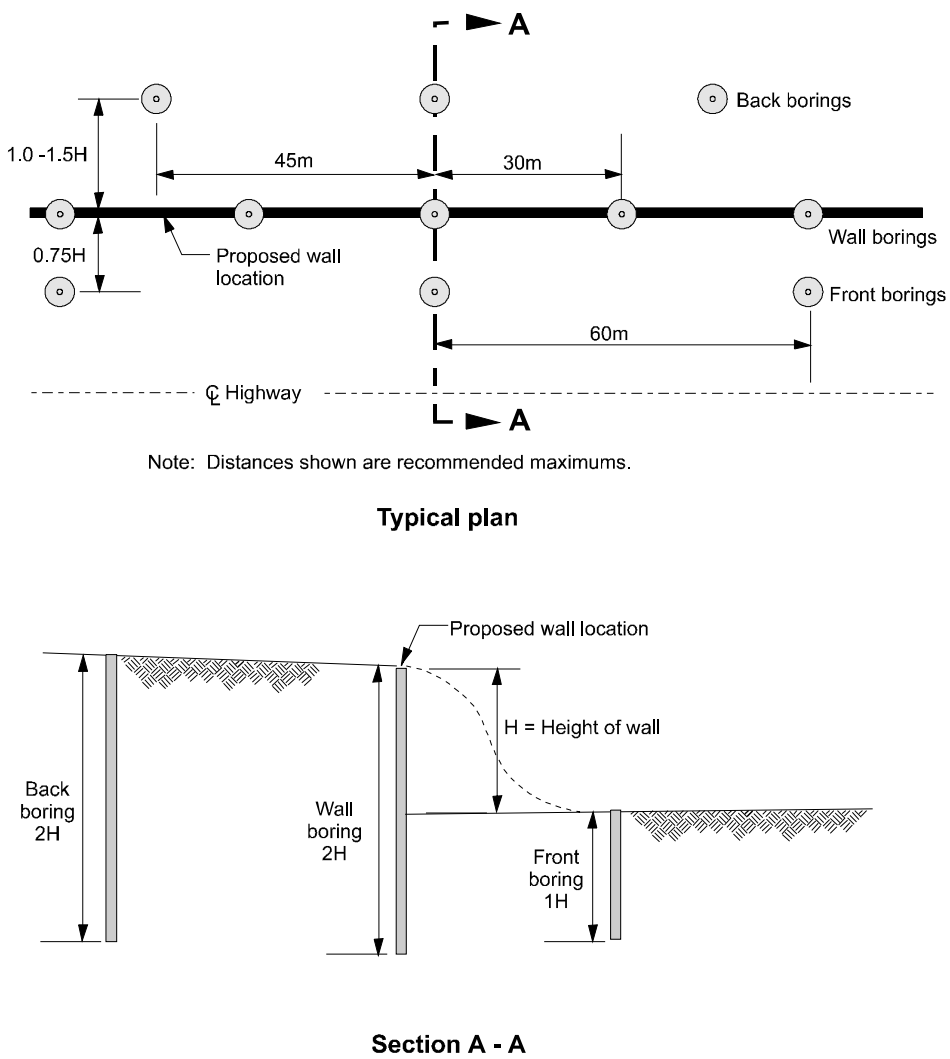
Borings

Field testing and sampling for the design of soil nail walls is commonly performed with exploratory borings. Borings serve different purposes and provide: (1) SPT N-values (described in a subsequent section) to classify soil and delineate the stratigraphy, (2) both disturbed and undisturbed soil samples, and (3) observations of groundwater.

The type, number, location, and depth of borings are dictated, to a large extent, by the project stage (i.e., feasibility study, preliminary, or final design), availability of existing geotechnical data, variability of subsurface conditions, and other project constraints. Figure 3.1 can be used as a preliminary guide to help designers plan the number, location, and frequency of borings for soil nail wall applications. For soil nail walls more than 30 m (100 ft) long, borings should be spaced between 30 to 60 m (100 to 200 ft) along the proposed centerline of the wall. For walls less than 30 m (100 ft) long, at least one boring is necessary along the proposed centerline of the wall.

Borings are also necessary in front and behind the proposed wall. Borings behind the wall should be located within a distance up to 1 to 1.5 times the height of the wall behind the wall and should be spaced up to 45 m (150 ft) along the wall alignment. If the ground behind the proposed wall is sloping, the potentially sliding mass behind the wall is expected to be larger than for horizontal ground. Therefore, borings behind the proposed wall should be located farther behind the wall, up to approximately 1.5 to 2 times the wall height. Borings in front of the wall should be located within a distance up to 0.75 times the wall height in front of the wall and should be spaced up to 60 m (200 ft) along the wall alignment.

The depth of borings must be selected based on the controlling mechanisms related to the stability of the proposed wall and on subsurface aspects that may affect the future performance of the wall. Borings should extend at least one full wall height below the bottom of the excavation (Figure 3.1). Borings should be deeper when highly compressible soils (i.e., soft to very soft fine-grained soils, organic silt, and peat) occur at the site behind or under the proposed soil nail wall. The required boring depths for soil nail wall projects may be greater if deep loose, saturated, cohesionless soils occur behind and under the proposed soil nail wall and the seismic risk at the site require that the liquefaction potential be evaluated. The subsurface investigation depths may need to be deep at proposed sites of soil nail walls where seismic amplification is of concern, particularly in deep, soft soils. If rock is encountered within the selected depth, a core at least 3 m (10 ft) long retrieved in two 1.5 m (5 ft) long runs should be obtained to inspect the nature of the rock and its discontinuities.



From Cheney (1988) and Sabatini et al. (1999).

Figure 3.1: Preliminary Geotechnical Boring Layout for Soil Nail Walls.

Common Field Procedures

Table 3.1 summarizes common geotechnical procedures and field tests that can be used in site investigations of soil nail wall projects. The table lists field procedures and field tests and presents related ASTM standards and FHWA references, the soil type applicability, and a brief description of the information and properties derived from these field activities.

Table 3.1: Common Geotechnical Field Procedures and Tests.

	Activity	Standard(1) /FHWA Reference	Most suitable for	Not suitable for	Obtained from field activity
Field Procedure	Preservation and Transportation of Soil Samples	ASTM D4220-95	All	NA	Representative samples
	Thin-Walled Tube Sampling	ASTM D1587-00	Clays, Silts	Sands, Gravel	Undisturbed samples
	Subsurface Explorations (Soil and Rock)	ASTM D5434-97	All	NA	Various
Field Test	Standard Penetration Test (SPT)	ASTM D1586-99 ASTM D6066-96e1(5)	Sand, Silt	(2)	Stratigraphy, SPT N-values relative density, groundwater, samples
	Cone Penetration Test (CPT)	ASTM D5778-95, Briaud (1992)	Sand, Silt, and Clay	Gravel, bouldery soil	Continuous stratigraphy, soil type, strength, relative density, K_0 (3), pore pressures, no sample
	Field Vane Shear Test (VST)	ASTM D2573-94	Soft to Medium Clay	Sand and Gravel	Undrained shear strength
	Pressuremeter Test (PMT)	ASTM D4719-00, Briaud (1989)	Soft Rock, Dense Sand, Non-Sensitive Clay, Gravel, Till	Soft Clays, Loose Silts and Sands	Soil type, strength, K_0 (3), OCR (4), compressibility, soil modulus, no sample
	Flat Plate Dilatometer Test (DMT)	ASTM D6635-01, Briaud and Miran (1992)	Sand and Clay	Gravel	Soil type, K_0, OCR, undrained shear strength, soil modulus, no sample

Notes: (1) Individual ASTM standards can be found in ASTM (2002). Arman et al. (1997) and Sabatini et al. (2002) present general discussions on these field procedures.
(2) SPT can be used with limitations in clays and gravels.
(3) K_o is the at-rest earth lateral pressure coefficient.
(4) OCR is the overconsolidation ratio.
(5) ASTM D6066-96e1 for the use of SPT in liquefaction resistance evaluation.

Standard Penetration Test

The Standard Penetration Test (SPT) is by far the most popular technique for assessing site conditions. In the case of soil nailing projects, the SPT has been the most widely used exploratory

technique because most soil nail walls in the United States have been built in cohesionless soils, for which the SPT generally provides meaningful results.

The SPT provides the SPT N-value, which is the measured number of blows, N_{meas}, required to drive a standard split-spoon sampler a distance of 300 mm (12 in.) at the bottom of boreholes. SPT N-values can be used in the feasibility phase of the design of a soil nail wall, as described in the previous chapter. Several correlations between SPT N-values and engineering properties have been developed over the past several decades.

The SPT is also used to obtain disturbed samples from the subsurface, typically spaced at vertical intervals of 1.5 and 3 m (5 and 10 ft). In layers with loose or soft soil, or when other features of interest are encountered (e.g., soil lenses and highly inhomogeneous conditions), sampling should be continuous. The SPT provides a good measurement of the relative density of cohesionless soils (Table 3.2). With limitations, the SPT can also provide an estimate of the consistency of fine-grained soils (Table 3.3).

Table 3.2: Cohesionless Soil Density Description Based on SPT N-Values.

(Source: Terzaghi et al., 1996)

Relative Density	SPT N_{meas} (blows/300 mm or blows/ft)
Very loose	0–4
Loose	5–10
Medium Dense	11–30
Dense	31–50
Very Dense	>51

Table 3.3: Fine-Grained Soil Consistency Description Based on SPT N-Values.

(Source: Terzaghi et al., 1996)

Consistency	SPT N_{meas} (blows/300 mm or blows/ft)
Very Soft	0–1
Soft	2–4
Medium Stiff	5–8
Stiff	9–15
Very Stiff	16–30
Hard	31–60
Very Hard	>61

A number of correction factors are applied to the measured blowcount, N_{meas}, to account for numerous factors related to energy inefficiencies and procedural variations. A corrected value, N_{60},

is obtained for an energy efficiency of 60 percent (typical in the U.S. engineering practice). N_{60} is defined as:

$$N_{60} = C_E\ C_B\ C_S\ C_R\ N_{meas} \qquad \text{(Equation 3.1)}$$

where the correction factors are (C_E) for energy correction efficiency, (C_B) for borehole diameter, (C_S) for sampling method, and (C_R) for rod length. Sabatini et al. (2002) presents a summary of these correction factors.

The corrected value, N_{60}, is commonly normalized to a reference effective overburden pressure to account for the increase of N-values with increasing overburden in a homogeneous material. The reference effective overburden pressure is the normal atmospheric pressure, P_a, which is equal to 1 atmosphere (atm) or approximately 100 kPa [approximately 1 ton per square foot (tsf)]. The normalized corrected blowcount is referred to as $(N_1)_{60}$ and is defined as:

$$(N_1)_{60} = C_N\ N_{60} \qquad \text{(Equation 3.2)}$$

where C_N is a normalization factor for effective overburden pressure and is calculated as:

$$C_N = (P_a/\sigma'_{vo})^n \qquad \text{(Equation 3.3)}$$

where (σ'_{vo}) is the effective vertical pressure at the SPT test depth at the time of testing (in the same units as P_a) and (n) is an exponent typically equal to 1 in clays and 0.5 in sands (see Sabatini et al., 2002, for references on C_N).

Some correlations based on SPT-values provide estimates of shear strength parameters for both cohesionless and fine-grained soils. These correlations are presented in a subsequent section.

Sampling

Samples obtained with the SPT sampler are disturbed and only adequate for soil classification and some laboratory tests, including particle gradation (sieve analysis), fines content, natural moisture content, Atterberg limits, specific gravity of solids, organic contents, and unconfined compressive strength tests. Grab samples obtained from cuttings in borings, test pits, and test cuts can also be used for soil classification and laboratory determination of index parameters, as long as they are sufficiently representative and the in situ moisture content was preserved during sampling and transportation.

SPT samples should not be used for strength or compressibility testing. Excessive disturbance is caused to soil samples because the SPT sampler has a large wall thickness/diameter ratio and the sample tends to be extruded through the sampler lower end (shoe) and into the sampler. As the shear strength and compressibility of fine-grained soils are heavily affected by sample disturbance, samples obtained with the SPT standard split-spoon sampler are unsuitable for laboratory testing of shear strength and compressibility of fine-grained soils. Undisturbed thin-walled samplers, including the Shelby tube sampler with an outer diameter (OD) of 76 mm (3 in.), should be used instead to obtain samples of fine-grained soil for laboratory testing of shear strength and consolidation.

Alternative Field Testing Techniques

As mentioned above, the SPT is not suitable for providing in situ estimates of the consistency and the shear strength of fine-grained soils. More appropriate in situ tests should be used for estimating the undrained shear strength (S_u) of fine-grained soils directly from the field. Some of these in situ tests include the cone penetration test (CPT), the field vane shear test (VST), the pressuremeter test (PMT), and the flat plate dilatometer test (DMT).

The CPT, also known as the piezocone penetration test, has become a common field investigation technique. The CPT is a valuable investigation tool that allows rapid and cost-effective development of subsurface soil profiles. As the CPT-based soil profiles are continuous, this technique permits the identification of thin soil layers that would be otherwise difficult to detect within a relatively homogeneous soil mass. This capability may prove particularly useful when investigating the presence of thin layers of weak soil that may prompt instability behind the proposed soil nail wall. In general, CPT is more cost-effective and faster than conventional SPT. The major disadvantage of this technique is that no sample is recovered. Additionally, CPT sounding cannot be performed in gravelly or bouldery soil.

For some large projects, the phased use of CPT and conventional borings is attractive because it provides comparatively more geotechnical information at costs that are comparable than with conventional borings alone. In the first phase, the CPT soundings allow the rapid depiction of the soil stratigraphy and early identification of layers with potential deficiencies (e.g., low strength or high compressibility) that may have an impact in the design. An initial CPT- based stratigraphy can help determine the location of zones where undisturbed soil samples should be obtained. In the second phase, conventional borings can be used and samples are obtained only at the depths of interest. Using this two-phase investigation strategy, sampling can be optimized and the number of samples can be reduced.

The VST is commonly used in conjunction with conventional soil borings to obtain the in situ undrained shear strength of fine-grained soils. The advantage of the VST is that it can provide a direct, in situ estimate of S_u in fine-grained soils. The PMT and the DMT are also available, but their use is not as widespread. For more comprehensive discussion, refer to Sabatini et al. (2002).

Test Pits

In soil nail wall applications, the use of test pits (in flat areas or test cuts in sloping ground) can be particularly beneficial. Test pits are relatively inexpensive and can help assess whether an excavation face will stand unsupported and define the feasibility of soil nailing at a site. Test pits should be approximately 6- to 8-m (20- to 25-ft) long and 2- to 2.5-m (6- to 8-ft) deep and should be excavated parallel to and in front of the proposed wall face. To evaluate the stand-up time for the excavation, the test pit should be left open for 3 to 4 days. Daily inspection of the excavated test pit is recommended.

3.4.3 Soil Stratigraphy

Once sufficient information is available from borings and field tests, it is important to develop the stratigraphy and to identify any significant spatial variability of subsurface conditions that may

affect the design and construction of a soil nail wall. Developing the site stratigraphy is critical for soil nail walls because the nature, extent, and distribution of the various layers dictates the type of drilling equipment and methods, control the size of the potential sliding soil mass behind the wall, and have an impact on the soil nail lengths. The identification of varying subsurface conditions on plan view is particularly important in long walls, where soil conditions are more likely to vary considerably. Soil stratigraphy is first assessed via visual logging or in situ testing results during the site investigation and subsequently is corroborated or adjusted from laboratory testing results.

The location of the soil-bedrock contact must be identified if the soil nails will be partially or totally embedded in underlying rock. The dissimilar subsurface conditions above and below this contact will have an impact on the suitability of the drilling methodology, the drilling equipment, the construction costs, and the nail lengths. Soil nails that are partially or totally embedded in rock are expected to be shorter than those embedded entirely in soil due to the higher bond strengths that tend to develop in rock. If soil nails are to be installed in weathered rock, layers with varying degrees of weathering and/or planes of weakness should be identified.

3.4.4 Groundwater

The occurrence of water (under non saturated or saturated conditions) in the soil may affect various aspects of the design and long-term performance of a soil nail wall. These aspects include stability of temporarily unsupported cuts, soil strength and bond strength, corrosion potential, pressure on the facing, drillhole stability, grouting procedures, drainage, and other construction considerations. Therefore, the presence of a groundwater table and/or perched groundwater zones must be identified during the subsurface investigation program.

Groundwater depth should be obtained from borings during drilling and should be monitored for at least 24 hours after drilling. If drilling fluid is used during boring advancement, it may not be possible to locate the groundwater in borings. For soils exhibiting relatively high fines content, the groundwater levels obtained during drilling do not commonly represent stabilized levels of the groundwater table, as the observed levels of water are most likely affected by the relatively low permeability of the surrounding soil. In these soils, it is a good practice to measure the groundwater level a few times over the course of a few hours or days to allow groundwater to reach its equilibrium level. In soils with very low permeability, more extended periods of time, up to several weeks or months, may be necessary for the groundwater level to stabilize. For these cases, it is valuable to convert some of the exploratory borings into piezometers. It is desirable to obtain (or estimate) the seasonal (high and low) groundwater levels from piezometers or other sources (e.g., existing nearby wells).

Underestimating grossly the elevation of groundwater during a field investigation can have serious consequences for any earth retaining system. This is particularly true for soil nail walls because these systems are not particularly suited to high groundwater conditions, as discussed in the previous chapter. When the actual groundwater level is higher than previously estimated, field modifications may be required. In extreme cases, soil nailing may be found altogether to be impractical and/or uneconomical.

3.5 LABORATORY SOIL TESTING

Laboratory testing of soil samples recovered during the site investigation is performed to produce soil classification, index properties, unit weight, strength, and compressibility. Table 3.4 presents laboratory tests commonly used to develop index parameters and other engineering properties of soils that may be necessary for the design of a soil nail wall. Table 3.4 also lists relevant ASTM and AASHTO testing standards. Additionally, Table 3.4 lists other tests, such as those for evaluating frost heave, thaw susceptibility, collapse potential, and swelling potential, which may exist for some soil nail wall projects.

Table 3.4: Common Procedures and Laboratory Tests for Soils.

PROCEDURE	TEST NAME	STANDARD		APPLICABILITY
		ASTM (1)	AASHTO (2)	
Classification	Visual and Manual Description and Identification of Soils	D2488-00	-	All soils
	Classification of Soils according to USCS (3)	D2487-00	M145	All soils
Index Parameters	Particle-Size Analysis (with sieves)	D422-63 (1998)	T88	Granular soils
	Soil Fraction finer than No. 200 (75-µm) Sieve	D1140-00	T11	Fine-grained and granular materials boundary
	Moisture Content	D2216-98	T265	All soils
	Atterberg Limits	D4318-00	T89, T90	Fine-grained soil
	Organic Contents	D2974-00	T194	Fine-grained soil fraction
	Specific Gravity of Soil Solids	D854-00	T100	All soils
Strength	Unconfined Compressive Strength (UC)	D2166-00	T208	Fine-grained soil
	Unconsolidated-Undrained Triaxial Compression (UU)	D2850-95 (1999)	T296	Fine-grained soil
	Consolidated Undrained Triaxial Compression (CU)	D4767-95	T234	Fine-grained soil
	Direct Shear (Consolidated)	D3080-98	T236	Sands and Fine-grained soils
Hydraulic Conductivity	Permeability (Constant Head)	D2434-68 (2000)	T215	Granular soils
Compressibility	One-Dimensional Consolidation	D2435-96	T216	Fine-grained soil
	One-Dimensional Consolidation (Controlled-Strain Loading)	D4186-89 e1 (1998)	-	Fine-grained soil
Other	Frost Heave and Thaw Weakening Susceptibility	D5918-96 (2001)	-	Silts
	Collapse Potential	D5333-92 (1996)	-	Loess, silt
	Swelling Potential	D4546-96	T258	Fine-grained soil

Notes: (1) Individual ASTM standards can be found in ASTM (2002).
(2) Individual AASHTO standards can be found in AASTHO (1992).
(3) USCS: Unified Soil Classification System.

3.6 SELECTION OF SOIL PARAMETERS FOR DESIGN

3.6.1 Introduction

The purpose of this section is to present a brief discussion of the methods used to obtain soil parameters and provide guidance for the selection of these parameters in soil nail wall design. The primary design parameters for soil nail walls are discussed in the next few sections.

3.6.2 Soil Classification and Index Properties

For most soil nail wall projects, the following soil basic properties should be determined from the field investigation and laboratory testing:

1. soil classification (all soils);
2. sieve analyses (cohesionless soils);
3. fines content (mixed fine- and coarse-grained soils);
4. natural moisture (mostly fine-grained soils);
5. Atterberg limits (fine-grained soils); and
6. organic content (fine-grained soils).

A correct soil classification and typifying is important because the anticipated soil response is commonly associated with typified soil types. As discussed in the previous chapter, although soil nails can be installed in a relatively wide range of soils, soil nail walls are more economically competitive in certain select soil types. Soils should be classified according to the Unified Soil Classification System (USCS), which requires that the gradation and Atterberg limits be determined.

Sieve analyses in granular soils, along with fines content determinations, can help determine whether soil conditions are favorable (e.g., well graded cohesionless soils with little fines) or not.

The natural in situ moisture content determined mostly in fine-grained soils can help detect certain unfavorable conditions. For example, unusually high moisture contents in fine-grained soils [greater than 50 or moisture contents on the order of the liquid limit (LL)] may suggest problematic soil conditions including low shear strength, high compressibility, potentially high creep potential, and higher corrosion potential. Moisture contents less than about 2 percent in granular soils (with little or no fines) may indicate the inability of vertical cuts to remain unsupported.

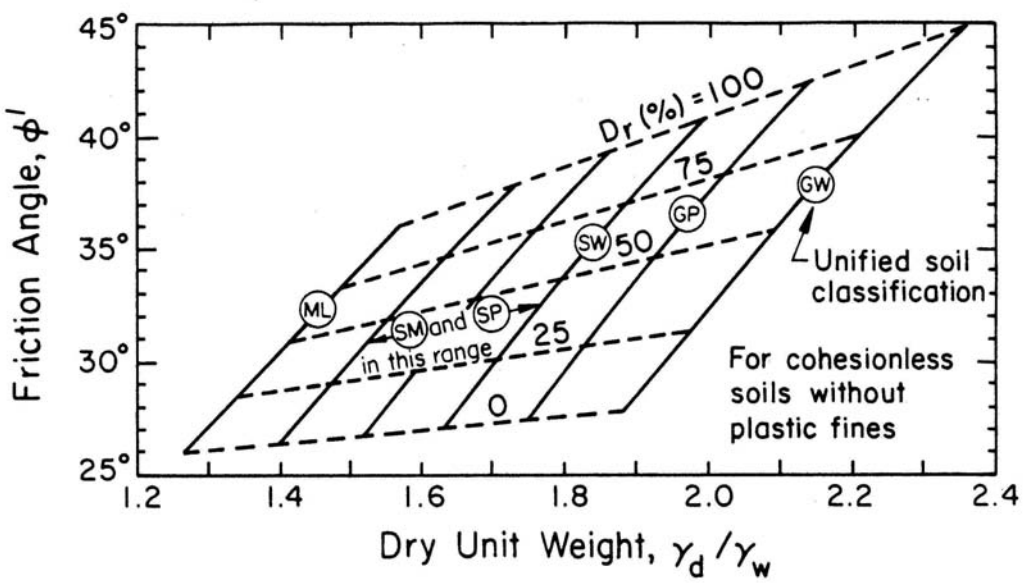
Atterberg limits of fine-grained soils must be used to classify fine-grained soils and help assess the potential for creep deformations of the soil behind the proposed wall. These index test results may be used to estimate the shear strength of fine-grained soils, using appropriate correlations as presented in a subsequent section.

If the presence of organic materials is suspected from review of existing information or from field observations (e.g., odor, color, and/or texture of soil samples), organic contents tests (i.e., loss of

organics mass on-ignition) must be performed in the laboratory. In general, organic soils with high natural soil moisture have a higher corrosion potential than inorganic soils.

3.6.3 Soil Unit Weight

An important soil parameter used in the analysis of soil nail walls is the unit weight because the destabilizing forces acting behind a soil nail wall are directly affected by the unit weight. The unit weight of granular soils and some fine-grained soils can be estimated from soil descriptions in conjunction with descriptions of the relative density, D_r, (Figure 3.2) or other correlations (e.g., Kulhawy and Maine, 1990). In Figure 3.2, γ_d/γ_w is the ratio of the dry unit weight of the soil to the unit weight of water. For saturated soil, the in situ soil moisture content, w_n, must be incorporated to calculate the saturated unit weight, γ_{sat} [i.e., $\gamma_{sat} = \gamma_d (1+ w_n)$]. The unit weight of cohesionless soils can be estimated from correlations with the SPT N-value. Laboratory testing of in situ unit weight of granular soils is not practical because the in situ soil density cannot be easily reproduced in the lab due to sample disturbance. The unit weight of fine-grained soils may be determined in the lab from undisturbed samples (e.g., from thin-walled Shelby tubes).



Modified after U.S. Navy (1982), Kulhawy and Mayne (1990).

Figure 3.2: Correlation of Effective Friction Angle as a function of Soil Classification, Relative Density, and Unit Weight.

3.6.4 Shear Strength

Cohesionless Soils

The shear strength of cohesionless soils is represented by a drained, effective angle of internal friction, ϕ'. The value of the friction angle is commonly estimated from correlations with results

from field tests (e.g., SPT and CPT). Values of the friction angle as a function of parameters obtained from SPT and CPT are included in Table 3.5.

Figure 3.2 presented estimates of the friction angle based on USCS soil classification. Figure 3.3(a) presents an estimate of the friction angle based on SPT N-values. The bar at the right hand side of Figure 3.3 provides an estimate of the soil friction angle as a function of the soil density descriptions presented in Table 3.5, column (a). Values of the friction angle obtained from uncorrected SPT N-values in Table 3.5 and Figure 3.3(a) are approximations over a wide range of conditions and can be used conservatively as preliminary values. However, these approximations do not take into account the increase of SPT N-values with increasing overburden that is commonly observed in homogeneous cohesionless soils. Figure 3.3(b) considers this effect and provides values of the friction angle based on the SPT N-value and as a function of the normalized in situ effective overburden, σ'_{vo}/P_a, where σ'_{vo} is the in situ effective overburden pressure, and P_a is the atmospheric pressure. Friction angles obtained from Figure 3.3(b) are more accurate than those presented in Figure 3.3(a) and should be used when sufficient information is available. Note that when $\sigma'_{vo}/P_a = 1$, the N-value is equivalent to the normalized corrected blowcount defined as $(N_1)_{60}$ in Section 3.4.2 (i.e., the equivalent N-value assuming a 60 percent energy efficiency).

Table 3.5: Correlations Between SPT and CPT Results and Friction Angle of Cohesionless Soils.

(Source: Kulhawy and Maine, 1990).

	In-Situ Test Results	Relative Density	ϕ' (degrees)	
			(a) [3]	(b) [4]
SPT N-Value [1] (blows/300 mm or blows/ft)	0 to 4	Very Loose	< 28	< 30
	4 to 10	Loose	28 to 30	30 to 35
	10 to 30	Medium	30 to 36	35 to 40
	30 to 50	Dense	36 to 41	40 to 45
	> 50	Very Dense	> 41	> 45
Normalized CPT cone bearing resistance (q_c/P_a) [2], [4]	< 20	Very Loose	< 30	
	20 to 40	Loose	30 to 35	
	40 to 120	Medium	35 to 40	
	120 to 200	Dense	40 to 45	
	> 200	Very Dense	> 45	

Notes:
(1) SPT N-values are field, uncorrected values.
(2) P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.
(3) Range in column (a) from Peck, Hanson, and Thornburn (1974).
(4) Ranges in column (b) and for CPT are from Meyerhof (1956).

Figure 3.3(b) illustrates the relatively large variation of the resulting value of friction angle as the soil changes from very loose to very dense, with the normalized in situ effective overburden being kept constant. Because in situ compactness of natural cohesionless soils cannot be easily reproduced in the laboratory due to sample disturbance, the friction angle of these soils is not commonly evaluated with lab testing. Therefore, for cohesionless soils, it is common to rely on the STP results and correlations similar to those presented in Figure 3.3.

Fine-Grained Soils

In fine-grained soils, the mobilized strength is a function of the rate of loading in relation to the ability of the soil to drain excess pore pressures and of the nature of the soil. Fine-grained soils may exhibit drained and undrained shear strengths. The drained strength develops when no excess porewater pressures are generated during loading (i.e., porewater is allowed to dissipate completely while loading) and volume changes are allowed to occur. The undrained shear strength of a saturated, fine-grained soil develops when excess porewater pressures are generated during loading (i.e., no drainage of porewater occurs while soil is loaded) and the soil undergoes no change in volume. For normally consolidated, saturated fine-grained soils, an increase of porewater pressure during loading decreases the effective stress in the soil and thus decreases the undrained strength of the soil, whereas a decrease of porewater pressure during loading increases the effective stress in the soil and a corresponding increase in the undrained strength of the soil.

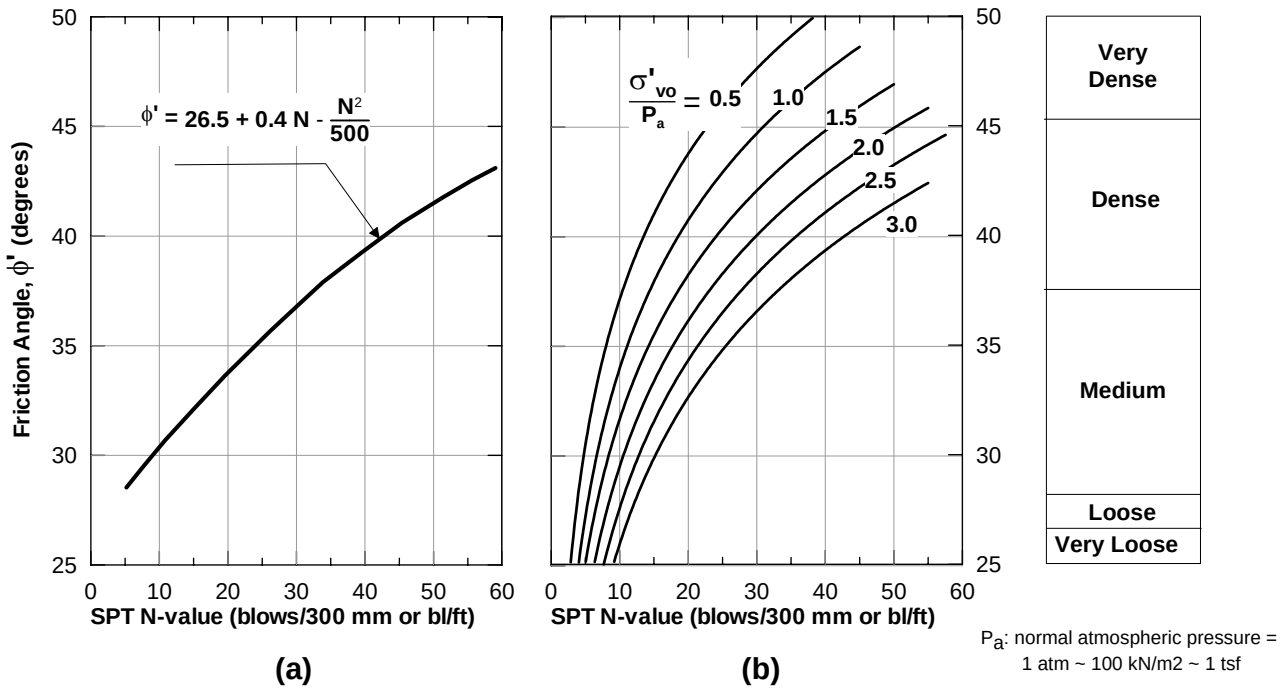
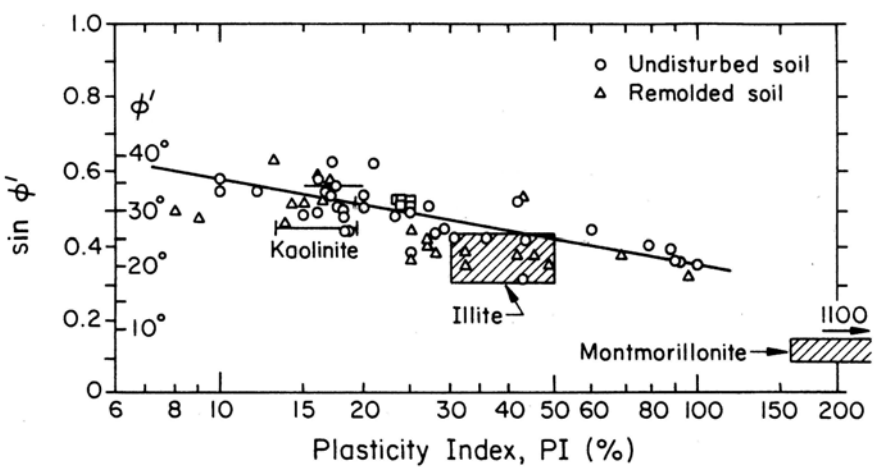


Figure 3.3: Friction Angle of Cohesionless Soils (a) from Uncorrected SPT N-Values (Modified after Peck, Hanson, and Thornburn, 1974) and (b) as a Function of Normalized Overburden (Modified after Schmertmann, 1975).

In soil nail walls in fine-grained soils, the drained strength should be considered only when analyzing the long-term stability of a soil nail wall under a steady, static loading condition. For this case, the drained strength is mobilized when loads are applied at a slow rate and excess pore pressures are not generated. However, this condition is typically not the most critical in normally consolidated fine-grained soils. The drained shear strength of fine-grained soils is expressed as an effective friction angle, ϕ'. The correlation shown in Figure 3.4 between the peak effective friction angle and the plasticity index, PI, can be used to estimate the drained shear strength of fine-grained soils. Because of the scatter shown in Figure 3.4, it is recommended that laboratory tests be conducted to determine the effective friction angle of fine-grained soils. The consolidated

undrained triaxial tests with porewater pressure measurements are the most commonly performed laboratory test to evaluate this parameter.

The undrained shear strength must be considered for the short-term stability of slopes and any soil structure constructed in nearly saturated, soft to medium stiff fine-grained soils. It was discussed previously that the construction of soil nail walls in these soils is not typically advantageous. However, if soil nailing is considered, the short-term stability of a soil nail wall must be evaluated using the undrained shear strength, S_u. Most commonly, the short-term strength (i.e., without enough time for consolidation) is more critical than the strength for long-term conditions in these soils. The undrained strength is mobilized when the loads are applied so that the excess porewater pressure dissipation, which is controlled by the low hydraulic conductivity (k) of these soils (typically, $k < 10^{-5}$ cm/s), cannot take place fast enough. This situation may arise during construction. The undrained shear strength must also be considered for other situations where the rate of loading is relatively high, as is the case for seismic loading.



Source: Mitchell (1993), Kulhawy and Mayne (1990).

Figure 3.4: Correlation Between Drained Angle of Friction of Fine-Grained Soils and Plasticity Index.

The undrained shear strength of fine-grained soils can be assessed using field or laboratory testing techniques. A detailed discussion of the methods used to evaluate the undrained shear strength is beyond the scope of this document, but this information may be found elsewhere (e.g., Kulhawy and Mayne, 1990 and Sabatini et al., 2002). For completeness, however, a brief introduction to the techniques (and limitations) of undrained strength measurements follows. A discussion is presented for: (1) correlations to field testing results, (2) correlations to index properties, and (3) direct measurement in the laboratory.

In the feasibility stage of a soil nail wall, the undrained shear strength in fine-grained soils can be estimated from the field VST where the undrained strength is directly measured. In addition, it is common to use field STP N-values (not very reliable) or CPT results to assess the undrained strength of fine-grained soils. Table 3.6 presents some common and useful correlations for the undrained strength using SPT and CPT results. The correlation of undrained shear strength to SPT

N-values has significant limitations and should be used only for very preliminary estimates. The correlation of undrained shear strength to CPT results using N_k = 15 typically provides reasonable estimates. One of the significant advantages of using the CPT is that a continuous profile of estimated values of the undrained shear strength can be developed.

In the absence of field testing results, Table 3.7 presents a summary of typical correlations between the normalized undrained shear strength ratio, the plasticity index (PI), and the preconsolidation history. The preconsolidation pressure, σ'_p, is determined from consolidation tests (see Table 3.4 for reference on this test).

Table 3.6: Correlations Between SPT and CPT Results and Undrained Strength of Fine-Grained Soils.

(Source: Kulhawy and Maine, 1990).

Equation	Soil Condition	Associated Test
$S_u = 0.29\ P_a\ N^{0.72}$	Normally consolidated to lightly overconsolidated	SPT
$S_u = \dfrac{q_c - \sigma_{vo}}{N_k}$	Various	CPT

Symbols: P_a = atmospheric pressure, N = uncorrected SPT blow count, q_c = cone tip resistance, σ_{vo} = total overburden stress (same units as q_c), N_K = cone factor; typical value is 15.

Table 3.7: Correlations with Index Parameters and Preconsolidation History for Clays.

(Source: Kulhawy and Mayne, 1990).

Equation	Soil Condition	Associated Test	Reference
$S_u/\sigma'_p = 0.11+0.0037\ PI$	Normally consolidated and overconsolidated	VST	Skempton (1957), Chandler (1988)
$S_u/\sigma'_c = 0.129+0.00435\ PI$	Normally consolidated	Isotropically Consolidated Triaxial Compression (based on Cam-Clay, a theoretical model)	Wroth and Houlsby (1985)
$S_u/\sigma'_p = (0.23 \pm 0.04)\ OCR^{0.8}$	Normally consolidated and overconsolidated, low to moderate PI	Mainly Isotropically Consolidated Triaxial Compression	Jamiolkowski et al. (1985)
$S_u/\sigma'_p = 0.22$	Slightly overconsolidated, low PI	Direct Simple Shear	Mesri (1988)

Symbols: PI = Plastic Index.
σ'_p = Effective preconsolidation pressure. Use $\sigma'_p = \sigma'_{vo}$ = effective vertical overburden pressure for normally consolidated conditions.
σ'_c = Effective consolidation pressure in an isotropically consolidated triaxial CU test.
OCR = Overconsolidation ratio = σ'_p/σ'_{vo}.

Correlations are valuable indicators of anticipated performance, particularly at a preliminary stage of the project. When the project moves from a preliminary to a "design" stage, the undrained shear strength of fine-grained soils should always be assessed using the soils from the project site. Estimations of shear strength based on correlations should not replace strength parameters obtained directly from VST or from laboratory testing.

In the laboratory, the undrained shear strength of fine-grained soils is commonly estimated using unconfined compressive strength (UC) tests or other triaxial tests. While UC tests are routinely performed because they are simple and inexpensive, it is recommended that design strength parameters not be based on the UC tests because these results are less reliable than those based on other laboratory strength tests. In general, UC-based values of the undrained shear strength tend to be conservative. Triaxial compression tests are more costly than UC tests but provide a more reliable indication of the soil shear strength. The two most common triaxial tests are the isotropically consolidated undrained compression triaxial (ICU) test with pore pressure measurements and unconsolidated undrained (UU) compression triaxial test. Of the two tests, ICU tests provide more reliable assessment of the total and effective strength parameters of fine-grained soils.

One final comment regarding the shear strength of overconsolidated fine-grained soils is warranted. These soils are typical of the fine-grained soils where soil nail walls may be feasible. Overconsolidated, fine-grained soils tend to expand during undrained loading while the soils shear strength is mobilized. Negative excess porewater pressures are induced and cause the soil effective stresses to increase and thereby cause a more stable condition. The excess pore pressures are dissipated after some time and the strength decreases in the long-term. Therefore, the short-term strength in these soils is unconservative as compared to the long-term strength. This situation may take place in the excavation of overconsolidated fine-grained soils. In overconsolidated soils, design analyses should be performed in terms the effective fully remolded friction angle ϕ_{rem}', as progressive slope failure is always of concern in these soils. The use of the (peak) effective friction angle is unconservative. The remolded friction angle can be determined with consolidated large-displacement ring shear tests or from correlations with plasticity indices (Kulhawy and Mayne, 1990).

3.7 SOIL CREEP POTENTIAL

Sustained, long-term loading of fine-grained soils surrounding soil nails may cause creep deformation. Creep takes place under constant effective stresses in the soil and may cause deformations that adversely affect the lateral deflection of soil nail walls.

There are no specific criteria that can be used to establish whether a soil exhibits unfavorable creep potential. However, practice has shown that soils with potential for creep include:

1. fine-grained soils with a liquid limit (LL) ≥ 50;
2. fine-grained soils with plasticity index (PI) ≥ 20;
3. fine-grained soils with undrained shear strengths ≤ 50 kPa (1,000 psf);

4. a liquidity index (LI) ≥ 0.2; and
5. organic soils.

Creep potential can be directly evaluated during the field testing of individual soil nail load tests. In these tests, a load is applied to the nail in various load increments, and at selected load increments, a creep test is conducted. The creep test consists of holding the load applied to the soil nail during a period of up to an hour and measuring the cumulative nail head displacement at increasingly longer intervals. By relating the increment of nail head displacement over a certain time, a creep rate can be obtained. Creep rates exceeding 2 mm (0.08 in.) in a time period between 6 and 60 minutes in logarithmic scale indicate substantial creep potential. If excessive creep is calculated, it is necessary to modify the design by reducing nail spacing or increasing the nail length. Details of creep testing are provided in Chapter 8.

3.8 ROCK CLASSIFICATION

Soil nail walls can be constructed in weathered rock or fractured rock, provided that planes of weakness, if present, are not preferentially aligned towards the open excavation face. Rock core samples are classified based on a visual identification in the field and laboratory, and from field descriptions following the procedure contained in ASTM Guide D5878-00 (ASTM, 2002). Descriptions of rock samples include rock type, mineral composition, texture (i.e., stratification, foliation), degree of weathering, and discontinuities. Engineering rock properties are evaluated considering both the rock mass and the intact rock. Rock mass refers to a large-scale view of the in situ rock, which normally contains numerous defects or discontinuities (i.e., joints and fractures) that separate discrete blocks of relatively competent rock. In contrast, in a small-scale, the blocks of rock that are free of macro discontinuities are referred to as intact rock.

The quality of the rock mass for engineering purposes (in particular strength) can be estimated from the core recovery per run (in percent), rock type, the Rock Quality Designation (RQD, as presented in Deere et al., 1967 and Deere and Deere, 1988), and the frequency and orientation of discontinuities. The rock suitability for engineering applications is commonly expressed though indices. Examples of these indices include the Rock Structure Rating (RSR) and Rock Mass Rating System (RMR) developed by Bieniawski (1976, 1989), the Unified Rock Classification System (URCS) developed by Williamson (1984), and the Slope Rock Mass Rating System (SMS) developed by Bieniawski (1989).

The strength parameters of the mass rock can be estimated from empirical methods (Hoek and Brown, 1980; Hoek, 1983; Hoek, 1994). The specific determination of strength parameters of intact rock and rock mass is beyond the scope of this document. Table 3.8 presents procedures and laboratory tests commonly used to assess the strength of rock. The reader should refer to the cited references or to Sabatini et al. (2002) for additional information.

Table 3.8: Common Rock Procedures and Laboratory Tests.

TEST	STANDARD		APPLICABILITY
	ASTM (1)	AASHTO (2)	
Rock Mass Classification	D5878-00	—	Rocks Mass
Rock Unconfined Compressive Strength	D2938-95	—	Intact Rock
Direct Shear of Rock	D5607-95	—	Interface of Intact Rocks
Rock Triaxial Compression (no pore pressure measurements)	D2664-95a	T226	Intact Rock

Notes: (1) Individual ASTM standards can be found in ASTM (2002).
(2) Individual AASHTO standards can be found in AASHTO (1992).

3.9 SOIL CORROSION POTENTIAL

For all permanent soil nail walls and, in some cases, for temporary walls, the soil corrosion potential must be evaluated. Subsurface conditions exhibiting high corrosion potential usually do not preclude the use of soil nails, providing the design life, type of structure, and proper corrosion protection for the soil nail bars are properly considered. Various ground conditions promote corrosion including: (1) low electrical resistivity of soil, (2) high concentration of chlorides or sulfates, and (3) too low or too high hydrogen potential (pH) of soil or groundwater. Examples of soils with corrosion potential include: (1) acidic soils, (2) organic soil, and (3) soils with materials of industrial origin (slag, fly ash, fills with construction debris, mine tailings, and acid mine waste).

Corrosion potential is of primary concern in aggressive soil applications and is evaluated based on tests results of the following properties:

- pH (potential of hydrogen);
- electrical resistivity;
- chloride content;
- sulfate content; and
- presence of stray currents.

The test-based criteria listed in Table 3.9 are used to classify the corrosion potential of the ground. The ground is classified with a strong corrosion potential or aggressive if any one of the conditions listed in the first column of Table 3.9 exceeds the limits listed in the third column of the table during the service life of the soil nail wall. If all the conditions listed in the first column of Table 3.9 satisfy the conditions listed in the fourth column of Table 3.9, the ground is classified with no (or occasionally mild) corrosion potential. Additional information on corrosion is provided in Appendix C, which also provides guidelines for the selection of corrosion protection measures.

Table 3.9: Criteria for Assessing Ground Corrosion Potential.

Test	Units	Strong Corrosion Potential/Aggressive	Mild to no Corrosion Potential/Non-Aggressive	ASTM Standard	AASHTO Test Method
PH	–	< 4.5, > 10	5.5 < pH < 10	G51	T 289-91
Resistivity	ohm-cm	< 2,000	Greater than 5,000	G57	T 288-91
Sulfates	ppm(1)	> 200	Less than 200	D516	T 290-91
Chlorides	ppm	> 100	Less than 100	D512	T 291-91
Stray current	–	Present	–	–	–

Note: (1) ppm = parts per million.

3.10 BOND STRENGTH

The pullout capacity of a soil nail installed in a grouted nail hole is affected by the size of the nail (i.e., perimeter and length) and the ultimate bond strength, q_u. The bond strength is the mobilized shear resistance along the soil-grout interface. The bond strength is rarely measured in the laboratory and there is no standard laboratory testing procedure that can be used to evaluate bond strength. Therefore, designs are typically based on conservative estimates of the bond strength obtained from field correlation studies and local experience in similar conditions. As a result of this dependency on local conditions, contract specifications include a strict requirement that some percentage of the soil nails be load tested in the field to verify bond strength design. Load testing is described in Chapter 8.

From experience, it is known that for drilled and grouted nails, the bond strength is affected by:

- ground conditions around the nail (soil type and conditions);
- soil nail installation including:
  - drilling method;
  - grouting procedure;
  - grout nature;
  - grout injection (e.g.; gravity or under pressure); and
- the size of the grouted zone.

For drilled and grouted nails in cohesionless soil, the magnitude of the overburden pressure and the nature of the granular soil affect the soil friction angle, which in turn affects the bond strength. For grouted nails in fine-grained soil, the bond strength is in general a fraction of the undrained shear strength of the soil. In general, the bond strength increases (but not linearly) with the undrained shear strength of the soil. For softer soils, the ratio of bond strength to the soil undrained shear strength, q_u/S_u, is higher than for stiffer soils.

Typical values of ultimate bond for drilled and grouted nails installed in various soils and using different drilling methods are presented in Table 3.10. The values in this table correspond to gravity grouting only. Table 3.10 allows the designer to estimate bond strengths by entering the name of common rock types and soils described by their geologic origin (e.g., colluvium, moraine,

etc.). Although Table 3.10 covers a wide variety of rock/soil types, drilling methods, and ground conditions, the database used to develop the table does not cover all the possible case combinations.

The lower and upper bounds contained in Table 3.10 correspond approximately to the least and most favorable conditions for a particular soil type and construction method. For example, in the case of granular soils, for which the ranges are relatively broad, the lower and upper bonds may correspond to conditions of loose and very dense materials, respectively. These values inherently contain some level of conservatism and can be used as preliminary values for design. The ultimate bond strength in soil may be estimated in the field during the site investigation phase of the project from the results of PMT (not commonly used in the United States), using the following correlation:

$$q_u \text{ (kPa)} = 14\ P_L \text{ (MPa)} \left[6 - P_L \text{ (MPa)} \right] \qquad \text{(Equation 3.4)}$$

where P_L is the limit pressure recorded in MPa as measured with the pressuremeter, and q_u is the ultimate bond strength calculated in kPa. In English units, the equation above is:

$$q_u \text{ (psi)} = \frac{1}{214} P_L \text{ (ksf)} \left[125 - P_L \text{ (ksf)} \right] \qquad \text{(Equation 3.5)}$$

In the case of rocks, the bond strength can also be estimated from correlations with the compressive strength of intact rock. The Post-Tensioning Institute (PTI, 1996) alternatively suggests that the ultimate bond stress between rock and grout can be approximated as 10 percent of the unconfined compressive strength of the rock up to a maximum of 4,000 kPa (600 psi). However, it is recommended that estimates of bond strengths do not exceed the values for typical rocks contained in Table 3.10.

Bond strength can increase significantly with pressure grout (Shields et al., 1978; Weatherby, 1982). For grout pressures of less than 350 kPa (50 psi) in soil, the bond strength has been found to be as much as two times the bond strength resulting from gravity-placed grout (Elias and Juran, 1991). In rock, the increase in bond strength due to pressure grouting can be even larger. The increment of bond strength is not proportional with pressure and tends to achieve a limiting value. For this condition, only field load tests can provide reliable estimates of bond strengths.

Because of the difficulty in estimating bond strength, it is common practice to require pre-production soil nail load tests to verify the bond strengths included in construction specifications (if the method or procedure type of specifications are used, see Chapter 7) and establish the minimum required nail length to support a specified nail design load.

For the special case where driven nails are used in granular soil (only practical application), the bond strength has been found to be predominantly frictional and to be affected by the same factors that affect soil friction strength including: overburden pressure along the bonded length, relative density of soil, and, in addition, by the nail installation method. The apparent friction angle at the soil-nail interface can be estimated to be between 0.75 to 1.0 times $\tan\phi'$, where ϕ' is the effective friction angle of the granular material (Elias and Juran, 1991).

Table 3.10: Estimated Bond Strength of Soil Nails in Soil and Rock.

(Source: Elias and Juran, 1991).

Material	Construction Method	Soil/Rock Type	Ultimate Bond Strength, q_u (kPa)
Rock	Rotary Drilled	Marl/limestone Phyllite Chalk Soft dolomite Fissured dolomite Weathered sandstone Weathered shale Weathered schist Basalt Slate/Hard shale	300 - 400 100 - 300 500 - 600 400 - 600 600 - 1000 200 - 300 100 - 150 100 - 175 500 - 600 300 - 400
Cohesionless Soils	Rotary Drilled	Sand/gravel Silty sand Silt Piedmont residual Fine colluvium	100 - 180 100 - 150 60 - 75 40 - 120 75 - 150
	Driven Casing	Sand/gravel low overburden high overburden Dense Moraine Colluvium	 190 - 240 280 - 430 380 - 480 100 - 180
	Augered	Silty sand fill Silty fine sand Silty clayey sand	20 - 40 55 - 90 60 - 140
	Jet Grouted	Sand Sand/gravel	380 700
Fine-Grained Soils	Rotary Drilled	Silty clay	35 - 50
	Driven Casing	Clayey silt	90 - 140
	Augered	Loess Soft clay Stiff clay Stiff clayey silt Calcareous sandy clay	25 - 75 20 - 30 40 - 60 40 - 100 90 - 140

Notes: Convert values in kPa to psf by multiplying by 20.9
Convert values in kPa to psi by multiplying by 0.145

CHAPTER 4 CONSTRUCTION MATERIALS AND METHODS

4.1 INTRODUCTION

This chapter presents information on construction materials and methods used for the construction of soil nail walls typically used in U.S. highway applications. Conventional U.S. practice includes the use of drilled and grouted soil nails. However, other soil nail installation methods and materials have been developed and are briefly introduced here.

Nail installation techniques can be classified as:

- **Drilled and grouted soil nails**: These are approximately 100- and 200-mm (4- to 8-in.) diameter nail holes drilled in the foundation soils. These holes are typically spaced about 1.5 m (5 ft) apart. Steel bars are placed and the holes are grouted. Grouted soil nails are the most commonly used soil nails for FHWA projects and they can be used as temporary and permanent applications (provided that appropriate corrosion protection is in place).

- **Driven soil nails:** These soil nails are relatively small in diameter [19 to 25 mm (¾ to 1 in.)] and are mechanically driven into the ground. They are usually spaced approximately 1 to 1.2 m (3 to 4 ft) apart. The use of driven soil nails allows for a faster installation (as compared to drilled and grouted soil nails); however, this method of installation cannot provide good corrosion protection other than by sacrificial bar thickness. For this reason, driven nails are only used in the United States for temporary applications. This method is not currently used in FHWA projects.

- **Self-drilling soil nails:** These soil nails consist of hollow bars that can be drilled and grouted in one operation. In this technique, the grout is injected through the hollow bar simultaneously with the drilling. The grout, which exits through ports located in a sacrificial drill bit, fills the annulus from the top to the bottom of the drillhole. Rotary percussive drilling techniques are used with this method. This soil nail type allows for a faster installation than that for drilled grouted nails and, unlike, driven soil nails, some level of corrosion protection with grout is provided. This system is most commonly used as temporary nails. This method is not currently used in FHWA projects.

- **Jet-grouted soil nails:** In this technique, jet grouting is performed to erode the ground and allow the hole for the nail (subsequently installed) to be advanced to the final location. The grout provides corrosion protection to the central bar. In a second step, the bars are typically installed using vibro-percussion drilling methods. This method is not currently used in FHWA projects.

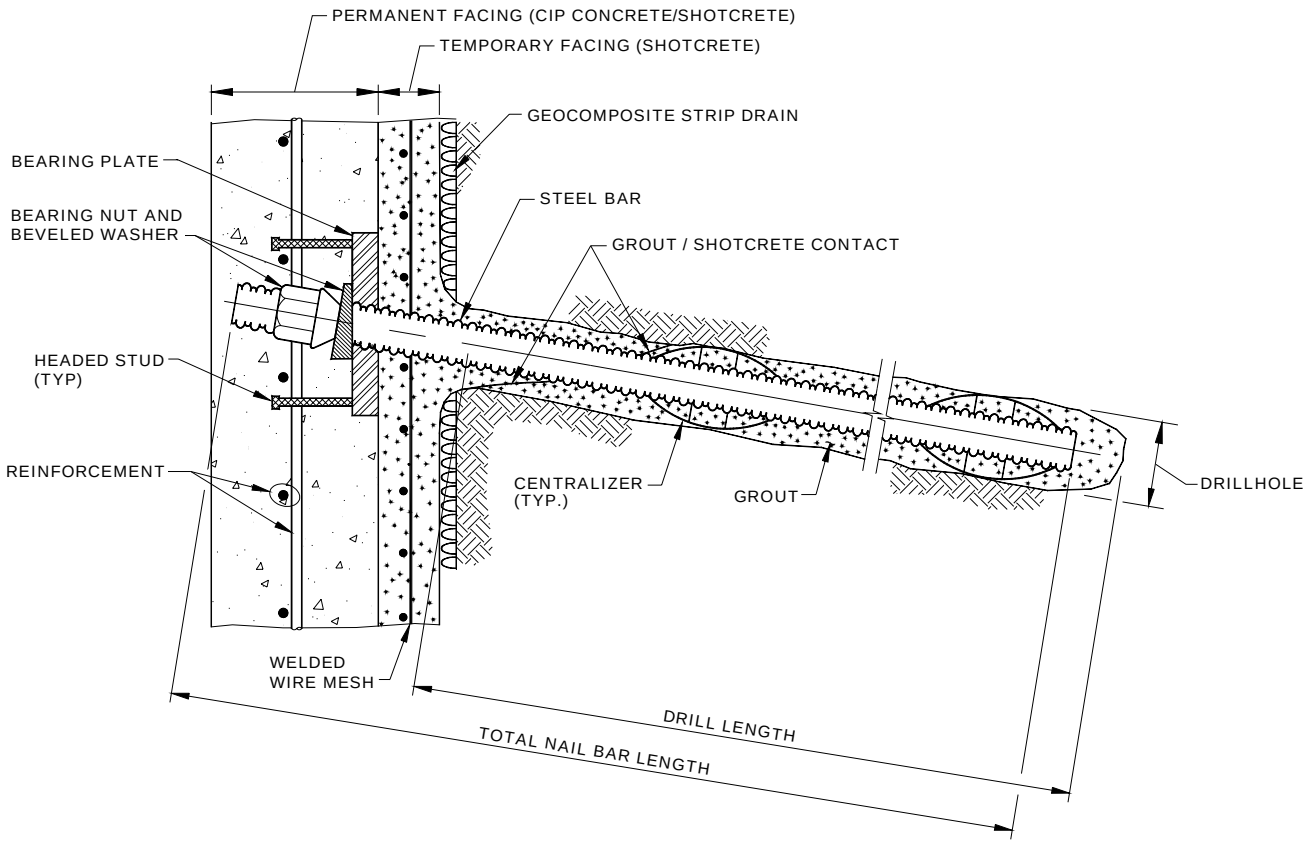
- **Launched soil nails:** In this method, bare bars are "launched" into the soil at very high speeds using a firing mechanism involving compressed air. Bars are 19 to 25 mm (¾ to 1 in.) in diameter and up to 8 m (25 ft) in length. This technique allows for a fast installation with little impact to project site; however, it may be difficult to control the length of nail that penetrates the ground. These types of soil nails are only used for temporary nails. This method is not currently used in FHWA projects.

4.2 COMPONENTS OF A SOIL NAIL WALL

The components of a soil nail wall installed using the techniques listed above vary from one technique to another. As discussed in Chapter 2, most commonly in the United States, soil nails are grouted nails consisting of a drilled hole in which a bar is placed and then grouted under gravity. Driven soil nails have been more commonly used in France and Germany than in the United States and are only used for temporary applications. Jet-grouted and launched nails are not common in the United States.

Although the concept of soil nail walls is not proprietary, several specialized components or procedures are under U.S. or international patents. Patented components include, but are not limited to, some threaded bars, corrosion-protection systems, and nail installation systems (e.g., self-drilling, jet-grouted, and launched soil nails). Hollow bars are proprietary systems and are used only in temporary walls because of concerns regarding the consistency of the corrosion protection.

The remainder of this section presents a description of the main components of a typical soil nail used in the U.S. practice (Figure 4.1).



Source: Porterfield et al. (1994).

Figure 4.1: Main Components of a Typical Soil Nail.

4.2.1 Nail Bars

Steel reinforcing bars used for soil nails are commonly threaded and may be either solid or hollow. Bars generally have a nominal tensile strength of 420 MPa [60 kips per square inch (ksi) or Grade 60] or 520 MPa (75 ksi or Grade 75). The common U.S. practice of soil nailing involves the use of solid steel bars of 420 or 520 MPa (Grade 60 or 75).

Bars with a tensile strength of 665 MPa (Grade 95) and as high as 1,035 MPa (Grade 150) may be considered for soil nailing. However, their use must be restrictive. Bars with lower grades are preferred because they are more ductile, less susceptible to corrosion, and readily available. Grade-150 bars should not be used in conventional soil nail applications because this material tends to be more brittle, particularly under shear, and more susceptible to stress corrosion than steel with lower grades. The use of high-grade steel is more typical for ground anchors [see GEC No. 4 "Ground Anchor Systems," by Sabatini et al. (1999)] where the design loads are usually much greater than for soil nails. Bars of 665 MPa (Grade 95) may be acceptable for soil nails as long as the steel ductility is comparable to lower-grade steels. Bars of 420 or 520 MPa (Grade 60 or 75) should conform to the standard ASTM A 615, whereas the 1,035 MPa (Grade 150) steel should conform to ASTM A722. Typical nail bar characteristics are presented in Appendix A.

Threaded bars for typical soil nail wall applications are available in 19-, 22-, 25-, 29-, 32-, 36-, and 43-mm diameter (No. 6, 7, 8, 9, 10, 11, and 14 in English units) up to approximately 18 m (59 ft) in length. Bars having diameters smaller than 25 mm (No. 8) should not be used or used with great care in applications where long bars are required (e.g., high walls) because they tend to bend excessively during handling and installation. If needed, couplers can be used to extend the length of bars in excess of 18 m (59 ft); however, soil nails in excess this length are typically not required for most highway projects. Table A.1 (Appendix A) presents the properties of the most commonly available sizes of threaded bars.

4.2.2 Nail Head

The nail head comprises two main components, the bearing-plate, hex nut, and washers; and the headed-stud (Figure 4.1). The bearing plate is made of Grade 250 MPa (Grade 36) (ASTM A36) steel and is typically square, 200- to 250-mm (8- to 10-in.) side dimension and 19-mm (¾-in.) thick. The purpose of the bearing plate is to distribute the force at the nail end to the temporary shotcrete facing and the ground behind the facing. The bearing plate has a central hole, which is inserted over the nail bar. Beveled washers are then placed and the nail bar is secured with a hex nut or with a spherical seat nut. Washers and nuts are steel with a grade consistent with that of the nail bar commonly of 420 or 520 MPa (Grade 60 or 75). Nuts are tightened with a hand-wrench. The head-stud connection may consist of four headed studs that are welded near the four corners of the bearing plate to provide anchorage of the nail head into the permanent facing. For temporary walls, the bearing plate is on the outside face of the shotcrete facing. Other types of devices may be used in lieu of headed studs to provide anchorage in final facings and are briefly described in this chapter.

4.2.3 Grout

Grout for soil nails is commonly a neat cement grout, which fills the annular space between the nail bar and the surrounding ground. In ground with potential for drillhole caving, a neat cement grout is always used. Sand-cement grout can also be used in conjunction with open hole drilling (i.e., for non-caving conditions) for economic reasons. Cement types I, II, III, or V conforming to ASTM C 150 can be used. Cement Type I (normal) is recommended for most applications. Cement Type III is grounded finer, hardens faster, and can be used when a target grout strength is required to be achieved faster than for typical project conditions. Cement Type II hardens at a slower rate, produces less heat, and is more resistant to the corrosive action of sulfates than Cement Type I.

The water/cement ratio for grout used in soil nailing applications typically ranges from 0.4 to 0.5. In some cases, a stiffer grout with a slump on the order of 30 mm (1½ in.) may be used. The need for a stiffer grout may arise when the hollow-stem auger drilling method is used or it is desired to control leakage of grout into highly permeable granular soils or highly fractured rock. Occasionally, the stiff consistency of the grout may cause difficulties with the installation of the centralizers. In this case, the grout itself may provide sufficient support to centralize the nail bar within the drillhole. Regardless of the ability of the stiff grout to support the nail bar, centralizers should always be used to assure that a minimum grout cover around the nail bar is achieved.

The characteristics of the grout have a strong influence on the ultimate bond strength at grout-ground interface. Commonly, a minimum 28-day unconfined compressive strength of 21 MPa (3,000 psi) is specified for the grout. Admixtures are not typically required for most applications, but plasticizers can be used to improve grout workability for projects located in high-temperature climates or where project constraints dictate that the grout must be pumped over long distances. Typically, the improved workability of grout due to plasticizers can be extended up to approximately one hour. The use of air entrainment agents can improve workability and reduce cracking potential, but they cause the grout to develop a more open matrix and lose some of the chemical corrosion protection provided by cement. Therefore, its use should be approved only when other corrosion protection methods, other than grout cover, are present, or the thickness of the grout cover is increased. Some proprietary grouts contain chemicals that provide zero-volume shrinkage, which is desirable to minimize cracking and enhance bond strength. Where admixtures are being considered for use, tests should be performed to verify that the grout and bond properties of the grout are not adversely affected.

Grout is pumped shortly after the nail bar is placed in the drillhole to reduce the potential for hole squeezing or caving. In solid nail bar applications, the grout is injected by tremie methods through a grout pipe, which is previously inserted to the bottom of the drillhole, until the grout completely fills the drillhole (Figure 4.2). The grout pipe typically consists of heavy-duty plastic tubing varying between 9.5- and 19-mm (0.4-in. and 0.8-in.) in OD. Grout pipes are removed when used as part of the installation of production nails and commonly left in place in when used for soil nails that are to be load tested. Grout injection must be conducted smoothly and continuously in such a way that the space between the drillhole and the nail bar is filled completely, with no voids or gaps. The bottom of the grout pipe must remain below the grout surface at all times while grout is being pumped into the drillhole. During grouting operations, the portion of the soil nail near the back of the temporary facing may not be completely filled with grout. Because this area is the most

vulnerable to corrosion, it is critical that this area be subsequently filled with shotcrete, or less commonly with a stiff grout to assure complete grout coverage.

Source: Porterfield et al. (1994).

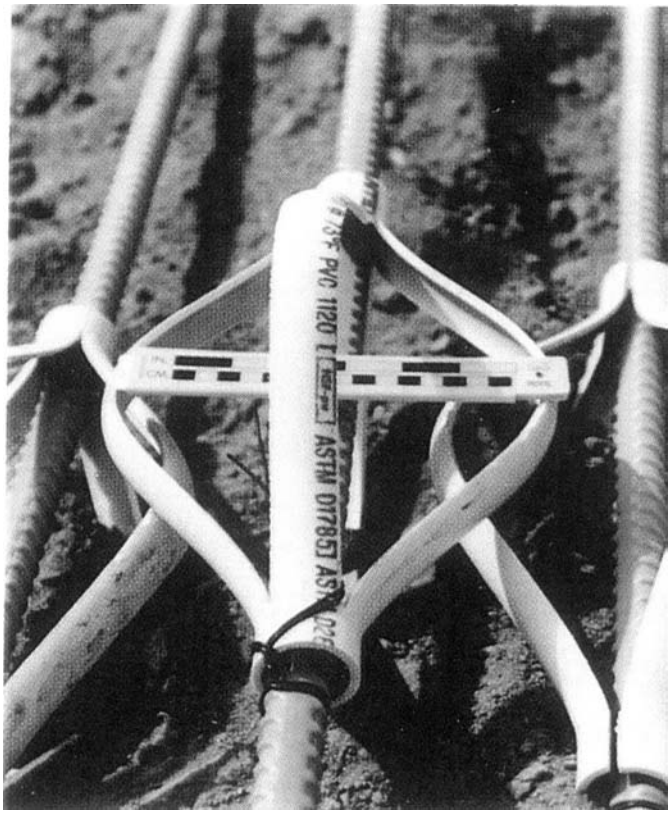
Figure 4.2: Grout Placement (Tremie) through Pipe.

4.2.4 Centralizers

Centralizers are devices made of polyvinyl chloride (PVC) or other synthetic materials that are installed at various locations along the length of each nail bar to ensure that a minimum thickness of grout completely covers the nail bar (Figure 4.3). They are installed at regular intervals, typically not exceeding 2.5 m (8 ft), along the length of the nail and at a distance of about 0.5 m (1.5 ft) from each end of the nail.

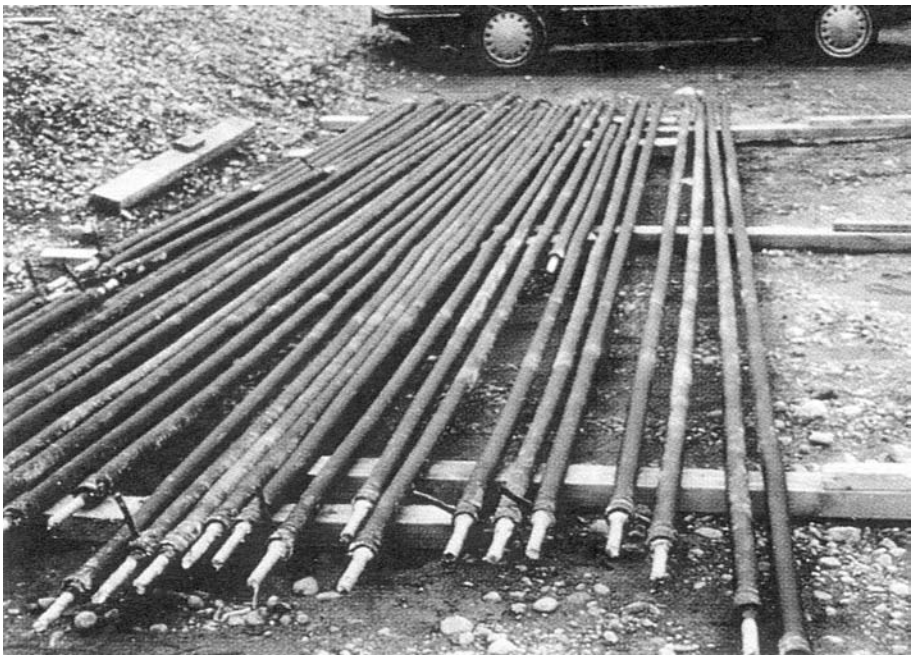
4.2.5 Corrosion Protection Elements

In addition to the cement grout, which provides both physical and chemical protection to the nail bars, other devices are typically used to provide additional corrosion protection, as necessary. Protective sheathings made of corrugated synthetic material [HDPE (High Density Polyethylene) or PVC tube) surrounding the nail bar (Figure 4.4) are usually used to provide additional corrosion protection. The internal annulus between the sheathing and the nail bar is prefilled with grout. This system is commonly referred to as corrosion protection by encapsulation. The sheathing must be a minimum 1-mm (40-mil) thick. In addition, fusion-bonded epoxy coated nail bars are used instead of the bare, threaded bars. The epoxy coating is a dielectric material that impedes the flow of electric currents, which favor corrosion. A more detailed description of methods of corrosion protection as well as guidelines for their selection is presented in Appendix C.



Source: Porterfield et al. (1994).

Figure 4.3: Typical PVC Centralizers Attached to a Nail Bar Prior to Nail Installation.



Source: Porterfield et al. (1994).

Figure 4.4: Nail Bars (in light color at end of nails) Encapsulated by Sheathing (dark color) Prior to Nail Installation.

4.2.6 Wall Facing

Nails are connected at the excavation surface (or slope face) to a facing system, which most commonly consists of a first-stage, temporary facing of shotcrete during construction and, a second-stage, permanent facing of CIP concrete. The purpose of the temporary facing is to support the soil exposed between the nails during excavation, provide initial connection among nails, and provide protection against erosion and sloughing of the soil at the excavation face. The purpose of the permanent facing is to provide connection among nails, a more resistant erosion protection, and an aesthetic finish. Temporary facing typically consists of shotcrete and WWM and additional shorter reinforcement bars (referred to as waler bars) around the nail heads, which are applied after each row of nails is installed in the ground. Permanent facing is commonly constructed of CIP reinforced concrete and WWM-reinforced shotcrete. Prefabricated panels may also be used to construct the permanent wall facing, especially for projects with special aesthetic requirements or where prefabricated panels are more cost-effective.

4.2.7 Drainage System

To prevent water pressure from developing behind the wall facing, vertical geocomposite strip drains are usually installed between the temporary facing and the excavation (Figure 4.5).

Source: Porterfield et al. (1994).

Figure 4.5: Strip Drains under Temporary Shotcrete.

The drainage system also includes a footing drain and weepholes to convey collected drainage water away from the wall face. In some cases, conventional horizontal drains are also installed. Additional information on drainage system construction is provided subsequently in this Chapter. Information on drainage system design is provided in Chapter 6.

4.3 CONSTRUCTION METHODS

4.3.1 Introduction

The sequence of construction for typical soil nail walls was described in Chapter 2 and consisted of:

1. excavation;
2. drilling of nail holes;
3. installation and grouting nails;
4. construction of temporary shotcrete facing;
5. construction of subsequent levels (i.e., repeat Steps 1-4); and
6. construction of a final, permanent facing.

A detailed description of these major construction activities is presented below.

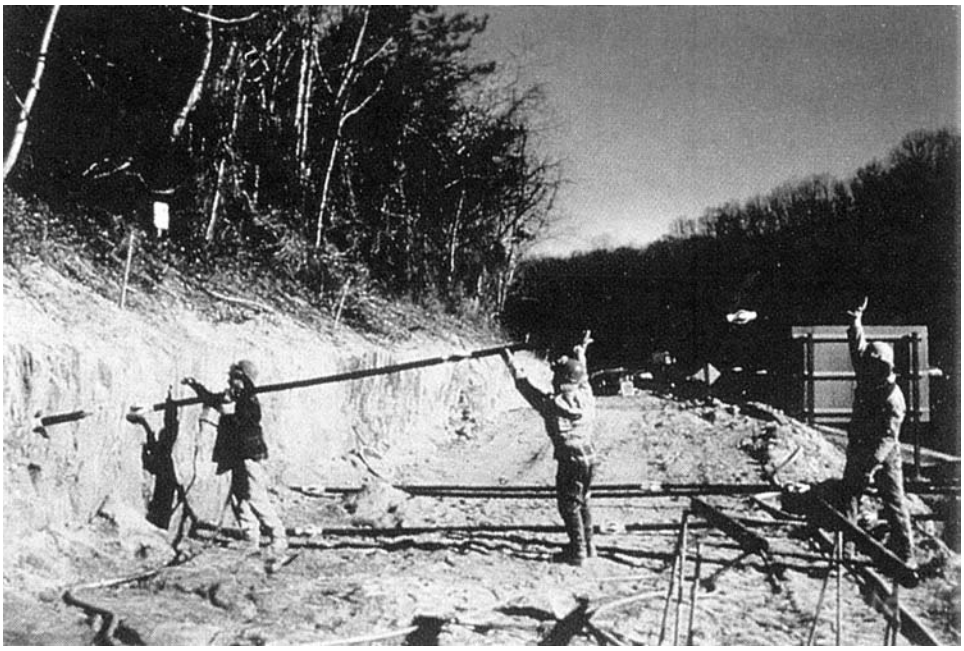
4.3.2 Excavation

Prior to any excavation, surface water controls should be constructed to prevent surface water from flowing into the excavation as this condition will adversely affect construction and potentially cause instability of the excavated face. Collector trenches behind the limits of the excavation usually intercept and divert surface water. Subsequently, soil excavation is performed using conventional earth-moving equipment from a platform, and final trimming of the excavation face is typically carried out using a backhoe or excavator from a platform. The initial lift is typically 1 to 1.2 m (3 to 4 feet) high (Figure 4.6). The excavated face profile should be reasonably smooth and not too irregular to minimize excessive shotcrete quantities. Soil profiles containing cobbles and/or boulders may require hand excavation. A level working bench on the order of 10-m (30-ft) wide is required to accommodate the conventional drilling equipment used for nail installation. Track drills smaller than the conventional drilling equipment can work on benches as narrow as 5 m (15 ft) and with headroom clearance as low as 3 m (9 ft).

In many instances, the most critical situation related to wall stability occurs during construction. For the intermediate and lower excavation lifts, the lowest margins of safety might occur when the open-cut is temporarily unsupported (i.e., before nails and shotcrete are placed at these levels). It is important to watch for early signs of instability during construction, such as bulging and excessive deformation of the excavated soil face. If early signs of wall instability are observed, it is critical to respond quickly and inform the construction superintendent and the design engineer. In addition, it is critical that experienced inspectors and other site supervisory personnel are familiar and trained to respond immediately if this situation arises. In conjunction with this, it is important that the contract specifications be closely followed with regard to the extent of excavation allowed. The exposed length of an excavation should be limited to that which can be stabilized and shotcreted during a single working shift.

For cases where the excavated slope face cannot stand unsupported for the required period of time, a continuous berm may be employed to stabilize the unsupported face section (Figure 4.7a). In this

case the soil nails are installed and grouted first through the stabilizing berm. Subsequently, the berm is excavated and shotcrete is applied along the entire excavation level.



Source: Porterfield et al. (1994).

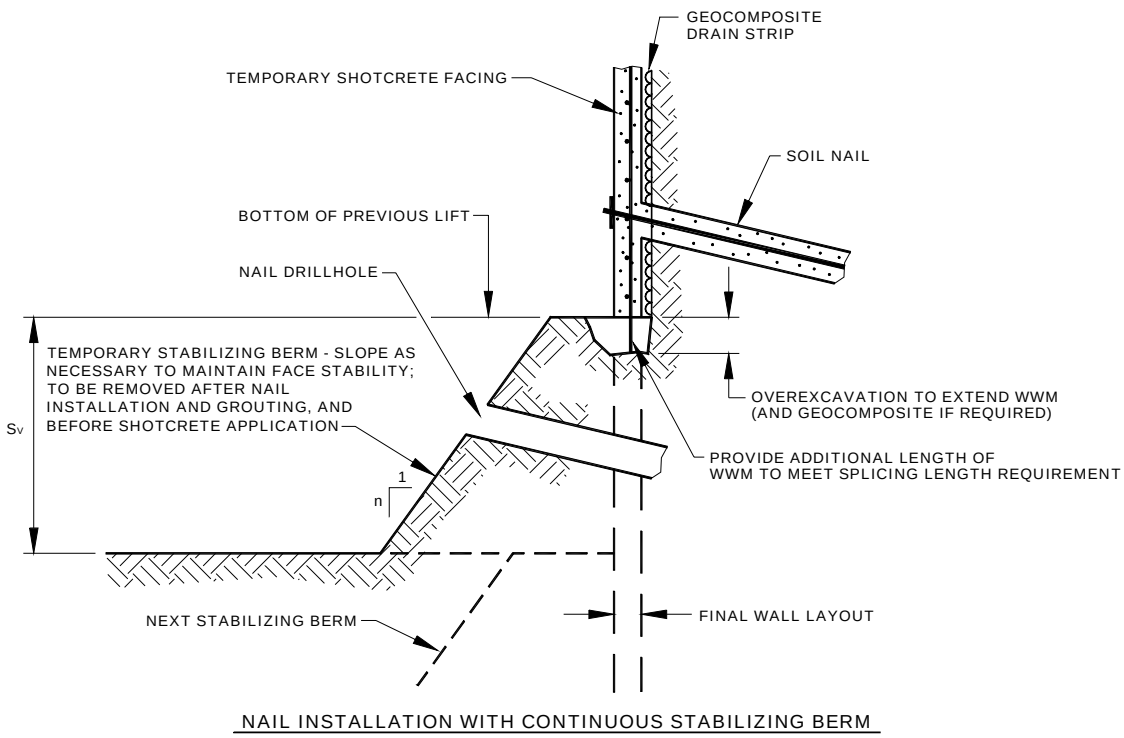
Figure 4.6: Initial Excavation Lift and Nail Installation.

Alternatively, the stabilizing berm can be removed in sequence in which alternating excavation slots are excavated and panels of the stabilizing berm are created in between (Figure 4.7b). The width of the excavation slots should not exceed approximately the soil nail horizontal spacing. Next, soil nails are installed and grouted, and shotcrete is applied on the face of the excavation slots. Subsequently, the stabilizing berm panels are removed and the intermediate soil nails are installed and shotcrete is applied.

The use of stabilizing berms should be used only as a last resource because this procedure may have a significant impact on construction costs and project schedule. If temporary excavation stand up time is a major concern and represents a potential risk, alternative top-down construction methods such as ground anchors may be considered.

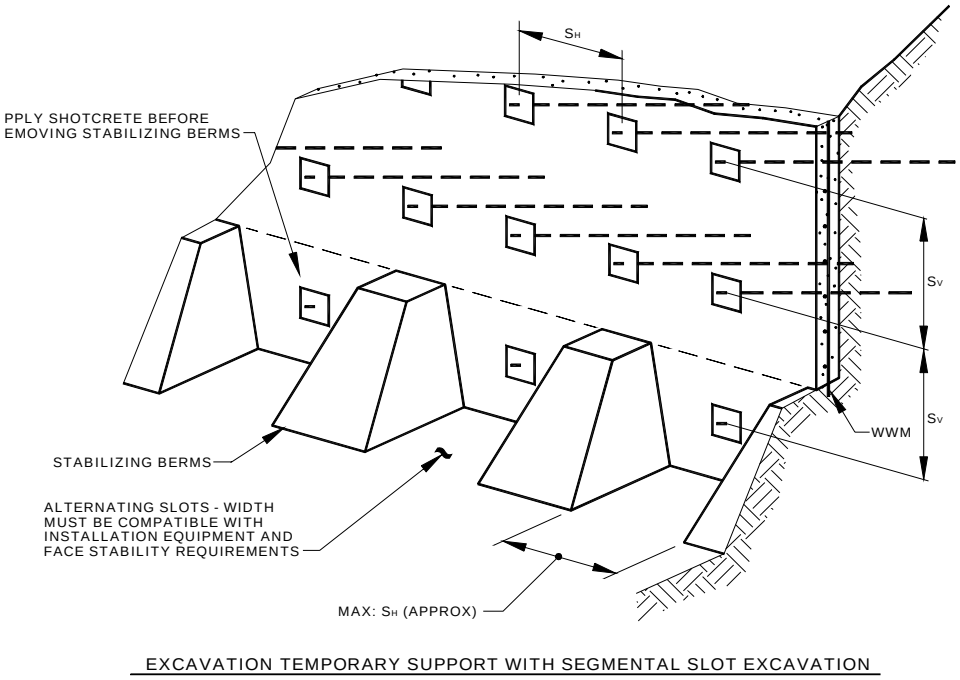
4.3.3 Drilling of Nail Holes

Nail holes (drillholes) are drilled using one of several available drilling methods, including rotary (Figure 4.8), percussion, auger, and rotary/percussion drilling. Table 4.1 presents a summary of the most typical drilling equipment and methods. The design engineer can use this table for guidance, as the selected equipment must be compatible with the anticipated ground conditions and the required total nail length and nail diameter.



Modified from Porterfield et al. (1994).

Figure 4.7 (a): Examples of Alternative Temporary Excavation Support: Stabilizing Berm.



Modified from Porterfield et al. (1994).

Figure 4.7 (b): Examples of Alternative Temporary Excavation Support: Slot Excavation.

Open-hole installation using auger drilling (in particular, hollow-stem augers) is most commonly used on soil nailing projects in the United States because no casing of the drillhole is necessary, high installation rates can be obtained, and costs are relatively low. Nail holes drilled using auger drilling can range between 100 to 300 mm (4 to 12 in.) in diameter. More commonly, drillholes are 150 to 200 mm (5 to 8 in.) in diameter. Contractors will usually select a relatively large drillhole diameter (e.g., 200 mm or 8 in.) to reasonably assure that the ultimate soil nail bond strength required in the construction specifications can be achieved without difficulties minimizing drilling equipment costs.

Percussive drilling methods displace soil by driving drill rods that have a knock-off point on the end. Rotary-percussive methods displace soil by drilling and driving drill rods. Nail holes drilled with rotary-percussive methods typically are 90 to 115 mm (3½ to 4½ in.) in diameter.

From Porterfield et al. (1994).

Figure 4.8: Typical Drilling of Soil Nails with Rotary Method.

The selection of the drilling method may also be controlled by the local availability of equipment and the specific ground conditions to be encountered. Typical soil nail wall contract documents allow the contractor to select the drilling method. However, the design engineer may occasionally restrict the choice of drilling methods and/or procedures based on the subsurface conditions or other project needs. For example, restrictions may be imposed on certain drilling methods if it is deemed that they might have an adverse effect on the integrity of adjacent structures or underground utilities. The selected drilling methods should not cause excessive ground caving into the drill hole. Additionally, the drilling of the upper nail rows should not cause excessive ground surface heave. These potentially damaging effects may be avoided by using drill casing (or hollow self-drilling bars) in unstable ground or by restricting the use of large diameter drilling bits.

Casing the drill hole can increase the cost of soil nail walls significantly, to the point where alternative wall construction methods may be more economical. Cased methods of drilling include the use of single tube and the duplex rotary methods. The single tube method involves drilling with one tube (drill string) and flushing the cuttings outside the tube by air, water, or a combination of water and air. The duplex rotary method has an inner element (drill rods) and an outer tube (casing). The assembly allows drill cuttings to be removed through the annular space between the drill rods and outer casing. Drill hole diameters for these methods are generally 90 to 140 mm (3½ to 5½ in.).

4.3.4 Nail Bar Installation

The most common practice for placing nail bars is inserting them into a predrilled, straight-shafted drillhole ranging from 100 to 300 mm (4 to 12 in.) in diameter. After the nail bar is inserted in the drillhole, the drillhole is filled with clean cement grout, as discussed previously. This method is referred to as open-hole installation. As the grout sets, it bonds to the nail bar and the surrounding ground. The open-hole installation is by far the most commonly used method in soil nail wall construction.

The most common U.S. practice of grouting by gravity provides bond strengths that are sufficient for soil nailing to be a feasible and cost-effective solution. However, in cases where poor soil conditions are encountered, higher bond strengths might be required. High bond strength may be achieved in granular soils and weak fissured rocks by injecting grout or regrouting under pressure. Grouting under pressure increases resistance to pullout relative to the open-hole method due to several mechanisms including an increase in the confining pressure around the grout bulb, higher compaction of the material surrounding the bulb, an interlocking mechanism ("fingers"), and an increase of the grout bulbs effective diameter. Grouting under pressure is seldom used for fine-grained soils. If used, only minor increases in bond strength should be expected because the frictional component and the interlocking mechanism are absent or not significant.

There are instances when the bond strength can deteriorate if appropriate installation procedures are not followed. For example, poor removal of cuttings from the drillhole may cause the bond strength to significantly reduce. In the case of clayey soils with moderate plasticity, on the other hand, excessive mechanical cleaning of the drillhole may tend to remold the clay and thereby decrease the bond strength to values much lower than those expected for the encountered, undisturbed ground conditions. Although the nail capacity has to be verified with proof load tests and additional and/or longer nails may be installed if deemed necessary. It is always desirable to achieve the available bond strength in order to avoid cost overruns and/or change orders.

When nails are installed in ground usually considered to be unfavorable for soil nailing, some adjustments in the conventional soil nail installation procedures will be necessary to achieve the required design bond strengths. An example is when soil nails are installed in loess using the gravity grouting method. In this case, the bottom and lateral portions of the drillhole wall are weakened because the originally dry loess is wetted by the grout and the walls of the drillhole subsequently collapse. The grout may not reach the top of the wall of the drillhole, which thereby remains dry and practically undeformed due to the high stiffness of the loess. As a combined result, the shape of the drillhole is severely distorted and the majority of the grout/soil interface will provide extremely low bond strengths. For these loessial soils, experience has shown that adequate

bond strengths can be achieved if high-pressure grouting or regrouting under gravity is applied. However, higher costs and lower installation rates are expected for these conditions.

Table 4.1: Drilling Methods and Procedures.

Drill Rig Type	Drilling Method	Open Hole?	Cased or Auger-Cast?	Drillhole Diameters (mm/in.)	Drill Bit Types	Cuttings Removal	Comments
Auger	Lead Flight Kelley-Bar Driven	Yes	No	100–300/ 4–12	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock.
	Sectional Solid-Stem	Yes	No				
	Sectional Hollow-Stem	Yes	Yes			Mechanical (air support)	
	Continuous Flight Solid-Stem	Yes	No			Mechanical	
	Continuous Flight Hollow-Stem	Yes	Yes			Mechanical (air support)	
Rotary	Single-Stem Air Rotary	Yes	No	100–200/ 4–8	Button, Roller, Drag, etc.	Compressed air	Hydraulic rotary methods for drilling competent soils, rock, or mixed ground conditions (pneumatic hammers available).
	Duplex Air Rotary	Yes	Yes				
	Sectional Solid-Stem Augers	Yes	No	100–300/ 4–12	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock.
	Sectional Hollow-Stem Augers	Yes	Yes			Mechanical (air support)	
Air Track	Single-Stem Air Rotary	Yes	No	100–300/ 4–12	Button, Roller, Drag, etc.	Compressed Air	Pneumatic rotary methods for drilling non-caving competent soils or rock.

Modified from Elias and Juran (1991).

4.3.5 Drainage Installation

The migration of groundwater towards the excavation must be prevented. Therefore, conventional methods of surface water control and drainage are necessary during construction. These methods include intercepting ditches at the top of the excavation. Additionally, vertical geocomposite strip drains are installed behind the wall facing to prevent groundwater pressure to build up. The drain strips are secured against the excavation face and are placed with the geotextile side against the ground. During shotcrete application, the contamination of the geotextile side with shotcrete must be avoided to prevent loosing the flow capacity of the drains. Strip drains must be spliced at the bottom of each excavation lift and must have at least a 300 mm (12 in.) overlap such that the water flow is not impeded.

The groundwater collected at strip drains is removed by a series of footing drains at the bottom of the excavation. The footing drain consists of a trench at the bottom of the excavation, which is filled with aggregate free of fines and has a PVC slotted collection pipe. The drainage geotextile must envelope the footing drain aggregate and pipe and conform to the dimensions of the trench. Additionally, weep holes can be installed through the wall facing at the lower portions of the wall. In special situation when the groundwater behind the proposed soil nail wall is high, conventional, deeper horizontal pipe drains are necessary. Additional aspects of the design of the drainage system are presented in Chapter 6.

4.3.6 Temporary Wall Facing Construction

4.3.6.1 General

Temporary wall facings for soil nail wall applications are usually constructed using shotcrete. The thickness of the temporary shotcrete facing is typically between 75 and 100 mm (3 and 4 in.). Shotcrete provides a continuous supporting layer over the excavated face that can also serve to fill voids and cracks on the excavated face. Temporary shotcrete applications have been constructed using both WWM or fiber reinforcement and bars. WWM is the preferred method among contractors because it requires less time to install while the excavated face is unsupported. A shotcrete facing for a wall under construction is shown in Figure 4.9.

4.3.6.2 Shotcrete Application

For shotcrete mixtures, there are two opposing requirements: “shootability” and “pumpability.” Shootability is the ability of a mix to stick to a surface, build up thickness, and resist sloughing. Pumpability is the ability of a mix to flow like a viscous fluid. For shooting, a high flow resistance and high viscosity are ideal, whereas for pumping, a low flow resistance and low viscosity are ideal. Once it is applied, a shotcrete mix with high flow resistance and high viscosity will tend to “stick” and remain as layers of the facing are formed. Contractors want high shootability to achieve the desired thickness in one pass. With the proper mix design, shootability to a thickness of 300 mm (12 in.) can readily be achieved without sloughing or sag cracks below rebar. Methods for shotcrete mix design can be found in existing literature (ACI, 1994, 1995a, 1995b).

Two types of shotcrete methods are commonly used: dry mix and wet mix. In the dry mix method, the aggregate and cement are blended in the dry and fed into the shotcrete gun while the mix water is added at the nozzle. Depending on their features, admixtures can be added at the mix plant or with the water. The addition of water at the nozzle allows the plasticity of the shotcrete to be adjusted at the nozzle, if required. In the wet mix method, the aggregate, cement, water, and admixtures are mixed in a batch plant and conveyed to the nozzle by a hydraulic pump. The plastic mix is applied at higher velocities by compressed air.

Both shotcrete methods produce a mix suitable for wall facings. Dry mix and wet mix shotcrete use a water-cement ratio of about 0.4 and produce roughly the same mix quality, although shotcrete obtained with the wet mix process yields a slightly greater flexural strength. Keeping water cement ratios at about 0.4 and using air entrainment, which is difficult with the dry-mix process, enhances the durability of shotcrete. Low water-cement (W/C) ratios (i.e., W/C $<$ 0.45) result in high

strength, high durability, and low permeability as long as proper in situ compaction (i.e., elimination of entrapped air in the shotcrete) is achieved.

Source: Porterfield et al. (1994).

Figure 4.9: Shotcrete Temporary Facing.

Two of the most significant factors determining shotcrete quality and durability are cement content and in situ density. Shotcrete mixtures have high cement factors, typically 350 to 410 kg of cement per m^3 (600 to 700 pounds/yd^3) of mixture. The amount of cement determines the pumpability and shootability of the mixture. The in situ density of the mixture is controlled by the amount of air that is entrained within the mixture. Increasing the air content will proportionally decrease strength (approximately 5 percent compressive strength reduction per 1 percent of air). Unless it is needed for freezing and thawing durability, the use of entrained air is not necessary. Although the requirements of durability for temporary facing can be relaxed in relation to the requirements for permanent facing, the requirements are usually similar.

Steel fiber reinforcement has been added to shotcrete as part of a wet mix to increase ductility, toughness, and impact resistance. Fibers tend to reduce the shotcrete brittleness and thereby reduce crack propagation, but they have little effect on compressive strength and produce only a modest increase in flexural strength. Wet mix is often preferred for the construction of shotcrete facing walls because:

- wet mix yields higher production of fresh shotcrete, typically 6 to 8 m^3/hour (2.2 to 3.0 yd^3/hour) versus 4 to 6 m^3/hour (1.5 to 3.0 yd^3/hour) for dry mix;
- the shotcrete rebound (i.e., loss of material due to lack of “stick”) for a wet mix is typically only about 5 percent, compared to 15 percent for a dry mix;
- there is no need to add water at the nozzle, as in the case of a dry mix, thus it is less dependent on the nozzle operator’s experience;

- equipment (e.g., concrete pump) is more readily available because shotcrete gun and moisturizer are not needed as with dry mix; and

- supply of ready-mix concrete from commercial batch plants is readily available and convenient.

Although not necessary for adequate performance of a shotcrete facing, tolerances typically used for formed concrete facing (which may represent a significant aesthetic issue) cannot be easily achieved in shotcrete. Suggested tolerances for temporary shotcrete facings should be on the order of 1.5 to 2 times the tolerances normally specified for CIP.

4.3.6.3 Shotcrete Reinforcement

Welded wire mesh is commonly used as reinforcement for temporary facing but occasionally is also used in permanent facing. The cross-sectional area and mesh opening of the WWM are selected to satisfy structural requirements (i.e., flexural and punching shear capacities) and constructibility constraints. Typical dimensions for WWM are summarized in Table A.2 (Appendix A). WWM is available in the United States in rolls or sheets 1.5 to 2.5 m (4.5 to 7.5 ft) wide. The selected WWM must have a width that is consistent with the excavation lift height (equivalent to the vertical nail spacing), plus an overlap of at least 0.2 m (8 in.). For example, if the selected nail vertical spacing were 1.5 m (4.5 ft), the ideal width of the WWM panel would be approximately 1.70 m (5.5 ft). WWM is commonly available in 1.5- to 2.4-m (5- to 8-ft) wide sheets and rolls. Sheets up to 3.6-m (12-ft) wide are also available. Wider sheets can be manufactured on request. Sheets can be commonly obtained in 3.8, 4.6, 6.1, and 7.6 m (12.5, 15, 20, and 25 ft) in length. On request, WWM sheets can be manufactured up to 12 m (40 ft) in length.

The dimensions of the WWM (i.e., bar size and spacing) are evaluated as part of the soil nail wall design. The design engineer should check the availability of different WWM widths near the project site. In some cases when the availability of WWM widths is restricted locally, the design engineer may decide to modify the vertical nail spacing as a function of WWM width availability.

Additional reinforcement ("waler bars") may be placed around nail heads to provide additional flexural capacity at these locations. The waler bars consist of two vertical (one bar at each side of the nail head) and two horizontal bars. Typical characteristics of steel rebar are presented in Table A.3 (Appendix A).

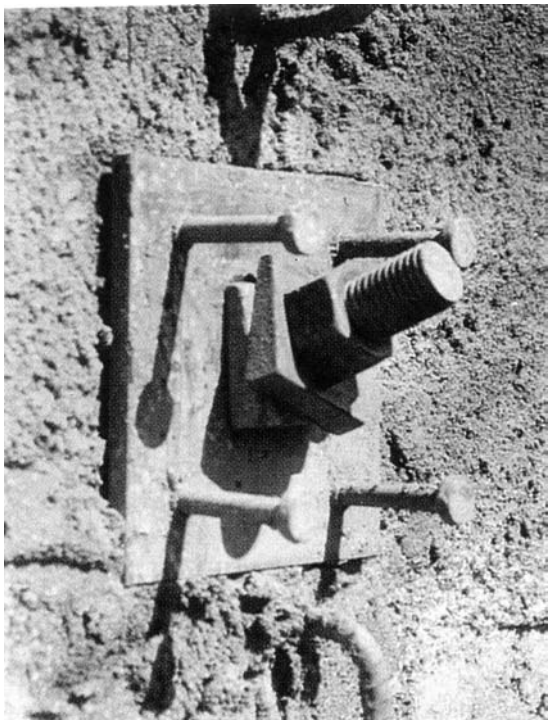
4.3.7 Permanent Wall Facing Construction

4.3.7.1 Introduction

Several methods to provide a permanent facing have been used by contractors. The following sections describe the most common applications, including reinforced shotcrete, CIP reinforced concrete, and precast concrete.

4.3.7.2 Reinforced Shotcrete

The thickness of permanent facing consisting of reinforced shotcrete is typically between 150 and 300 mm (6 and 12 in.). This thickness does not include the thickness of the temporary facing. The permanent facing can be applied at the same time as the lift is excavated or can be applied later after the final lift of the temporary facing is completed and advance in phases from the bottom. The shotcrete facing is reinforced with WWM or rebar mesh. The final wall facing thickness is obtained by applying successive layers of shotcrete, each layer 50- to 100-mm (2- to 4-in.). In order to achieve a good connection between the permanent facing and the nail bar, the nail head must be fully encapsulated in the permanent shotcrete facing. Headed studs are welded to the bearing plate to provide this connection (Figure 4.10).



Source: Porterfield et al. (1994).

Figure 4.10: Headed-Studs Welded to Bearing Plate.

4.3.7.3 Cast-in-Place Facing

In general, the requirements for shotcrete mixes and WWM reinforcement for permanent facing are similar to those of shotcrete for temporary facing. Permanent facings with reinforced shotcrete usually do not require the use of waler bars. The use of epoxy coated WWM is highly recommended for corrosion protection. As in the case of temporary facing, suggested tolerances for permanent shotcrete facings should be on the order of 1.5 to 2 times the tolerances normally specified for CIP concrete. Typical tolerances for shotcrete require that the thickness of shotcrete remain within 15 mm (0.6 in.) if troweled and 30 mm (1.2 in.) if left as shot of specified dimensions. In addition, the planeness of the finished face surface should be such that the gap under a 3-m (10-ft) straightedge should remain within 15 mm (0.6 in.) and 30 mm (1.2 in.) of

specified dimensions if troweled or left as shot, respectively. Additional tolerances are included in the suggested technical specifications (Appendix E).

The reinforcement of permanent facing using CIP concrete typically consists of a mesh (standard reinforcing bars) and occasionally waler bars placed over the nail head. Reinforcement is placed approximately at the center of the facing section thickness. Because bonding between temporary shotcrete and permanent CIP concrete facings cannot be assured, the temporary facing is typically disregarded as a resisting element in the section design. A variety of finishes can be implemented by using commercially available form liners, as shown in Figure 4.11.

The advantage of the CIP reinforced concrete is that the finish is more aesthetically pleasing and the quality of the concrete tends to be more homogeneous. The main disadvantage is the need for formwork and potentially longer construction time for facing installation. The use of reinforced shotcrete as permanent facing has the potential benefit of cost savings and efficiency as the same shotcrete equipment used for the temporary facing can be utilized. A major limitation of this technique is that the conventional finish of shotcreted walls is typically relatively rough and may not meet aesthetic requirements for a finished wall face, as discussed below.

Soil nail walls to be constructed in heavily trafficked areas may pose special aesthetic requirements. As the typically uneven as-shot finish of the permanent shotcrete facing may not meet these requirements, other finishes might be necessary. Wood, steel, or rubber tools can be used on the as-shot shotcrete surface to condition the surface and achieve the required aesthetic finish. Finished shotcrete surfaces can also be colored with pigmented sealers. In exceptionally sensitive environments, where a minimum aesthetic impact is desired to the surrounding landscape, the permanent shotcrete layer may be thickened and can be sculpted while the shotcrete is fresh. Ultimately, the final surface is stained for a natural look. The final result is an irregular shotcrete surface that matches the surrounding landscape.

4.3.7.4 Precast Concrete Facing

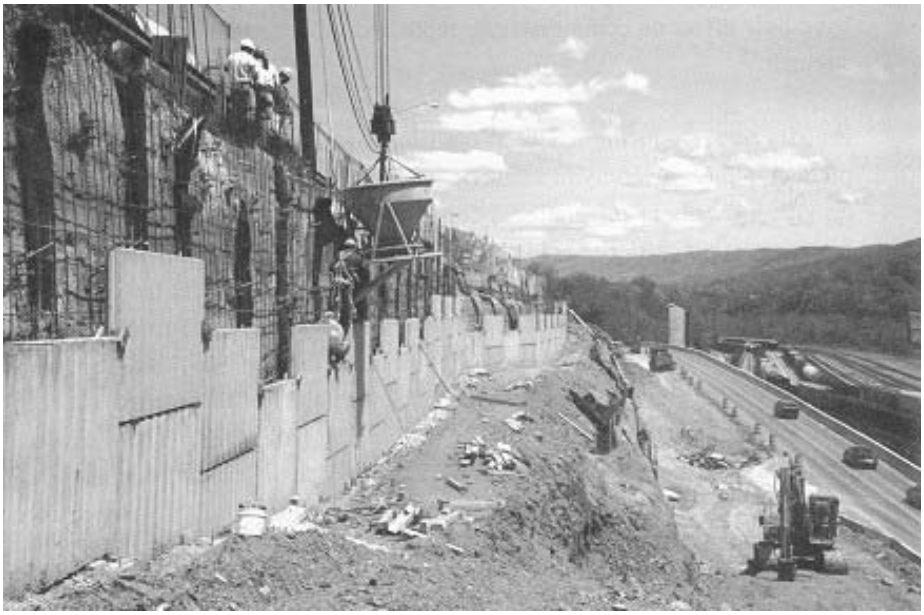
Precast concrete facing has been used in permanent applications to meet a variety of aesthetic, environmental, and durability criteria. A project using this method is shown in Figure 4.12. Precast facings also provide a means of integrating a continuous drainage blanket behind the facing and a frost protection barrier in cold climates.

The panels are either small segmental (i.e., nonstructural) elements or full-height tilt-up (i.e., structural) elements. Small segmental panels resist smaller, localized loads than structural elements, which can resist loads globally. If non-structural elements are used for the permanent facing, the temporary facing requires adequate strengthening to resist the design loads.

Prefabricated panels can be attached to the nails or nail head by a variety of devices. With some systems, the connection is often made at the corner of each large precast panel by using truncated wedging heads between adjoining panels at each nail location. Alternatively, nails can be attached to vertical pre-fabricated or cast-in-place columns; panels are then inserted between columns, the same way as lagging is inserted in soldier beam and lagging wall systems.

Figure 4.11: Cast-in-Place Facing.

Because connection details require a high degree of precision in locating nails, templates are used to ensure accuracy. Some projects may have architectural or aesthetic requirements and call for the use of special nonstructural pre-cast panels, masonry stone, or masonry blocks on top of the permanent shotcrete or CIP facing.



Source: Elias et al. (2001).

Figure 4.12: Precast Panel Facing.

Alternatively, the nail plate may be provided with shear stud connectors or hoop-shaped rebars cast at the back of the precast panels. The space between them is then filled with low-strength concrete to form the connection. This method has been used with some success in the eastern United States. Patents may exist over some connection details developed by various specialty contractors.

CHAPTER 5 ANALYSIS OF SOIL NAIL WALLS

5.1 INTRODUCTION

The primary purpose of this chapter is to provide principles that govern the analysis and design of soil nail walls. The chapter is organized to first introduce the technical concepts related to the mechanisms underlying soil nail wall response to construction and operation. Following this introduction, specific topics related to analysis and design are introduced, starting with a presentation of the two specific limit states that must be considered by the designer, namely, the strength limit states and service limit states. This is followed by a description of potential failure modes for soil nail walls. This discussion is followed by a presentation of the governing equations that are used for analysis and design of soil nail walls in consideration of each potential failure mode. The chapter then introduces and compares calculations resulting from SNAIL and GOLDNAIL, two of the most widely used computer programs in the United States. The chapter concludes by recommending minimum factors of safety to be used and by providing guidelines for loading conditions typically used in the design of soil nail walls.

5.2 LOAD TRANSFER CONCEPT IN SOIL NAIL WALLS

Prior to discussing the analysis of soil nail walls, the underlying mechanisms in the response of soil nail walls during the various stages of construction are briefly introduced. The following wall response and load transfer mechanisms take place during a conventional soil nail construction:

- Soil excavation is initiated from the ground surface and the Excavation Phase 1 is completed (Figure 5.1). Because of the soil ability to stand unsupported, the upper portion of the soil behind the excavation is stable (or at least marginally stable) before the first row of nails (Nails 1) is installed. Soil strength is mobilized along the uppermost potential critical failure surface to allow the unsupported soil wall to stand.

- As Nails 1 and the temporary facing are installed, some load derived from the deformation of the upper soil is transferred to these nails through shear stresses along the nails and translate into and axial forces. The top portion of Figure 5.1 shows schematically the axial force distribution in Nails 1 at the end of excavation Phase 1. At this point, the temporary facing supports the excavation surface and provides connectivity between adjacent nails in row of Nails 1.

- As excavation proceeds to Excavation Phase 2, the uppermost and the unsupported portions of the soil nail wall deforms laterally. At this point, another potential sliding surface, one originated from base of Excavation Phase 2 is formed (Figure 5.1). The critical failure surface at this excavation level is different than that at the previous excavation level.

- Nails 2 are then installed. Subsequently the temporary facing between the bottom of excavation Phases 1 and 2 is installed and integrated to the facing constructed in Phase 1. Subsequent movements of the soil above the Phase 2 depth will cause additional loads to be

transferred to Nails 1 and generate loads in Nails 2. Note the increased nail force distribution for Nails 1 at the end of excavation Phase 2.

- To provide global stability, the soil nails must extend beyond the potential failure surface. As lateral deformation increases due to subsequent excavation, additional shear stresses along the soil nail/soil interface and axial forces of the previously installed nails are mobilized. As the depth of excavation increases, the size of the retained soil mass increases, as shown in Figure 5.1.

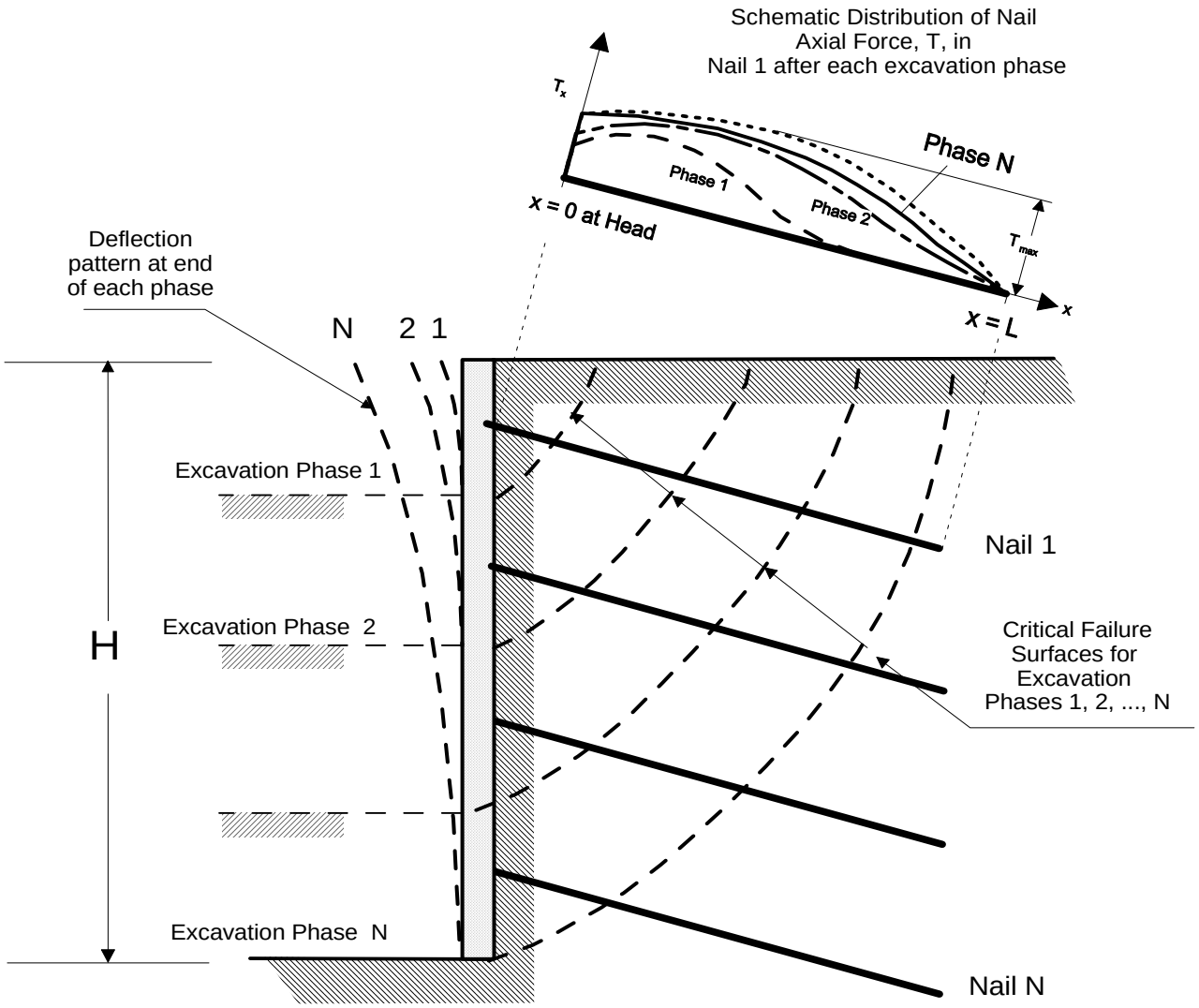


Figure 5.1: Potential Failure Surfaces and Soil Nail Tensile Forces.

- As the size of the retained zone increases, the stresses at the soil/nail interface and the axial forces in the nails increase. The induced tensile stresses are transferred behind the retained zone in an anchorage effect. These stresses ultimately tend to stabilize the potentially sliding mass.

- While the tensile force in the intermediate and lower nails may increase as the excavation depth increases, the tensile force in some of the upper nails may decrease due to load redistribution. For example, the upper portion of Figure 5.1 shows schematically that the axial force distribution for Nails 1 at the end of the last excavation Phase N does not exhibit the largest values.

- As the critical failure surface becomes deeper and larger, the contribution of the upper nails to the stabilization of this larger sliding mass diminishes. In some cases, upper nails may be entirely ineffective in the assessment of deep critical failure surfaces. However, the upper nails should not be considered superfluous, because they contribute to the stability during earlier stages of excavation and help reduce lateral displacements.

The analysis of soil nail walls must consider both "during construction" and "post construction" loading conditions to establish the most critical case at each soil nail level. The most critical situation may arise after the wall is completed due to a combination of long-term design loads (e.g., dead load, live load, and traffic) and extreme loads (e.g., earthquake). In other situations, the most critical case may occur during construction when the then lowest excavation surface remains temporarily unsupported and the soil nails and shotcrete are not yet installed (Figure 5.2). These critical short-term loading conditions can be exacerbated by temporary seepage conditions. The selection of safety factors for temporary excavation conditions that are lower than those adopted for long-term conditions is acceptable and commonly done because the duration of the exposure to this failure risk is much shorter. Safety factors for both short- and long-term loading conditions are summarized in Section 5.9.

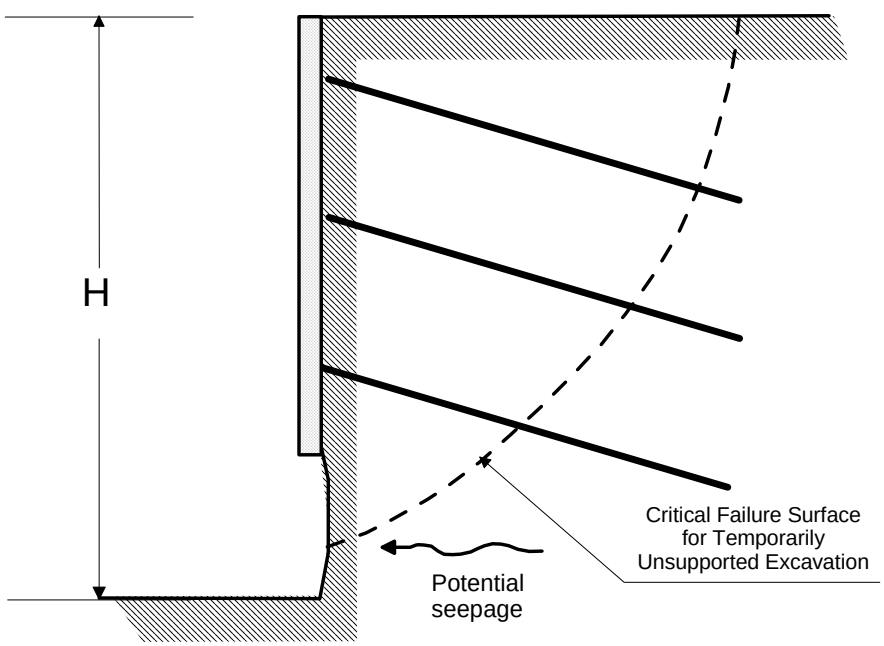


Figure 5.2: Potential Critical Stability During Construction.

5.3 LIMIT STATES

The analysis and design of soil nail walls must consider two distinct limiting conditions: Strength Limit States and the Service Limit States.

- *Strength limit states.* These limit states refer to failure or collapse modes in which the applied loads induce stresses that are greater than the strength of the whole system or individual components, and the structure becomes unstable. Strength limit states arise when one or more potential failure modes are realized. The design of a soil nail wall should ensure that the system is safe against all of the potential failure conditions presented in Figure 5.3 and classified as:

  - external failure mode;
  - internal failure mode; and
  - facing failure mode.

A discussion of each of these strength limit states is presented in Sections 5.4 to 5.6, respectively.

- *Service limit states.* These limit states refers to conditions that do not involve collapse, but rather impair the normal and safe operation of the structure. The major service limit state associated with soil nail walls is excessive wall deformation. A discussion regarding deformations of soil nail wall systems and the acceptance criteria for these deformations are presented in Section 5.7. Other service limit states, which are beyond the scope of this document, include total or differential settlements, cracking of concrete facing, aesthetics, and fatigue caused by repetitive loading.

There are other important factors that, if not properly addressed during design, can result in problems during operation. Two of these additional factors are drainage of the soil behind the wall, and corrosion of the soil nail bar and/or other metallic components. Aspects related to the design of drainage systems are discussed in Section 5.11.1. Corrosion is a long-term effect that has to be considered in relation to strength limit states, as corrosion affects the tensile capacity of soil nails. Corrosion of soil nail bars can lead to excessive deformations and, in an extreme case, can cause the eventual collapse of the system. The conditions that define the ground as corrosive were presented in Section 3.9. Methods to provide corrosion protection to soil nail wall components are discussed in Appendix C.

5.4 EXTERNAL FAILURE MODES

5.4.1 Introduction

External failure modes refer to the development of potential failure surfaces passing through or behind the soil nails (i.e., failure surfaces that may or may not intersect the nails). For external failure modes, the soil nail wall mass is generally treated as a block. Stability calculations take into account the resisting soil forces acting along the failure surfaces to establish the equilibrium of this block. If the failure surface intersects one or more soil nails, the intersected nails contribute to the stability of the block by providing an external stabilizing force that must be added to the soil resisting forces along the failure surface. Within this framework, the three failure modes identified

by Byrne (1998) as “external” (i.e., failure surfaces not intersecting the nails), “internal” (i.e., failure surfaces intersecting all nails), and “mixed” (i.e., failure surfaces intersecting some nails) are classified as external failure modes in this document.

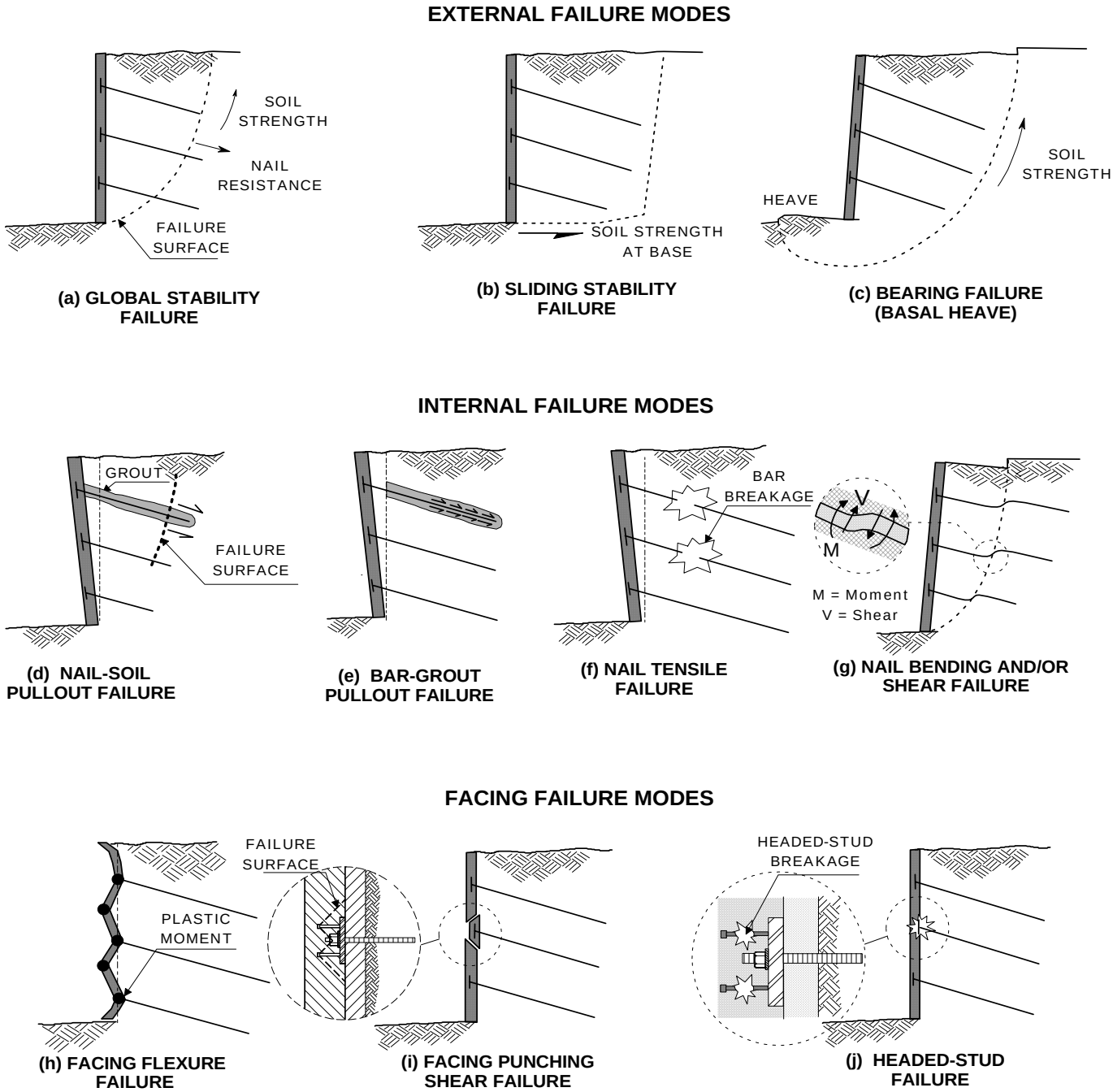


Figure 5.3: Principal Modes of Failure of Soil Nail Wall Systems.

The evaluation of external stability is an important aspect in the design of soil nail walls because the magnitude and consequence of failure can be significant. External stability analyses are performed

to verify that the proposed soil nail wall is able to resist the destabilizing forces induced by the excavation, service loads, and extreme loads (e.g., seismic) for each of the potential failure modes. Factors that control external stability include wall height; soil stratigraphy behind and under the wall; width of the nailed zone (i.e., soil nail lengths); and soil, nail, and interface strengths. The following external failure modes (Figures 5.3a–c) are considered in the analysis of soil nail wall systems:

- global failure mode;
- sliding failure mode (shear at the base); and
- bearing failure mode (basal heave).

Each of these external failure modes is discussed in the following three sections. Seismic impacts on global and sliding failure modes are also presented in this section. It is noted that if limit-equilibrium based computer programs (discussed subsequently in Section 5.8.2) are used for design of soil nail walls (e.g., SNAIL), the explicit consideration of sliding and bearing capacity failure modes may be unnecessary. Specifically, in searching for the most critical failure surface, computer programs used for analysis and design of soil nail walls routinely consider failure surfaces that result from these two failure modes, as shown in Figures 5.3b and 5.3c. However, for the sake of completeness, the procedures for sliding and bearing capacity are presented in this section.

5.4.2 Global Stability

5.4.2.1 Resisting Mechanisms

Global stability refers to the overall stability of the reinforced soil nail wall mass. As shown in Figure 5.2a, the slip surface passes behind and beneath the soil nail wall system. In this failure mode, the retained mass exceeds the resistance provided by the soil along the slip surface and the nails, if intersected.

5.4.2.2 Limit-Equilibrium in Global Stability Analysis

The global stability of soil nail walls is commonly evaluated using two-dimensional limit-equilibrium principles, which are used for conventional slope stability analyses. In limit-equilibrium analysis, the potentially sliding mass is modeled as a rigid block, global force and/or moment equilibrium is established, and a stability factor of safety that relates the stabilizing and destabilizing effects is calculated. As with traditional slope stability analyses, various potential failure surfaces are evaluated until the most critical surface (i.e., the one corresponding to the lowest factor of safety) is obtained. Different assumptions and numerical procedures give rise to different stability methods. In particular, different shapes of the failure surface behind the soil nail wall have been considered in various methods to analyze the global stability of soil nail walls. Some of these shapes of the failure surfaces include: (1) planar (Sheahan and Oral, 2002), (2) bi-linear with a two-wedge slipping mass (e.g., German method as in Stocker et al., 1979; Caltrans, 1991), (3) parabolic (Shen et al., 1981a), (4) log spiral (Juran et al., 1990), and (5) circular (Golder, 1993). Numerical comparisons among different methods show that differences in the geometry of the failure surface do not result in a significant difference in the calculated factors of safety (e.g., Long et al., 1990).

Similarly, comparable nail lengths are obtained if different shapes of the failure surfaces are assumed for the same target factor of safety. Most of these comparisons are based on soils generally exhibiting frictional resistance.

The main shortcoming of limit equilibrium-based methods is that they do not provide a prediction of deformations, nor do they consider the deformation required to mobilize the resisting forces in the soil and soil nails. These methods cannot provide a thorough description of the contribution of each soil nail to global stability based on the deformation pattern behind the wall . Estimates of deformation can be obtained using numerical techniques (e.g., finite element and finite difference methods); however, these methods are rarely used in the design of soil nail walls. More commonly, semi-empirical methods based on previous experience are used to assess deformation in relation to assumed serviceability limits for soil nail walls. Guidance related to acceptable deformations of soil nail walls is provided later in this chapter.

5.4.2.3 Simplified Global Stability Analysis

To illustrate the elements of a global stability analysis for soil nail walls, a simple, single-wedge failure mechanism is shown in Figure 5.4.

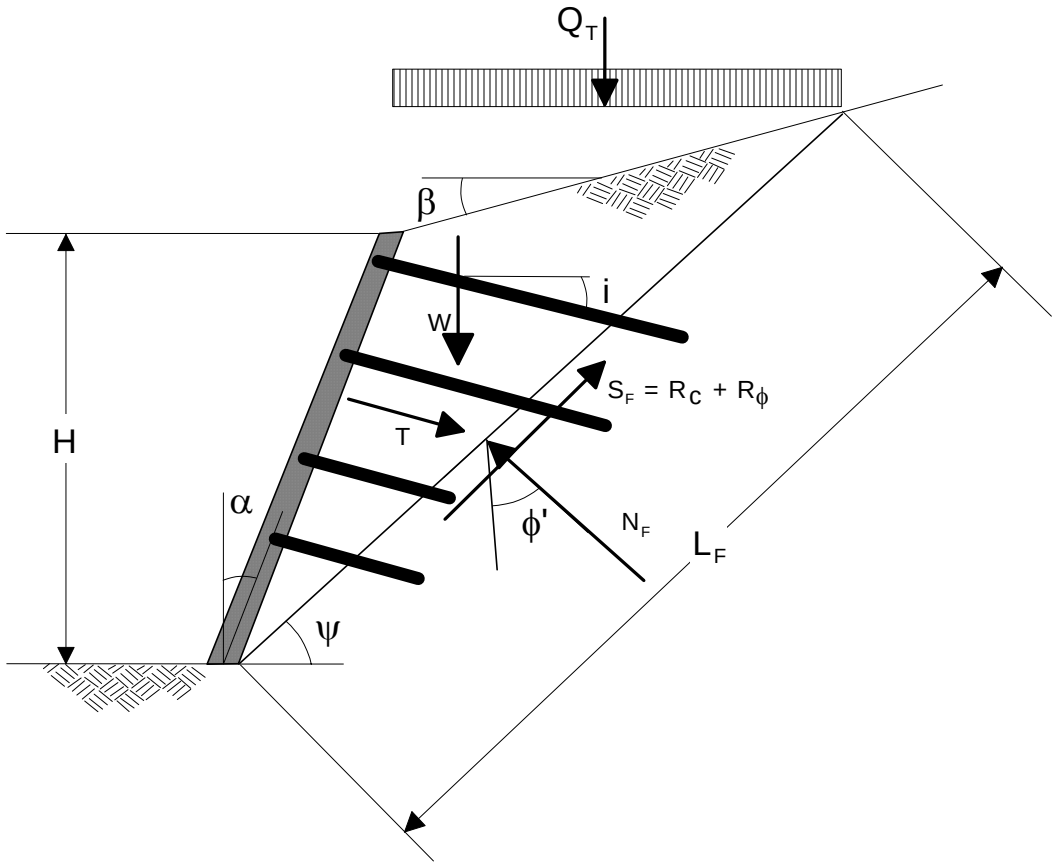


Figure 5.4: Global Stability Analysis of Soil Nail Wall using a Single-Wedge Failure Mechanism.

where:

α = wall face batter angle (from vertical);
β = slope angle;
ϕ' = soil effective angle of internal friction;
c' = soil effective cohesion;
ψ = inclination of failure plane;
i = nail inclination;
L_F = length of failure plane;
W = weight of sliding mass;
Q_T = surcharge load;
T_{EQ}= equivalent nail force;
N_F = normal force on failure surface;
S_F = shear force on failure surface;
R_c = cohesive component of S_F; and
R_ϕ = frictional component of S_F.

The destabilizing forces consist of the driving components of the weight (W) and the surcharge loads (Q). The stabilizing forces along the failure surface are the shear force (S_F) and the equivalent nail tensile force (T_{EQ}). It is noted that the force T_{EQ} is a resultant force that combines the effect of all nails installed to that depth H. The factor of safety against global failure (FS_G) is expressed as the ratio of the resisting and driving forces, which act tangent to the potential failure plane:

$$FS_G = \frac{\sum \text{ resisting forces}}{\sum \text{driving forces}} \qquad \text{(Equation 5.1)}$$

The normal and tangent forces on the failure plane are:

$$\sum \text{Normal Forces} = (W + Q_T)\cos\psi + T_{EQ}\cos(\psi - i) - N_F = 0 \qquad \text{(Equation 5.2)}$$

$$\sum \text{Tangent Forces} = (W + Q_T)\sin\psi - T_{EQ}\sin(\psi - i) - S_F = 0 \qquad \text{(Equation 5.3)}$$

where:

$$S_F = R_c + R_f = c_m L_s + N_F \tan\phi_m \qquad \text{(Equation 5.4)}$$

$$\tan\phi_m = \frac{\tan\phi'}{FS_G} \qquad \text{(Equation 5.5)}$$

$$c_m = \frac{c'}{FS_G} \qquad \text{(Equation 5.6)}$$

and ϕ_m is the mobilized friction angle, and c_m is the mobilized cohesion. A single global factor of safety is used for the cohesive and frictional strength components of the soil (c' and $\tan\phi'$, respectively). However, it is possible to select different safety factors for each strength component.

The simplistic analysis presented above only considers force equilibrium. More rigorous analysis methods allow establishing simultaneously moment and force equilibrium equations. Although it is possible to extend the single-wedge equations presented above and consider both two- or three-part wedge analyses, these types of hand-calculations to evaluate the global stability of soil nail systems are rarely performed in practice. Typically, designers use computer programs to analyze and design soil nail systems. The simple model shown in Figure 5.4 and presented in the above equations may be used to perform an independent verification of the computer's solution.

5.4.2.4 Procedures in Global Stability Analysis

A global stability analysis can be used to complete either (or both) of the following two tasks related to the analysis of soil nail walls:

1) calculate the critical (minimum) factor of safety FS_G of the sliding mass for a given soil nail length pattern; or
2) determine the required force T in all nails that will yield a selected target factor of safety against global failure.

The first task is to verify the stability of the soil nail wall. This task is similar to a conventional slope stability analysis, in which the most critical failure surface and thus the lowest factor of safety are identified. This is the most common practice for computer analysis for which the nail lengths and pullout capacities are assumed. Tensile forces in the soil nail are obtained as part of the results of these calculations.

With similar steps but different goals, the second task is to calculate the required force T and design the size of the soil nails. In this task, various nail length patterns are tried until the selected target minimum FS_G is achieved. This type of analysis is commonly iterative. The calculated required tensile force allows the soil nail length and diameter to be determined.

Global stability analyses are performed using computer programs specifically developed for the design of soil nail walls. The two computer programs most commonly used in the United States for the analysis and design of soil nail walls are SNAIL and GOLDNAIL. The main features of these programs are described in a later section. These programs can consider failure surfaces that are more complex than the simple planar, single-wedge. SNAIL uses two-part planar wedges; GOLDNAIL uses circular failure surfaces that consider multiple slices in lieu of wedges. These programs are similar, in many respects, to general slope stability computer programs (e.g., search routines, closed form force/moment equilibrium equations, etc.). However, computer programs dedicated to soil nail design include the iterative and interactive design of the soil nail length and the consideration of other failure modes (e.g., soil nail tensile force and facing punching shear failure).

The factors of safety for global stability of soil nail walls are selected based on the nature of the structure (i.e., temporary and permanent), service condition (during construction and in service), and loading type (static or seismic loads). In general, the recommended minimum factors of safety for global stability of soil nail walls are comparable to those used in conventional slope stability analyses that are performed in various areas of geotechnical practice, including natural or man-

made slopes, embankments. Section 5.9 presents discussions of the basis of values of safety factors in soil nail wall systems and provides recommended minimum values of FS_G, as well as safety factors for other failure modes.

5.4.3 Sliding

Sliding stability analysis considers the ability of the soil nail wall to resist sliding along the base of the retained system in response to lateral earth pressures behind the soil nails. Sliding failure may occur when additional lateral earth pressures, mobilized by the excavation, exceed the sliding resistance along the base (Figure 5.5).

Concepts similar to those used to assess sliding stability of gravity retaining structures (in which Rankine or Coulomb theories of lateral earth pressures are used) can be applied to assess the sliding stability of a soil nail wall system. Again, the soil nail wall system is modeled as a rigid block against which lateral earth forces are applied behind the retained soil. The rigid block here is defined by a nearly horizontal surface through the base of the wall (or slightly below the base, if a weak, horizontal seam of soil is present), extends behind the nails, and exits with a steep angle at the ground surface behind the reinforced zone (Figure 5.5). It is assumed that the displacements of the soil block along its base are large enough to mobilize the active pressure behind the block. The factor of safety against sliding (FS_{SL}) is calculated as the ratio of horizontal resisting forces (ΣR) to the applied driving horizontal forces (ΣD) as follows:

$$FS_{SL} = \frac{\sum R}{\sum D} \qquad \text{(Equation 5.7)}$$

where:

$$\sum R = c_b\ B_L + \left(W + Q_D + P_A \sin\beta\right) \tan \phi_b \qquad \text{(Equation 5.8)}$$

$$\sum D = P_A\ \cos \beta \qquad \text{(Equation 5.9)}$$

The active lateral earth force (P_A) is defined as:

$$P_A = \frac{\gamma H_1^2}{2} K_A \qquad \text{(Equation 5.10)}$$

The terms in the equations above and in Figure 5.5 are identified as follows:

H = wall height;
ΔH = slope rise up to bench (if present);
β = backslope angle;
β_{eq} = equivalent backslope angle [for broken slopes $\beta_{eq} = \tan^{-1}(\Delta H/H)$, for infinite slopes $\beta_{eq} = \beta$];
α = face batter angle;
θ = inclination of wall face from horizontal (i.e., $\theta = \alpha + 90^\circ$);
c_b = soil cohesion strength along the base;
B_L = length of the horizontal failure surface where c_b is effectively acting;
W = weight of soil nail block;
Q_D = permanent portion of total surcharge load Q_T;

ϕ'_b = effective angle of internal friction of the base (remolded or residual values may be needed if significant movement takes place);

ϕ' = effective friction angle of soil behind soil nail block;

δ = wall-soil interface friction angle [for a broken slope, $\delta = \beta_{eq}$, for infinite slope, $\delta = \beta$];

γ = total unit weight of soil mass;

H_1 = effective height over which the earth pressure acts [$H_1 = H + (B + \tan\alpha)\tan\beta_{eq}$]; and

K_A = active earth pressure coefficient for soil behind the soil nail wall system.

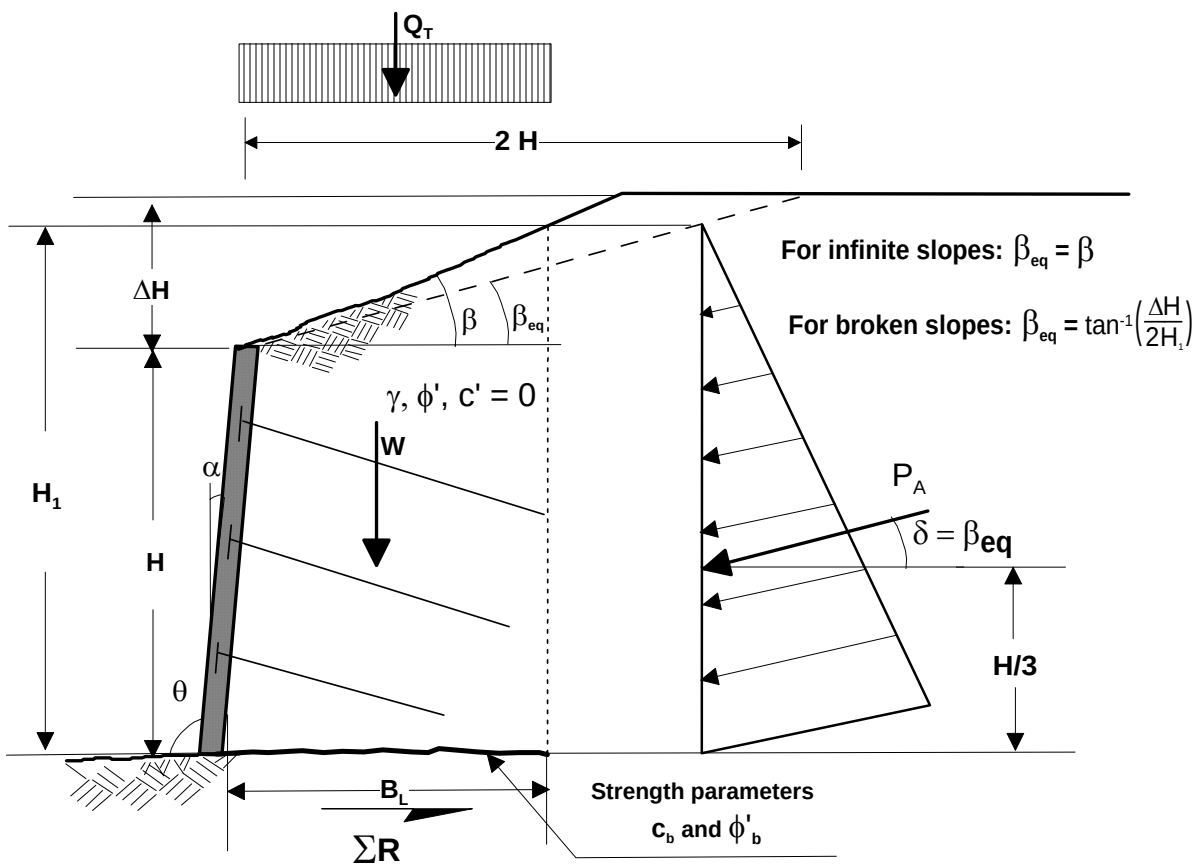


Figure 5.5: Sliding Stability of a Soil Nail Wall.

The active earth pressure coefficient, K_A, can be obtained using the formulation derived from the general Coulomb theory or the Rankine theory for cohesionless soil (assuming that the soil behind the soil nail wall behaves in accordance with $c' = 0$ in the long-term loading condition).

According to the Coulomb theory:

$$K_A = \frac{\sin^2(\theta + \phi')}{\sin^2\theta \, \sin(\theta - \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\,\sin(\phi' - \beta)}{\sin(\theta - \delta)\,\sin(\theta + \beta)}}\right]^2} \qquad \text{(Equation 5.11)}$$

where the parameters have been defined previously.

The Rankine theory provides, for walls with face batter angles $\alpha < 8$ degrees and dry, sloping ground behind the wall, the coefficient of active earth pressure as:

$$K_A = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}} \right] \qquad \text{(Equation 5.12)}$$

For the simple case of a vertical wall (i.e., $\alpha = 0$ or $\theta = 90^\circ$), with dry, horizontal ground (i.e., $\beta = 0$) behind the wall, and no shear stresses on the wall-soil interface (i.e., $\delta = 0$), the Rankine theory provides the coefficient of active earth pressure simply as the well-known expression:

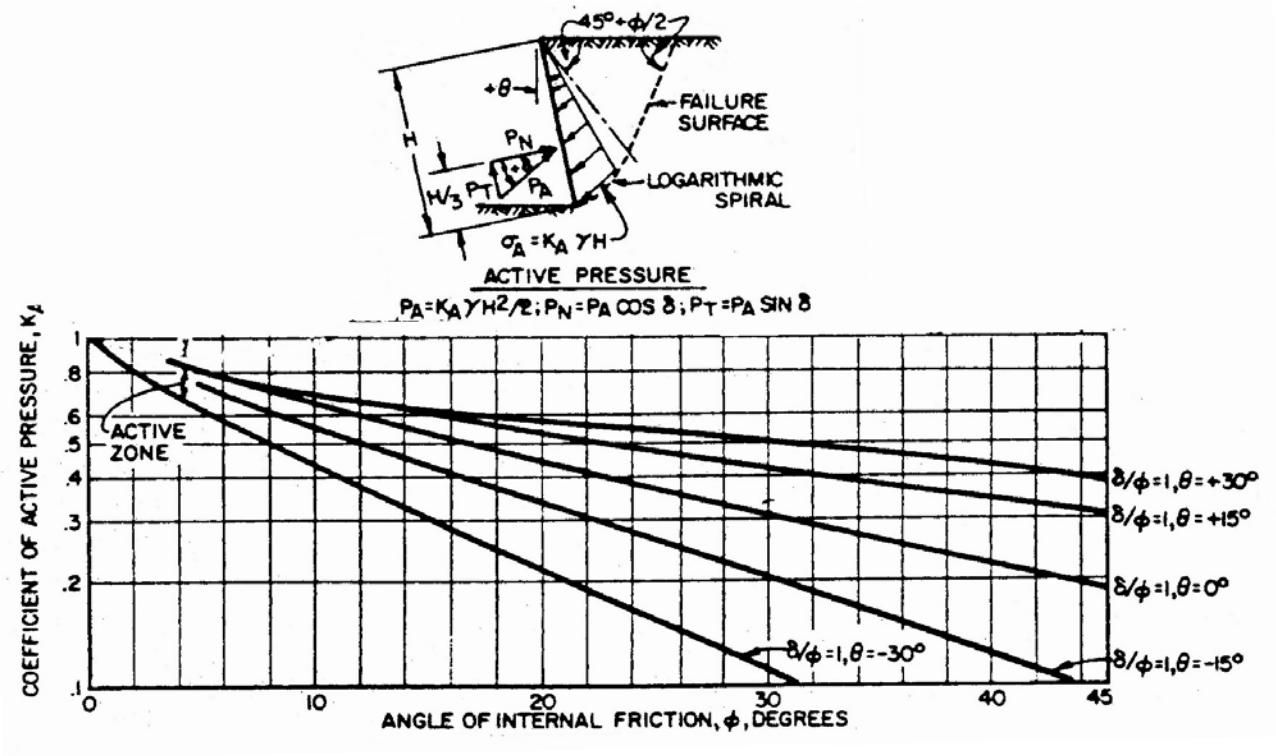
$$K_A = \tan^2\left(45 + \frac{\phi'}{2}\right) \qquad \text{(Equation 5.13)}$$

Equation 5.11 from the Coulomb theory is more general than Equation 5.12 from the Rankine theory. Both theories provide solutions that are approximately equivalent. Equation 5.13 is very simple and should be used only to provide preliminary values.

For non-vertical wall facings, the active earth pressure coefficient defined by Equation 5.12 can be estimated from charts presented in Figures 5.6 and 5.7 (Caquot and Kerisel, 1948; U.S. Navy, 1982). In these figures, a different nomenclature is used and the wall face batter angle is identified as "θ", instead of α. Figure 5.6 considers the effect of various wall face inclinations with a horizontal backslope and a wall-soil friction angle $\delta = \phi'$. The batter angle "θ" in Figure 5.6 can be a positive value (overhanging wall) or a negative value (conventional battered walls). From this figure, it is noted that the active earth coefficient decreases as the wall becomes flatter (face batter angle "θ" becomes more negative), which confirms that intuitively a "flatter wall" is more stable with all other conditions being unchanged. Figure 5.7 takes into account the effect of backslope angle for a vertical wall face and a wall-soil friction angle $\delta = \phi'$. Figure 5.7 clearly shows that the magnitude of the active coefficient grows significantly as the slope angle increases. The charts developed by Caquot and Kerisel are recommended as they rely on more realistic failure surfaces (i.e., log spiral failure surface). The factor of safety against sliding can also be calculated using standard slope stability computer programs. In such cases, the failure surfaces used in computer analysis must be non-circular, have a relatively long, nearly horizontal segment, and be forced to pass through the base of the wall.

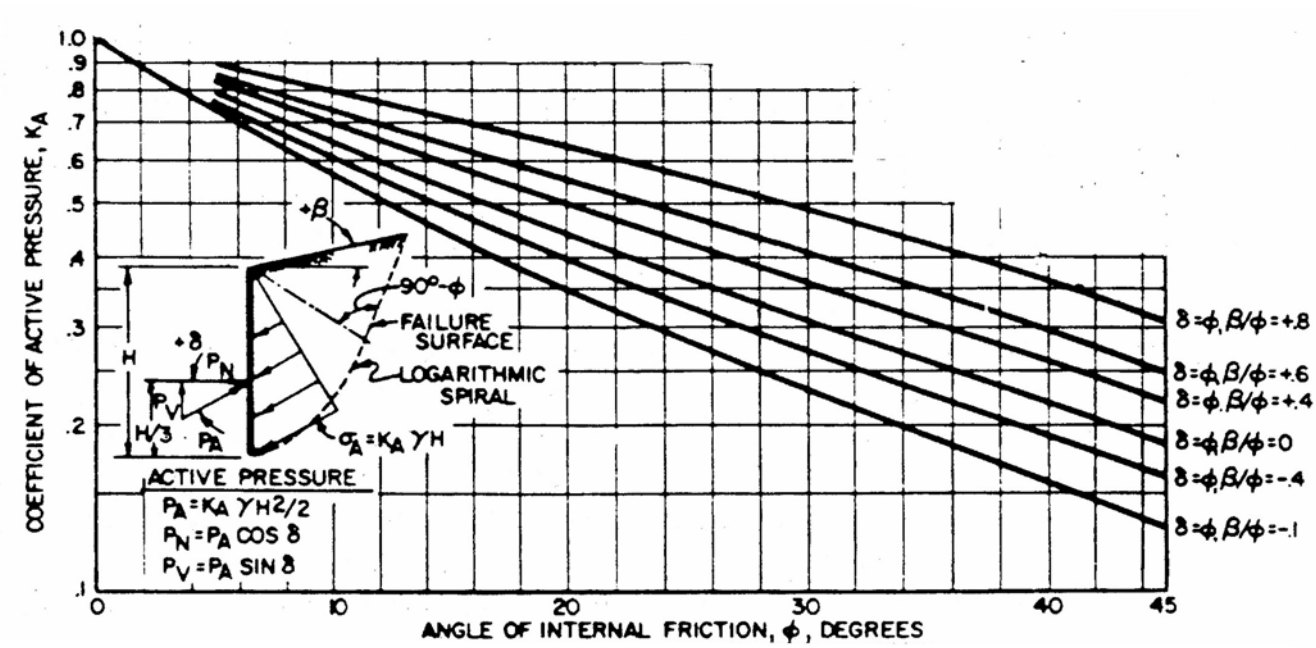
5.4.4 Bearing Capacity

Although not very often, bearing capacity may be a concern when a soil nail wall is excavated in fine-grained, soft soils. Because the wall facing does not extend below the bottom of the excavation (unlike soldier piles in cantilever or ground anchor walls), the unbalanced load caused by the excavation may cause the bottom the excavation to heave and trigger a bearing capacity failure of the foundation (Figure 5.8a).



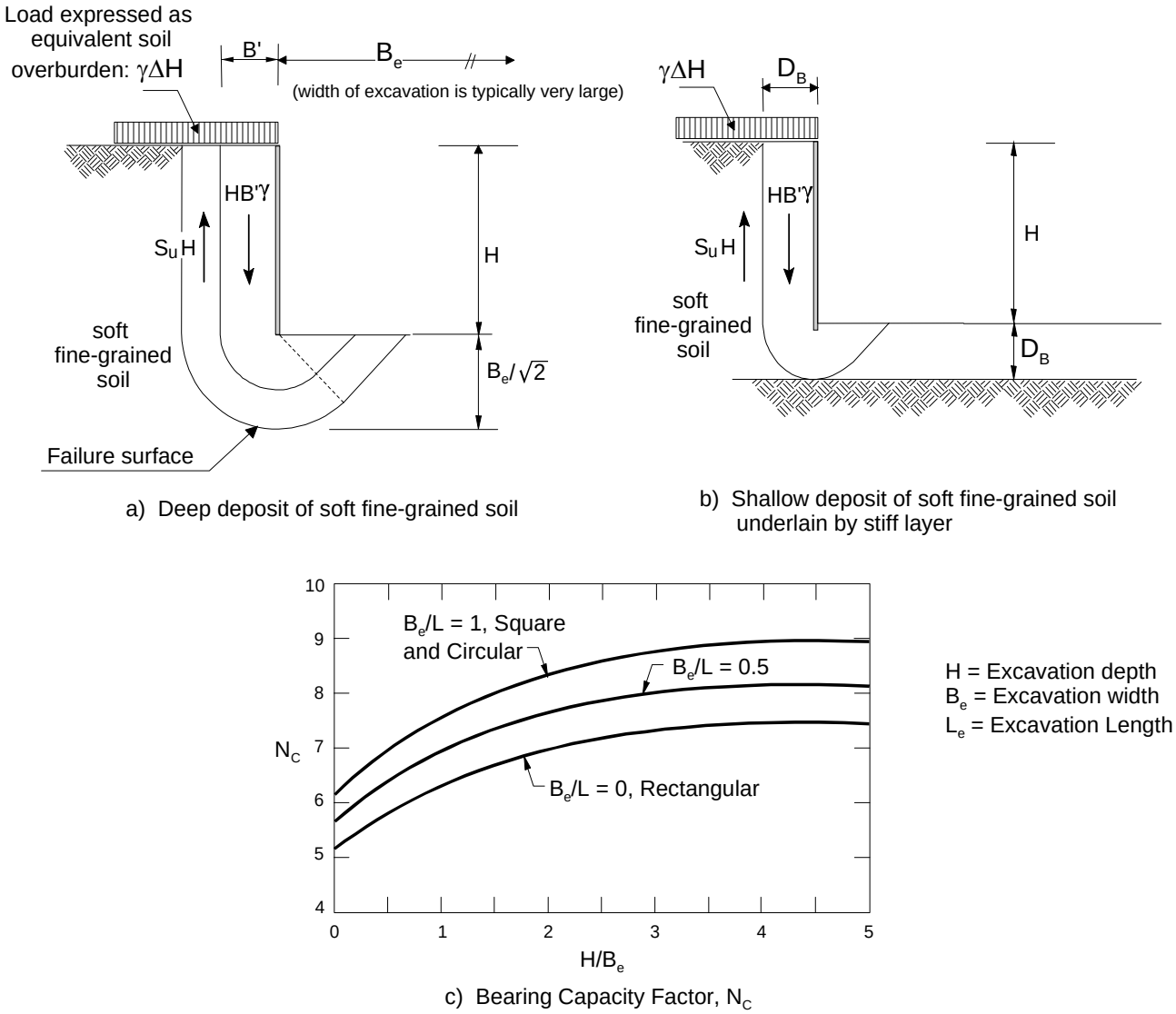
Source: U.S. Navy (1982).

Figure 5.6: Effect of Wall Batter Angle on the Active Earth Coefficient.



Source: U.S. Navy (1982).

Figure 5.7: Effect of Backslope Angle on the Active Earth Coefficient.



Source: Terzaghi, et al. (1996).

Figure 5.8: Bearing Capacity (Heave) Analysis.

Equations to assess heave potential at the bottom of excavations can be used to analyze this failure mode. The factor of safety against heave (FS_H) (Terzaghi et al., 1996), is:

$$FS_H = \frac{S_u \, N_c}{H_{eq}\left(\gamma - \frac{S_u}{B'}\right)} \qquad \text{(Equation 5.14)}$$

where these terms and the terms in Figure 5.8 are defined as:

S_u = undrained shear strength of the soil;
N_c = bearing capacity factor (Figure 5.8);
γ = unit weight of the soil behind wall;
H = height of the wall;

H_{eq} = equivalent wall height = H+ΔH, with ΔH is an equivalent overburden; and

B' = width of influence, B' = $B_e/\sqrt{2}$, where B_e = width of excavation.

The bearing capacity factor must be adopted based on the existing geometric conditions. For excavations very wide excavations (typical case for a soil nail wall), H/ B_e can be considered conservatively equal to 0. For very long walls, it is conservative to adopt B_e / L_e = 0, and N_c =5.14.

When a strong deposit underlying the soft layer and occurring at a depth D_B < 0.71 B_e below the excavation bottom is encountered (Figure 5.8b), B' in Equation 5.14 must be replaced by D_B. When the width of the excavation is very large or the contribution of the shearing resistance (i.e., S_u H) outside the failure block of width B' is neglected, equation 5.14 becomes conservatively $FS_H = N_c/ \gamma H_{eq}$. These equations are conservative because they neglect the shear contribution of the nails that are intersected by the failure surface shown in Figure 5.8a and 5.8b.

Equivalent bearing capacity analyses may also be performed using slope stability analysis programs that consider deep-seated failure surfaces through the foundation, similar to the ones shown in Figure 5.8. Bearing capacity analyses are routinely not necessary for cases where soft soils [e.g., S_u ≤ 25 (kPa) 500 psf] are not present at the bottom of the excavation. An exception to this general rule-of-thumb is when large loads are imposed behind the proposed soil nail wall. For this case, a bearing capacity analysis is recommended regardless of the soil conditions.

Factors of safety against heave for soil nail walls should be selected to be consistent with those typically used for heave analysis at the bottom of excavations. In general, FS_H can be adopted as 2.5 and 3 for temporary and permanent walls, respectively. As the great majority of soil nail walls are not constructed in soft fine-grained soils, this failure mode is not critical for most soil nail projects.

5.4.5 Seismic Considerations in Soil Nail Wall Stability

5.4.5.1 Introduction

The evaluation of seismic effects on the stability of soil nail walls is of primary importance in areas with high seismic exposure. Soil nail walls have performed remarkably well during strong ground motions, in contrast to the generally poor performance of gravity retaining structures. After the 1989 Loma Prieta, California; 1995 Kobe, Japan; and 2001 Nisqually, Washington earthquakes, it was reported that soil nail walls showed no sign of distress or significant permanent deflection, despite having experienced, in some cases, ground accelerations as high as 0.7g (Felio et al., 1990; Tatsuoka et al., 1997; and Tufenkjian, 2002). These observations indicate that soil nail walls appear to have an inherent satisfactory seismic response. This has been attributed to the intrinsic flexibility of soil-nailed systems (comparable to that of other flexible retaining systems such as MSE walls) and possibly to some levels of conservatism in existing design procedures. Similar trends have been obtained from centrifuge tests performed on reduced-scale models of soil nail walls [(e.g., Vucetic et al. (1993), Tufenkian and Vucetic (2000)]. The following three sections discuss some of the analyses needed to assess seismic effects on global and sliding stability.

5.4.5.2 Seismic Effects on Global Stability

The pseudo-static method is routinely used for the seismic stability analysis of soil nail walls. In this method, the earthquake-induced, time-varying forces of inertia acting within a potentially sliding rigid block involving the soil nail wall system are replaced by an equivalent, pseudo-static force (F_{in}) acting at the center of gravity of the analyzed block. In these cases, the horizontal and vertical components ($F_{in\,h}$ and $F_{in\,v}$, respectively) are expressed as:

$$F_{in\,h} = k_h W \qquad \text{(Equation 5.15)}$$

$$F_{in\,v} = k_v W \qquad \text{(Equation 5.16)}$$

where:

W = weight of the block;
k_h = a non-dimensional horizontal seismic coefficient; and
k_v = a non-dimensional vertical seismic coefficient.

The coefficients k_h and k_v are related to the site peak horizontal and vertical ground acceleration components, respectively. The coefficients k_h and k_v are input parameters in computer programs for the analysis of soil nail walls (e.g., SNAIL). The sign assigned to the horizontal seismic coefficient should be selected such that this component is directed away from the slope. The sign assigned to the vertical seismic coefficient should be selected such that this component is directed upwards. In practice, the vertical component is disregarded.

The coefficient k_h is a fraction of the normalized horizontal acceleration (A_m), which acts at the centroid of the wall-soil mass (AASHTO, 1996). A_m is a function of the normalized peak ground acceleration coefficient (A), which is the actual peak ground acceleration normalized by the acceleration of gravity (g), and is defined as:

$$A_m = (1.45 - A)\,A \qquad \text{(Equation 5.17)}$$

The values of A depend on the regional tectonic setting and are obtained from seismic maps for the entire United States. These seismic maps, contained in Division I-A “Seismic Design” in AASHTO (1996), provide contours of maximum horizontal ground acceleration. The A coefficient corresponds to peak ground accelerations in a profile of rock or very stiff soil (i.e., the so-called Soil Profile I in AASHTO, 1996) and have a 10-percent probability of exceedance in 50 years.

5.4.5.3 Selection of Seismic Coefficients

In flexible structures such as soil nail walls and MSE walls, it is reasonable to use seismic coefficients related to the seismically-induces wall displacement. The following expressions in SI and English units can be used to estimate the horizontal seismic coefficient as a function of the tolerable seismically induced wall lateral movement (d_e) before any wall/block sliding takes place (Elias et al., 2001):

$$k_h = 1.66 A_m \left(\frac{A_m}{d_e \text{ [mm]}} \right)^{0.25} \qquad \text{(Equation 5.18)}$$

$$k_h = 0.74 A_m \left(\frac{A_m}{d_e \text{ [in.]}} \right)^{0.25} \qquad \text{(Equation 5.19)}$$

The tolerable seismically induced displacement depends on the wall type and the service the wall provides. The equations above should be used only for $25 \leq d_e \leq 200$ mm ($1 \leq d_e \leq 8$ in.), with more typical values of d_e between 50 and 100 mm (2 to 4 in.). As shown by these equations, the selection of smaller tolerable seismically induced deformation results in larger seismic coefficients, which results in larger nail lengths. It is recommended (Elias et al., 2001) that Equations 5.18 and 5.19 should not be used when:

- the peak ground acceleration coefficient A is ≥ 0.3;
- the wall has a complex geometry (i.e., the distribution of mass and/or stiffness is abrupt); and
- the wall height is greater than approximately 15 m (45 ft).

In general, it is acceptable to select a seismic coefficient for soil nail walls between:

$$k_h = 0.5 A_m \quad \text{to} \quad 0.67 A_m \qquad \text{(Equation 5.20)}$$

This range has provided wall designs that yield tolerable deformations in highway facilities (Kavazanjian et al., 1997). Equation 5.20 is generally conservative and provides slightly larger values of the seismic coefficient than the values resulting from Equations 5.18 and 5.19. An exception should be made when the value of d is very small [e.g., less than about 25 mm (1 in.) and the normalized horizontal acceleration is large (approximately $A_m > 0.25$)].

In “soft” soils (i.e., deep deposits of medium to soft fine-grained soils occur at the site), significant ground acceleration amplification and non-linear site response may take place. Such conditions commonly require a specific site response dynamic analyses in which appropriate soil dynamic properties and representative time histories must be used.

Additionally, it may be necessary to perform a Newmark-type of post-seismic deformation analysis, in which the cumulative post-seismic displacement of a potentially unstable mass of soil is calculated in a manner analogous to the sliding of a rigid block on a ramp subjected to cyclic loading. Details of this method can be found elsewhere (Kramer, 1996; Kavazanjian et al., 1996). Dynamic and deformation analyses may be necessary when the simple pseudo-static method described above is not applicable, specifically in the case of large walls subjected to strong ground motions. However, this level of analytical complexity is commonly not required in the design of most soil nail walls.

5.4.5.4 Seismic Effects on Sliding Stability

When analyzing explicitly (not with computer programs) the sliding stability of a soil nail wall under seismic loads, the total active thrust (P_{AE}) during an earthquake due to earth pressures behind the soil block must be considered. This force is the combination of the static and dynamic active lateral earth pressures that are induced by the inertial forces. When considering sliding force equilibrium, this increased lateral earth force must be taken into account instead of the static force P_A in the sum of driving forces, ΣD, presented in Equation 5.9.

The lateral earth force, including seismic effects, can be evaluated using the Mononobe-Okabe (M-O) method, which is an extension of the Coulomb theory (Mononobe, 1929; Okabe, 1926). The M-O method assumes that:

- the total active thrust P_{AE} acts behind the wall;
- the wall and the nailed soil are considered a rigid block (the ground acceleration is fully transmitted to the system);
- the wall movement induces active earth pressure conditions behind the block (the soil behind the soil nail wall system is "yielding"); and
- the soil behind the soil nail wall system is drained, (i.e. neither excess pore pressures nor hydrodynamic effects are considered, which is typical for soil nail walls).

The total active thrust, P_{AE}, acting behind the wall-nailed soil block is expressed, in its more general case, as (Ebeling and Morrison, 1992):

$$P_{AE} = \frac{\gamma H_1^2}{2} K_{AE} (1 - k_v) \left\{ 1 + \frac{2\, q_s}{\gamma\, H_1} \left[\frac{\cos \alpha}{\cos(\beta - \alpha)} \right] \right\} \qquad \text{(Equation 5.21)}$$

where:

γ = total unit weight of soil behind block;
H_1 = effective height of soil mass that considers sloping ground;
k_v = the vertical seismic coefficient;
K_{AE} = total (static and dynamic) active pressure coefficient; and
q_s = distributed surface loading

In the general case of a wall (Figure 5.9), the total active pressure coefficient can be calculated using the M-O formulation:

$$K_{AE} = \frac{\cos^2(\varphi - \omega - \alpha')}{\cos\omega\ \cos^2\alpha'\ \cos(\alpha' + \delta + \omega)\ D} \qquad \text{(Equation 5.22)}$$

where:

ϕ = angle of internal friction of soil behind wall;
α' = batter angle (from vertical) of wall internal face;

β = backslope angle;
δ = wall-soil interface friction angle; and
ω = an angle relating the horizontal and vertical seismic coefficients as:

$$\omega = \tan^{-1}\left(\frac{k_h}{1 - k_v}\right) \qquad \text{(Equation 5.23)}$$

and D is defined as:

$$D = \left[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \omega - \beta)}{\cos(\delta + \alpha' + \omega)\cos(\beta - \alpha')}}\right]^2 \qquad \text{(Equation 5.24)}$$

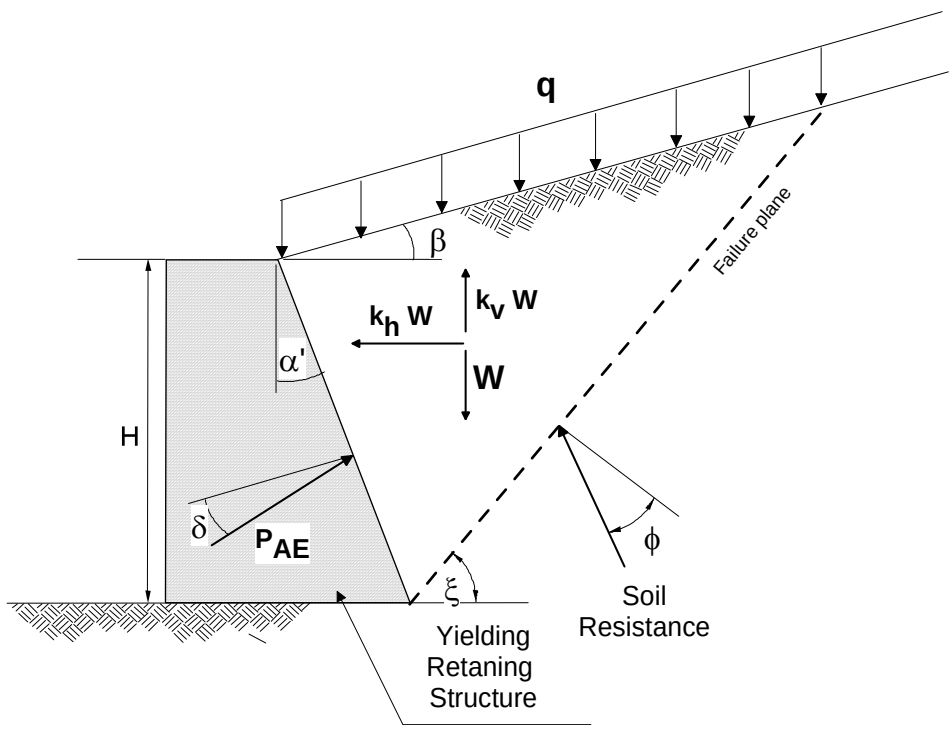


Figure 5.9: Generic Wall Geometry in the Mononobe Okabe Method.

The failure plane behind the wall is oriented at an angle ξ from the horizontal (Zarrabi-Kashani, 1979; Kramer, 1996), which is defined as:

$$\xi = \varphi - \omega + \rho^* \qquad \text{(Equation 5.25)}$$

where:

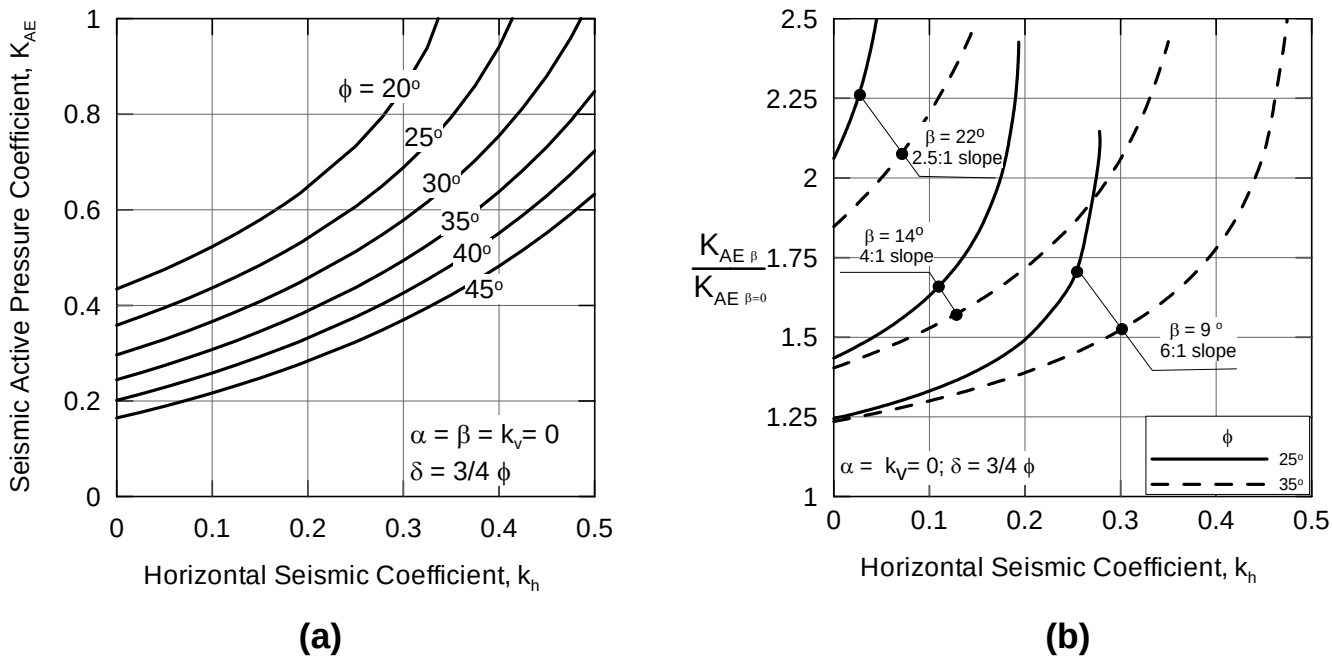
$$\rho^* = \tan^{-1}\left[\frac{\sqrt{A\left(A^2 + 1\right)\left(A + B\right)} - A^2}{1 + B\left(A^2 + 1\right)}\right] \qquad \text{(Equation 5.26)}$$

with:

$$A = \tan\left(\varphi - \omega - \beta\right) \qquad \text{(Equation 5.27)}$$

$$B = \tan\left(\delta + \omega + \beta\right) \qquad \text{(Equation 5.28)}$$

The total active pressure coefficient K_{AE} can also be estimated from Figure 5.10, which is based on the generalized M-O solution.



(a) **(b)**

Figure 5.10: Total Active Pressure Coefficients: (a) Horizontal Backslope, and (b) Correction for Non-Horizontal Backslope.

Figure 5.10a shows the variation of the total active pressure coefficient as a function of the horizontal seismic coefficient and the friction angle (for values of ϕ ranging from 20º to 45º) for horizontal backslope, no vertical seismic coefficient, and δ = 0.75 ϕ. Figure 5.10b presents a correction for the total active pressure coefficient when the backslope is not horizontal. Another graphical source that can be used to calculate the total active pressure coefficient with the M-O method is from Lam and Martin (1986). Note that the M-O formulation does not arrive at a solution for certain combinations of the variables, in particular, when the slope of the backslope is greater than 22°. Another limitation of the M-O method is that the seismic coefficient provides a relatively simple approximation and cannot capture the complex deformation response of a soil nail wall system. The M-O procedure does not need to be considered in computer programs that use the seismic coefficients.

The recommended design procedures presented in Chapter 6 provide a step-by-step description that includes the evaluation of seismic effects based on the M-O method and developed for MSE walls (Elias et al., 2001).

5.5 INTERNAL FAILURE MODES

5.5.1 Introduction

Internal failure modes refer to failure in the load transfer mechanisms between the soil, the nail, and the grout. Soil nails mobilize bond strength between the grout and the surrounding soil as the soil nail wall system deforms during excavation. The bond strength is mobilized progressively along the entire soil nail with a certain distribution that is affected by numerous factors. As the bond strength is mobilized, tensile forces in the nail are developed.

Depending on the soil nail tensile strength and length, and the bond strength, bond stress distributions vary and different internal failure modes can be realized. Typical internal failure modes related to the soil nail are (Figures 5.3d–g):

- *Nail Pullout Failure*: Nail pullout failure is a failure along the soil-grout interface due to insufficient intrinsic bond strength and/or insufficient nail length, Figure 5.3d.
- *Slippage of the Bar-Grout Interface*: The strength against slippage along the grout and steel bar interface (Figure 5.3e) is derived mainly from mechanical interlocking of grout between the protrusions and "valleys" of the nail bar surface. Mechanical interlocking provides significant resistance when threaded bars are used and is negligible in smooth bars. The most common and recommended practice is the use of threaded bars, which reduces the potential for slippage between the nail bar and grout.
- *Tensile Failure of the Nail*: The nail can fail in tension if there is inadequate tensile strength, Figure 5.3f.
- *Bending and Shear of the Nails*: Soil nails work predominantly in tension, but they also mobilize stresses due to shear and bending at the intersection of the slip surface with the soil nail (Schlosser, 1983; Elias and Juran, 1991), Figure 5.3g. The shear and bending resistances of the soil nails are mobilized only after relatively large displacements have taken place along the slip surface. Some researchers have found that shear and bending nail strengths contribute no more than approximately 10 percent of the overall stability of the wall. Due to this relatively modest contribution, the shear and bending strengths of the soil nails are conservatively disregarded in the guidelines contained in this document. A discussion of a methodology to account for shear and bending contributions is included in Elias and Juran (1991).

A discussion of the two most common internal failure modes (i.e., nail pullout and nail tensile failure) is presented in the following two sections. In addition, a section describing the relationship of the pullout resistance and the tensile force distribution in the nail is presented.

5.5.2 Nail Pullout Failure

Pullout failure is the primary internal failure mode in a soil nail wall. This failure mode may occur when the pullout capacity per unit length is inadequate and/or the nail length is insufficient. In

general, the mobilized pullout per unit length, Q, (also called the load transfer rate) can be expressed as:

$$Q = \pi\, q\, D_{DH} \qquad \text{(Equation 5.29)}$$

where:

q = mobilized shear stress acting around the perimeter of the nail-soil interface; and
D_{DH} = average or effective diameter of the drill hole.

Considering a single nail segment subjected to a tensile force, T_o, at one end, and applying equilibrium of forces along the differential length of the nail shown in Figure 5.11, the tensile force can be related to the interface shear stress as:

$$dT = \pi\, D_{DH}\, q\, dx = Q\, dx \qquad \text{(Equation 5.30)}$$

The above equation represents the transfer mechanism between the stresses on the nail-soil interface to tensile forces in the nail bar. In general, the tensile force (T) at a distance "x" along the bar is:

$$T(x) = \int_0^x \pi\, D_{DH}\, q\, dx = \int_0^x Q\, dx \qquad \text{(Equation 5.31)}$$

Actual distributions of mobilized bond shear stress (and load transfer rates) are not uniform, as illustrated in Figure 5.11, and depend on various factors including nail length, magnitude of applied tensile force, grout characteristics, and soil conditions. As a simplification, the mobilized bond strength is often assumed to be constant along the nail, which results in a constant load transfer rate, Q. As a result, the nail force at the end of the pullout length, L_p, is:

$$T(L_p) = T_o = Q\, L_p \qquad \text{(Equation 5.32)}$$

The pullout capacity, R_p, is mobilized when the ultimate bond strength is achieved and is expressed as:

$$R_p = T_{max} = Q_u\, L_p \qquad \text{(Equation 5.33)}$$

with:

$$Q_u = \pi\, q_u\, D_{DH} \qquad \text{(Equation 5.34)}$$

where:

Q_u = pullout capacity per unit length (also referred to as load transfer rate capacity); and
q_u = ultimate bond strength.

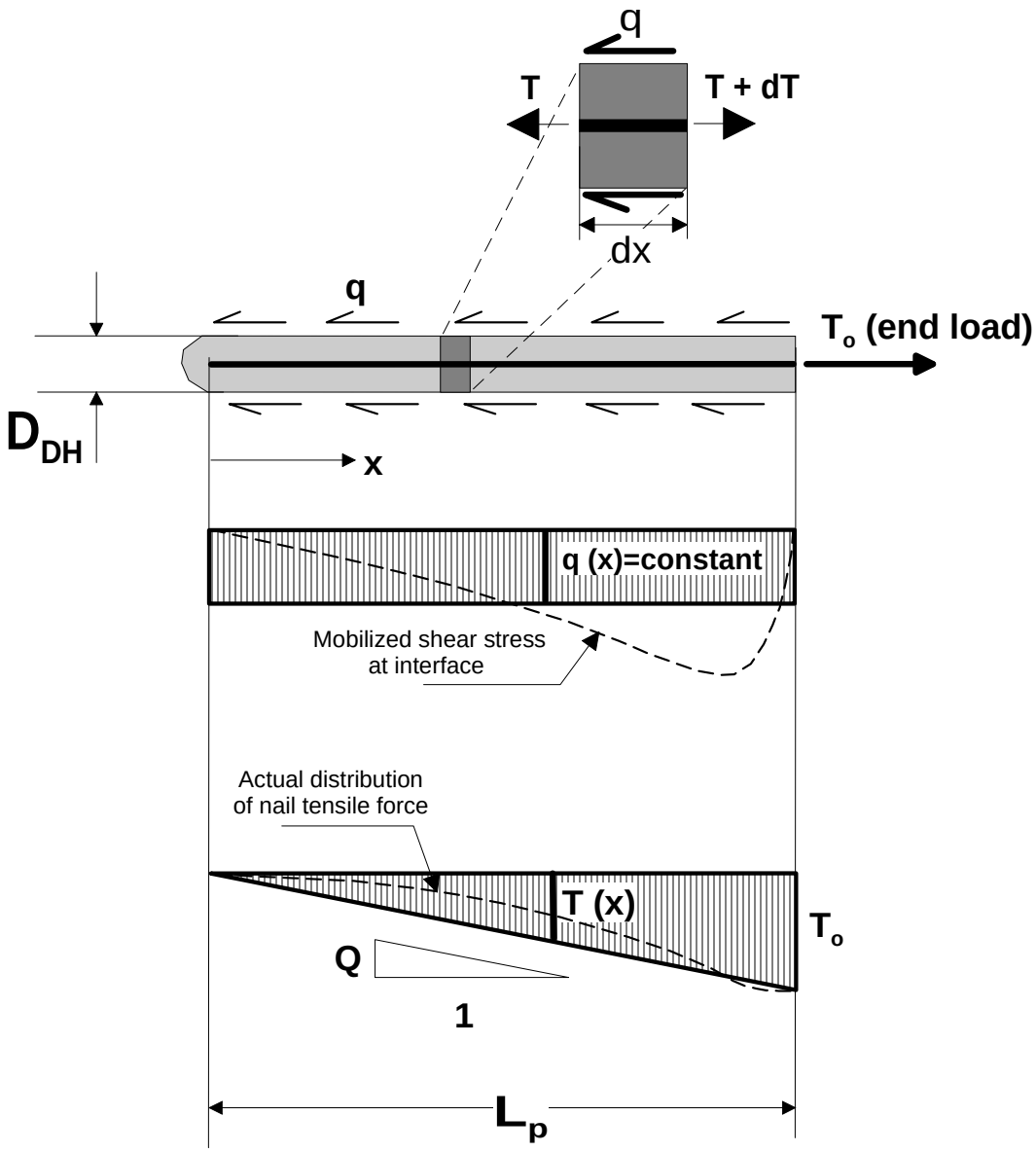


Figure 5.11: Single Nail Stress-Transfer Mode.

In the literature, it is common to find references to either q_u or Q_u. As discussed in Chapter 3, the bond strength depends on various factors, including the soil type, soil conditions, and the nail installation method. Typical values of ultimate bond strength for various soils and drilling methods were previously presented in Table 3.10. As an alternative to using published typical values, the equations above can be used to calculate apparently uniform, ultimate bond strengths and pullout capacity per unit length from nail pullout tests. These tests are described in Chapter 8.

To take into account uncertainties regarding bond strength and soil-grout interaction, the following allowable values of the bond strength or pullout capacity per unit length are used in design:

$$q_{ALL} = \frac{q_u}{FS_p} \qquad \text{(Equation 5.35)}$$

$$R_{p\,ALL} = \frac{R_p}{FS_P} \qquad \text{(Equation 5.36)}$$

where FS_P is the factor of safety against pullout failure. In general, a minimum factor of safety of 2 is recommended against pullout failure.

5.5.3 Soil-Nail Interaction and Tensile Force Distribution

5.5.3.1 Conceptual Model

The soil-nail interaction that occurs behind the wall facing is complex. The loads applied to the soil nails originate as reactions to the outward wall movement during excavation of the soil in front of the wall, as discussed earlier. The portion of the nail behind the failure surface (i.e., the anchoring zone) is pulled out of the soil slope. The tensile forces in the soil nail, T, vary from the anchoring zone to the facing as follows: they start as zero at the end of the nail, increase to a maximum, T_{max}, value in the intermediate length, and decrease to a value T_o at the facing (Figure 5.12).

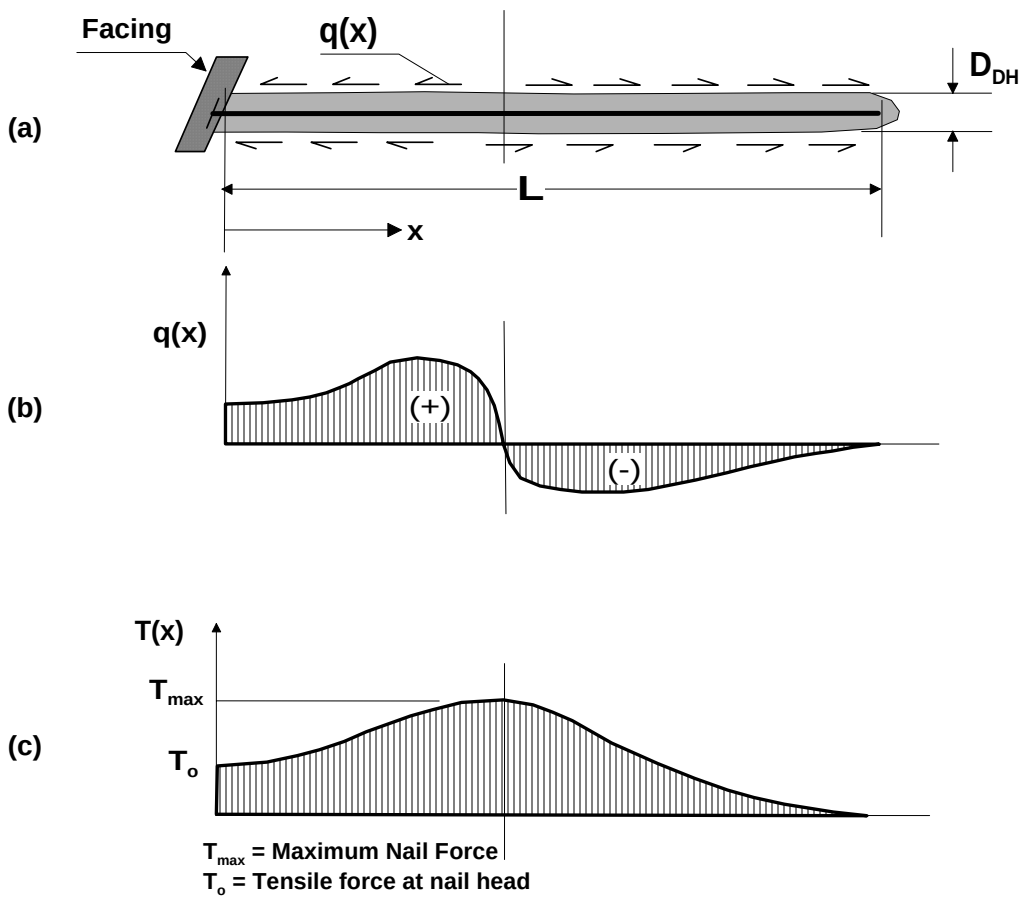
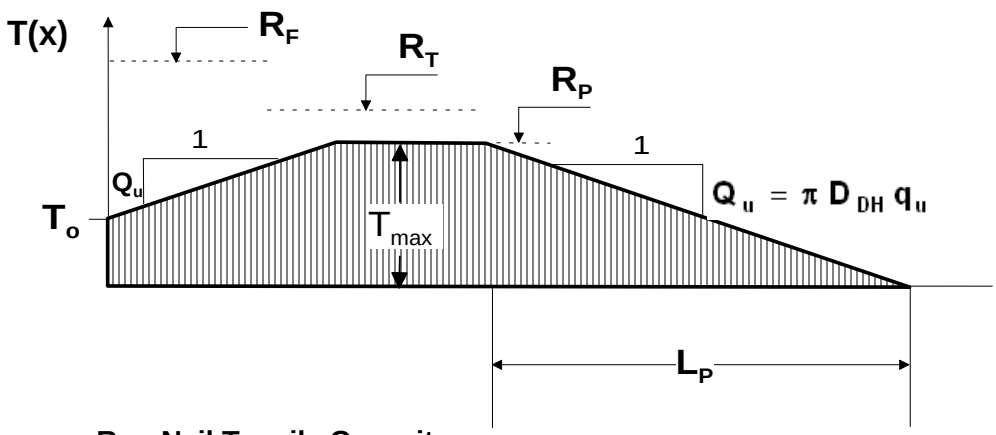


Figure 5.12: Soil Nail Stress-Transfer Mechanism.

The maximum nail tensile force in the nail bar nail does not necessarily occur at the point where the nail crosses the failure surface, as will be discussed subsequently. The mobilized shear stress along the grout-soil interface, q, is not uniform and, in fact, changes from "positive" to "negative", as shown in Figure 5.12a and b. The schematic distribution of the tensile force (T) along the soil nail is shown in Figure 5.12c.

5.5.3.2 Simplified Distribution of Nail Tensile Forces

For design, the tensile force distribution along the nail shown in Figure 5.12 can be simplified as shown in Figure 5.13. The tensile force the nail increases at a constant slope Q_u (equal to the pullout capacity per unit length), reaches a maximum value, T_{max}, and then decreases at the rate Q_u to the value T_o at the nail head. With reference to Figure 5.13, the following three conditions related to the maximum tensile force are noted. The value T_{max} is bounded by three limiting conditions: the pullout capacity, R_P, the tensile capacity, R_T, and the facing capacity, R_F. The pullout capacity was defined in section 5.5.2. The tensile capacity is defined in section 5.5.4, and the facing capacity is defined in section 5.6. If $R_P < R_T$ and R_F, pullout failure controls the value of T_{max}. If $R_T < R_P$ and R_F, tensile failure controls T_{max}. Finally, if $R_F < R_T$ and R_P, failure of the facing may control, depending on the ratio of T_o/T_{max}.



R_T = Nail Tensile Capacity
R_F = Facing Capacity
R_P = Pullout Capacity

Q_u, q_u = Ultimate load transfer rate and bond strength

T_O ~ 0.6-1.0 T_{max}

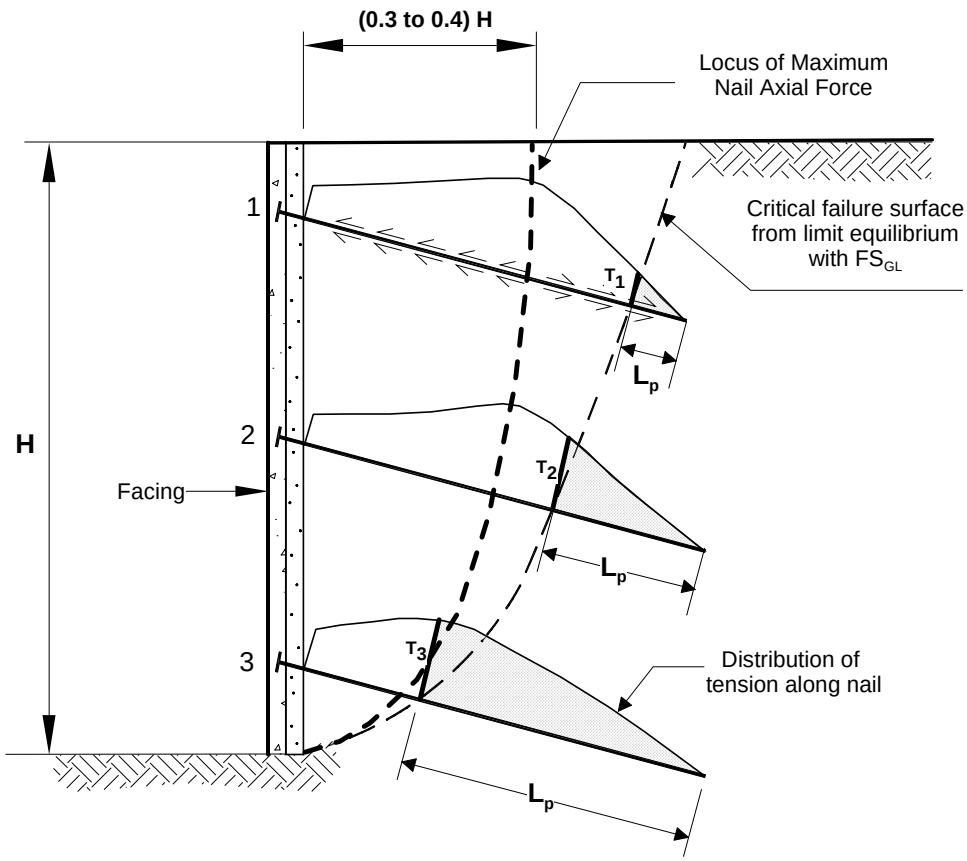
1) $R_P < R_T < R_F$ (pullout controls, shown in example above)
2) $R_T < R_P < R_F$ (tensile failure controls)
3) $R_F < R_P$ or R_T (facing failure may control depending on T_o/T_{max})

Figure 5.13: Simplified Distribution of Nail Tensile Force.

To achieve a balanced design, all of the resisting components in a system should have comparable margins of safety; no component should be significantly oversized or undersized. In the case of nail tensile forces, a good design should balance the capacities of all resisting elements; therefore, values of R_P, R_T, and R_F should be reasonably similar.

5.5.3.3 Maximum Tensile Forces Distribution

The tensile force in a particular nail is a function of the location where the nail crosses the failure surface. As shown schematically in Figure 5.14, the distribution of tensile forces in the soil nails varies throughout the cross-section of the wall system. Due to the complexities of load transfer within individual nails, the location of maximum nail tensile forces is close to, but generally does not coincide with, the location of the critical failure surface found during global stability analysis. The location of the failure surface is controlled by global limit equilibrium considerations. Strain measurements in instrumented soil nail walls have indicated that in the upper portion of the wall, the maximum tensile force occurs approximately between 0.3 H to 0.4 H behind the wall facing (Plumelle et al., 1990; Byrne et al, 1998). In the lower portion of the wall, the maximum tensile force occurs approximately between 0.15 H to 0.2 H behind the wall facing.



Modified after Byrne et al., 1998.

Figure 5.14: Schematic Location of Soil Nail Maximum Tensile Forces.

Figure 5.14 also shows that the contribution of tensile forces to global stability varies from nail to nail. In general, the stability contribution of each soil nail depends on the location of the intersection of the failure surface and the nail. The location of the failure surface determines the nail length behind the failure surface (L_p). For example, in Figure 5.14 the stability contribution of the upper soil nail, T_1, upon completion of the wall is not significant because the nail length behind the failure surface is insufficient to develop the maximum potential pullout capacity of the soil nail. The force contributions of the lower soil nails, T_2 and T_3, are relatively large because the pullout lengths of those nails are longer than the nails in the upper reaches of the wall. In this example, for the lowest nail shown, L_p is sufficient to mobilize the full pullout capacity. However, it remains to verify if the full pullout capacity is less than the soil nail tensile capacity.

The tensile forces in the nail develop gradually with time as excavation proceeds from top to bottom in front of the wall. Generally, maximum nail tensile forces in a given row develop when the two subsequent excavation lifts are exposed. Tensile forces may increase moderately (e.g., generally 15 percent) in the time period between the end of construction condition and the long-term, steady condition (Plumelle et al., 1990). These post construction increases occur due to post construction soil creep and stress relaxation. Although this additional load is not calculated, it is taken into consideration in the design of soil nail walls by means of factors of safety.

5.5.3.4 Measured Maximum Tensile Forces Distribution

A summary of maximum nail tensile forces measured in actual soil nail walls is shown in Figure 5.15 (Byrne et al., 1998). Nail forces are normalized with respect to the soil unit weight; the soil nail vertical and horizontal spacing, S_V, S_H; the wall height H; and the calculated active earth pressure coefficient, K_A, and are shown as a function of soil nail depth. These values are believed to represent the long-term soil nail forces (e.g., not the end-of-construction forces) and do not include any additional loads that may be induced by freezing (or other) forces at the face. It is important to note that these measured forces correspond to in-service conditions, not failure conditions.

As shown in Figure 5.15, the normalized nail loads range in the upper two thirds of the wall from 0.4 to 1.1, averaging approximately 0.75. The normalized maximum nail forces typically vary with depth: they increase from approximately 0.5-0.6 near the top to 0.75-0.85 in the central third, then decrease to about 0.4-0.5 at the lower third, and tend to 0 at the bottom. These observations are consistent with those trends observed in experimental walls in the Clouterre project (Plumelle et al., 1990). The in-service maximum tensile forces could be computed from the normalized nail loads in Figure 5.15 as the product of the equivalent pressure ($T/K_a\ H\ \gamma$) and an influence area around individual nails ($S_V \times S_H$).

For practical purposes, it can be considered that the normalized nail tensile force is uniform at 0.75 in the upper two thirds of the wall. Therefore, the average maximum in-service tensile force in the nails in this location is $T_{max} = 0.75\ K_A\ \gamma\ H\ S_V\ S_H$. The tensile force in the lower portion decrease considerably, to approximately 50 percent of the value in the upper part. Alternatively, Briaud and Lim (1997) suggest that the average maximum in-service tensile force in the top row of soil nails can be calculated as $T_{max} = 0.65\ K_A\ \gamma\ H\ S_V\ S_H$. For subsequent soil nail rows, Briaud and Lim (1997) also suggest that the maximum in-service tensile force is only half of the upper nails.

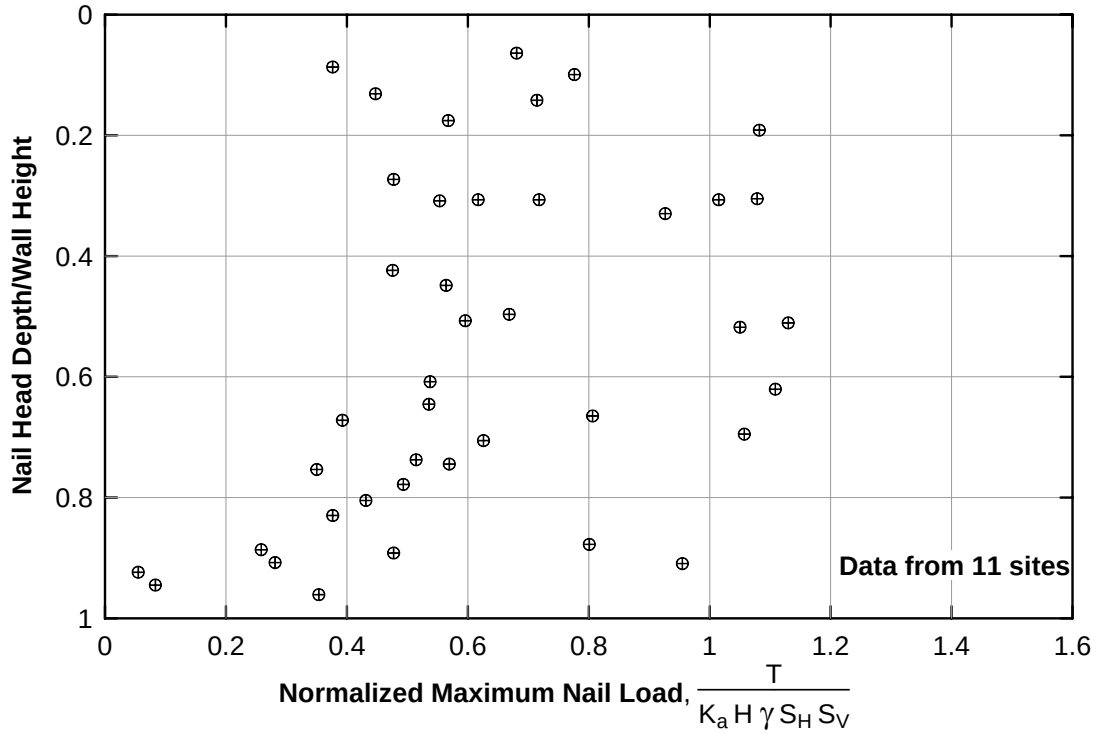


Figure 5.15: Summary of Maximum Nail Tensile Forces Measured in Walls.

The information provided above is intended to illustrate that the distribution of in-service tensile force in soil nail walls is complex and that the average nail force is smaller than that calculated by considering the full active earth lateral pressure distribution. This implication is important for the calculation of the tensile force at the nail facing, which is presented section 5.6.

5.5.4 Design for Nail Tensile Failure

To achieve a balanced design for all internal failure modes, the soil strength must be fully mobilized consistently with the full mobilization of the nail tensile strength at the same time. In other words, when $FS_G = 1.0$ (full soil mobilization), the safety factor for the tensile strength, $FS_T = 1.0$ (full nail tensile mobilization). The nail tensile force for this condition is the maximum design force in the nail ($T_{max\text{-}s}$). It is intuitive that when the loads kept constant, the design force $T_{max\text{-}s}$ will increase when $FS_G > 1.0$. This is caused because for $FS_G > 1$, the soil strength is not fully mobilized and the tensile forces must compensate to achieve equilibrium. Therefore, calculating $T_{max\text{-}s}$ directly from the global stability analysis giving $FS_G > 1$ is more conservative.

The program SNAIL automatically reports the average nail tensile force, but not the maximum tensile force corresponding to $FS_G = l$. Thus, to estimate the maximum nail tensile force for a $FS_G = l$ without performing an additional stability analysis, the following simplified method can be used. This procedure is based on the fact that the ratio of the maximum nail load calculated by SNAIL, T_{max}, to the average nail load, T_{avg}, for $FS_G > 1$, is similar to the ratio of the maximum nail load for $FS_G = 1$, $T_{max\text{-}s}$, to the average nail load, $T_{avg\text{-}s}$, for $FS_G = 1$. Therefore, a good approximation of the maximum design nail load ($T_{max\text{-}s}$) can be obtained by the following relationship:

$$\frac{T_{max-s}}{T_{max}} = \frac{T_{avg-s}}{T_{avg}} \qquad \text{(Equation 5.37)}$$

T_{avg-s} is the average design nail load and is reported by SNAIL in output files as the "Maximum Average Reinforcement Working Force". The design nail force T_{max-s} is used to verify the tensile capacity failure, which is defined as follows.

A tensile failure of a soil nail takes place when the longitudinal force along the soil nail, T_{max-s}, is greater than the nail bar tensile capacity (R_T), which is defined as:

$$R_T = A_t f_y \qquad \text{(Equation 5.38)}$$

where A_t is the nail bar cross sectional area and f_y is the nail bar yield strength. The tensile capacity provided by the grout is disregarded, due to the difference in stiffness (i.e., modulus of elasticity) between the grout and the nail. To take into account uncertainties related to material strength and applied loads, allowable values of the nail tensile capacity are used in design as follows:

$$R_{T\ ALL} = \frac{R_T}{FS_T} \qquad \text{(Equation 5.39)}$$

where FS_T is the factor of safety against soil nail tensile failure. In general, a minimum factor of safety of 1.8 is adopted for static loads.

5.6 FACING CONNECTION FAILURE MODES

5.6.1 Introduction

The most common potential failure modes at the facing-nail head connection are presented in Figure 5.3 and are shown in detail in Figure 5.16 as:

- *Flexure Failure*: This is a failure mode due to excessive bending beyond the facing's flexural capacity. This failure mode should be considered separately for both temporary and permanent facings.
- *Punching Shear Failure*: This failure mode occurs in the facing around the nails and should be evaluated for both temporary and permanent facings.
- *Headed-Stud Tensile Failure*: This is a failure of the headed studs in tension. This failure mode is only a concern for permanent facings.

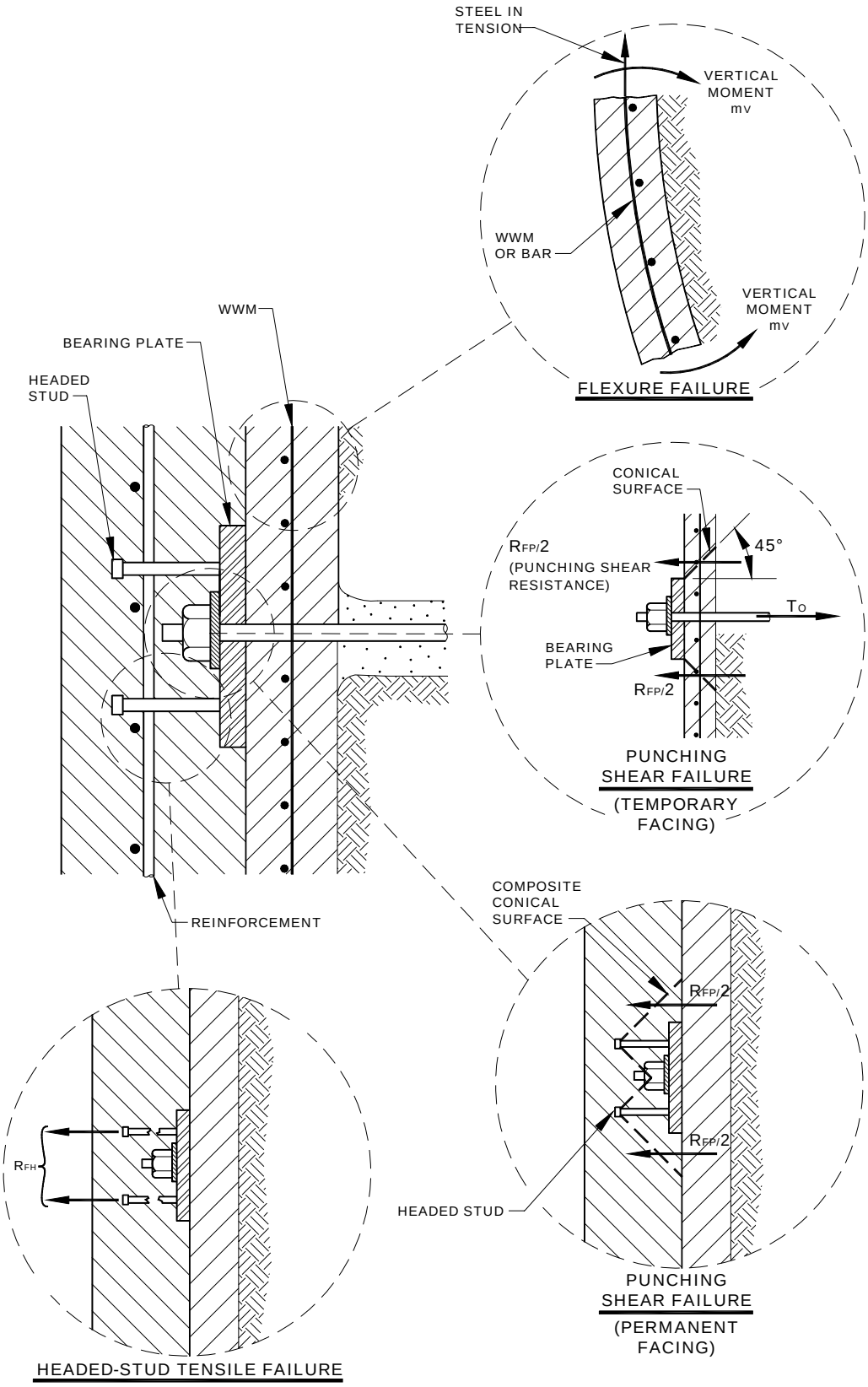


Figure 5.16: Facing Connection Failure Modes.

For each of these failure modes, the nail head and facing must be designed to provide capacity in exceedance of the maximum nail head tensile force (T_o) at the wall face. Appropriate dimensions, strength, and reinforcement of the facing and suitable nail head hardware (e.g., bearing plate, nut, and headed studs) must be provided to achieve the design capacities with adequate factors of safety for all potential failure modes. A description of how to calculate maximum nail head tensile force is subsequently presented. Then, a description of each of the failure modes described above is presented in the following three sections, as well as the associated equations used to calculate the capacity for these failure modes. Chapter 6 contains a step-by-step methodology to verify the design capacity of the soil nail wall facing.

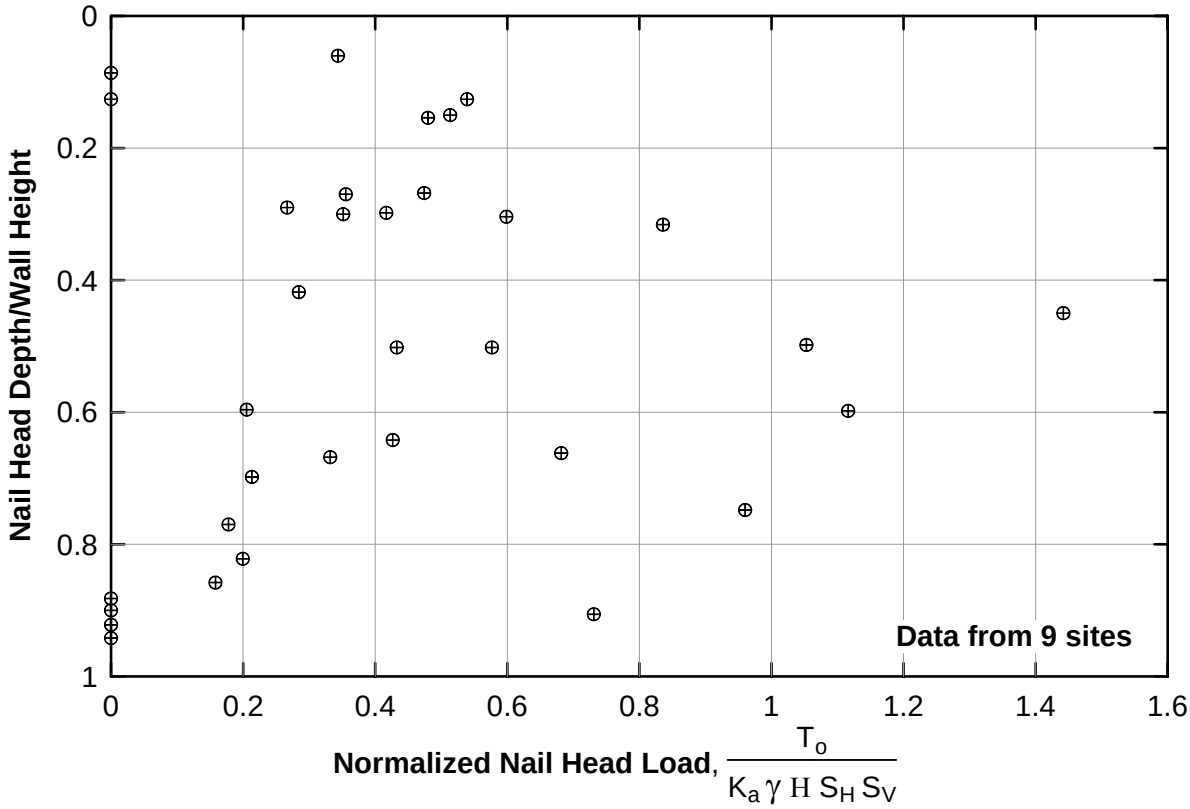
5.6.2 Tensile Forces at the Wall Facing

The nail tensile force at the wall face, T_o, is smaller than or equal to the maximum nail tensile force, as shown in Figure 5.17 (Byrne et al., 1998). Figure 5.17 presents the in-service normalized values of the nail tensile forces measured at the facing of actual soil nail walls. These values are related to long-term soil nail forces and do not include freezing (or other) forces at the face. The normalized nail forces at the facing, also referred to as the nail head force, are comparable in distribution to the normalized maximum nail tensile forces shown in Figure 5.15. By comparing these two figures, the ratio of normalized nail head force to the maximum nail force varies from 0.6 to 1.0. In the upper half of the wall, the mean, normalized nail head force ranges between 0.4 and 0.5; in the lower half, the normalized forces decrease gradually and tends to zero at the bottom. Considering the normalization and influence area described above, this trend shows that that head nail tensile force typically varies from $T_o = 0.60\ K_A \gamma H$ to $0.70\ K_A \gamma H$.

These observations are consistent with those made on experimental walls in Germany and in France. In Germany, actual earth pressure measurements, recorded via total stress cells located at the shotcrete-soil interface, indicate that the equivalent earth pressure on the facing between 60 to 70 percent of the Coulomb active earth pressure for most conditions (Gässler and Gudehus, 1981). In the French tests, the ratio of the nail head force to the maximum nail force generally varied between 0.4 and 0.5 in the upper portion of the walls (FHWA, 1993a). In addition, these test results showed that due to the effect of soil arching between nails, a closer spacing of the nails caused a reduction in the measured forces on the wall facing as compared to what would be expected using simple tributary area contributions.

Based on these results, the Clouterre (1991) design guidelines recommend adopting in-service values of the head nail tensile force as:

- 60 percent of the maximum nail service load for a nail vertical spacing of 1 m (3 ft) or less;
- 100 percent of the maximum nail service load for a nail vertical spacing of 3 m (10 ft) or more; and
- a linear interpolation for intermediate nail spacing.



From Byrne et al. (1998).

Figure 5.17: Summary of Facing Tensile Forces Measured in Walls.

This recommendation is incorporated in this document and is formally expressed in both SI and English units as follows:

$$T_o = T_{max\text{-}s}\left[0.6 + 0.2\,(S_{max}\,[\text{m}] - 1)\right] \qquad \text{(Equation 5.40)}$$

$$T_o = T_{max\text{-}s}\left[0.6 + 0.057\,(S_{max}\,[\text{ft}] - 3)\right] \qquad \text{(Equation 5.41)}$$

where:

T_o = Design nail head tensile force;

$T_{max\text{-}s}$ = Maximum design nail tensile force obtained from global stability analysis using results from a computer program (e.g., SNAIL) as described in Section 5.5.4; and

S_{max} = Maximum soil nail spacing. Use maximum of S_V and S_H, the vertical and horizontal nail spacing, respectively, in Equations 5.40 and 5.41.

For a typical nail head spacing of 1.5 m (5 ft), the nail head tensile force corresponds to a recommended facing service load of about 0.7 times the maximum nail service load.

5.6.3 Flexural Failure

5.6.3.1 Introduction

The soil nail wall facing can be considered a continuous reinforced concrete slab where the loading is the lateral earth pressure acting on the facing and the supports are the tensile forces in the soil nails (Figure 5.18a and b). The loads from the lateral earth pressure and the "reaction" in the soil nails induce flexural moments in the facing section. Positive moments (i.e., tension on the outside of the section) are generated in the midspan between nails; negative moments (i.e., tension on the inside of the section) are generated around the nails (Figure 5.18b). If these moments are excessive, a flexural failure of the shotcrete may occur. Similarities in the loading mechanism between wall facings and continuous concrete slabs supported on columns, suggest that conventional concrete slab analysis and design methods can be applied to the design of soil nail wall facing.

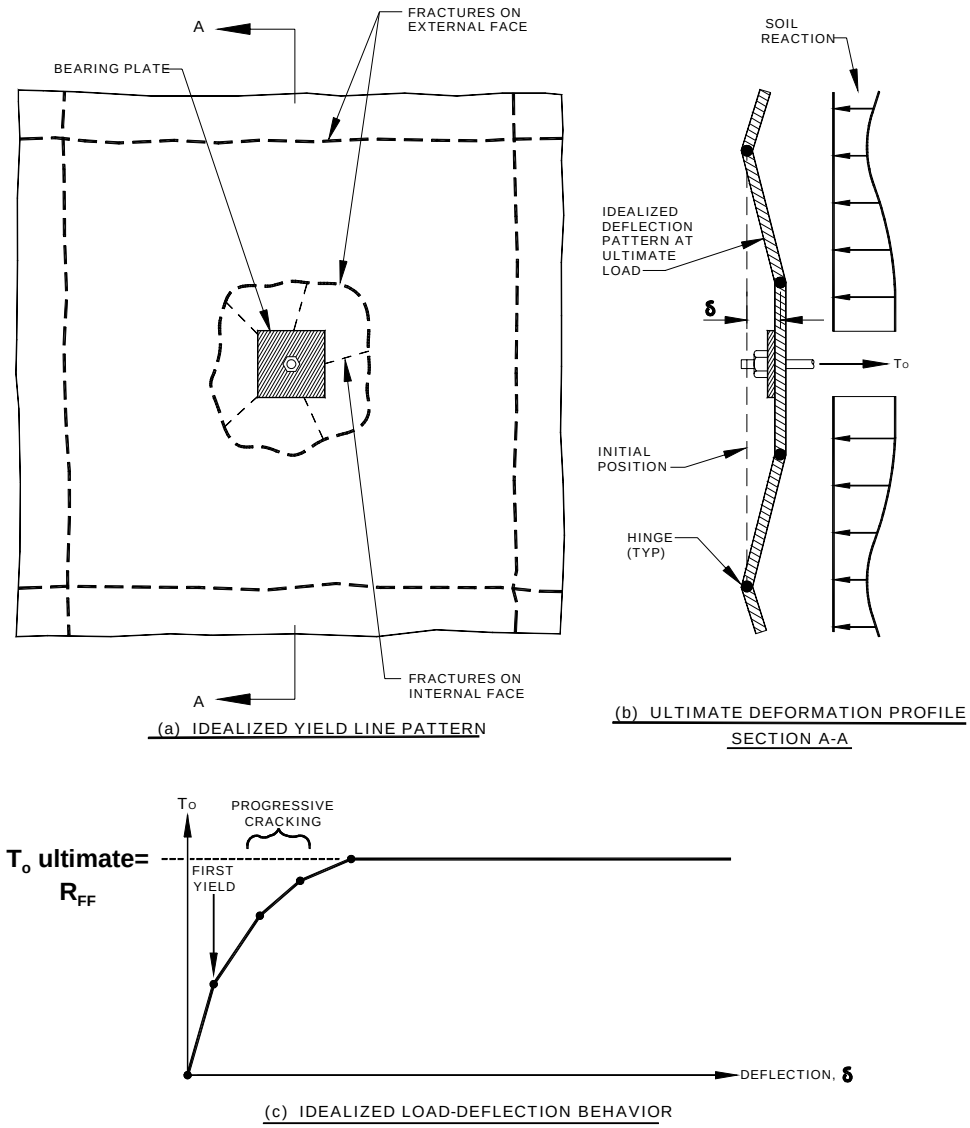


Figure 5.18: Progressive Flexural Failure in Wall Facings.

5.6.3.2 Facing Flexural Capacity

As with other reinforced concrete/shotcrete structures, flexural failure is achieved progressively. After the first yield of the facing section (Figure 5.18c), progressive cracking takes place on both sides of the facing as the lateral earth pressure increases. As the lateral pressure increases, fractures grow and deflections (δ) and nail tensile forces increase. Individual fractures indicate where the flexural capacity is achieved. Eventually, an ultimate stage of the structure is achieved when all fractures connect, act as hinges, and form a mechanism referred to as the critical yield line pattern. Yield line patterns are dependent on various factors including the soil lateral pressures, horizontal and vertical nail spacing, size of bearing plate, facing thickness, reinforcement layout, and concrete strength (Seible, 1996) and are associated with a maximum soil pressure.

In theory, the soil pressure that causes facing failure (i.e., the critical yield line pattern) can be applied to an influence area around the nail head, and a nail tensile force ("reaction") is obtained. This force is designated as the facing flexure capacity, R_{FF}, and is related to the flexural capacity per unit length of the facing. The flexural capacity per unit length of the facing is the maximum resisting moment per unit length that can be mobilized in the facing section. Based on yield-line theory concepts, R_{FF} can be estimated as the minimum of:

$$R_{FF}\,[\mathrm{kN}] = \frac{C_F}{265} \times (a_{vn} + a_{vm})\,[\mathrm{mm^2/m}] \times \left(\frac{S_h\,h[\mathrm{m}]}{S_v}\right) \times f_y\ [\mathrm{MPa}] \qquad \text{(Equation 5.42a)}$$

$$R_{FF}\,[\mathrm{kN}] = \frac{C_F}{265} \times (a_{hn} + a_{nm})\,[\mathrm{mm^2/m}] \times \left(\frac{S_v\,h[\mathrm{m}]}{S_H}\right) \times f_y\ [\mathrm{MPa}] \qquad \text{(Equation 5.42b)}$$

$$R_{FF}\,[\mathrm{kip}] = 3.8 \times CF \times (a_{vn} + a_{vm})\ [\mathrm{in^2/ft}] \times \left(\frac{S_H\,h[\mathrm{ft}]}{S_v}\right) \times f_y\ [\mathrm{ksi}] \qquad \text{(Equation 5.43a)}$$

$$R_{FF}\,[\mathrm{kip}] = 3.8 \times CF \times (a_{hn} + a_{hm})\ [\mathrm{in^2/ft}] \times \left(\frac{S_v\,h[\mathrm{ft}]}{S_H}\right) \times f_y\ [\mathrm{ksi}] \qquad \text{(Equation 5.43b)}$$

where:

C_F = factor that considers the non-uniform soil pressures behind the facing (Byrne et al., 1998);
h = thickness of facing (Figure 5.19);
d = half-thickness of facing;
a_{vn} = reinforcement cross sectional area per unit width in the vertical direction at the nail head;
a_{vm} = reinforcement cross sectional area per unit width in the vertical direction at midspan;
a_{hn} = reinforcement cross sectional area per unit width in the horizontal direction at the nail head;
a_{hm} = reinforcement cross sectional area per unit width in the horizontal direction at midspan;
S_H = nail horizontal spacing;
S_V = nail vertical spacing;
f_y = reinforcement tensile yield strength; and
f_c' = concrete compressive strength.

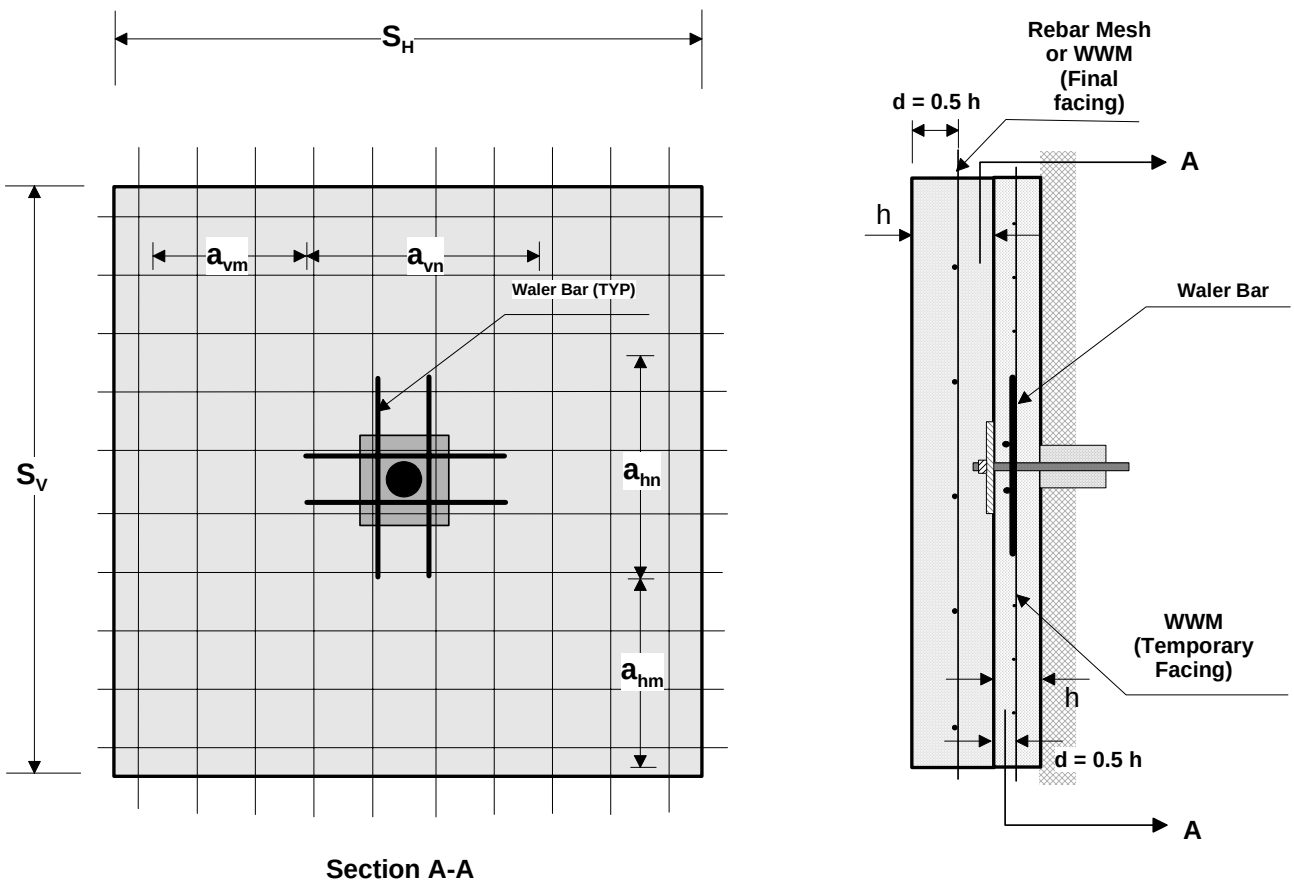


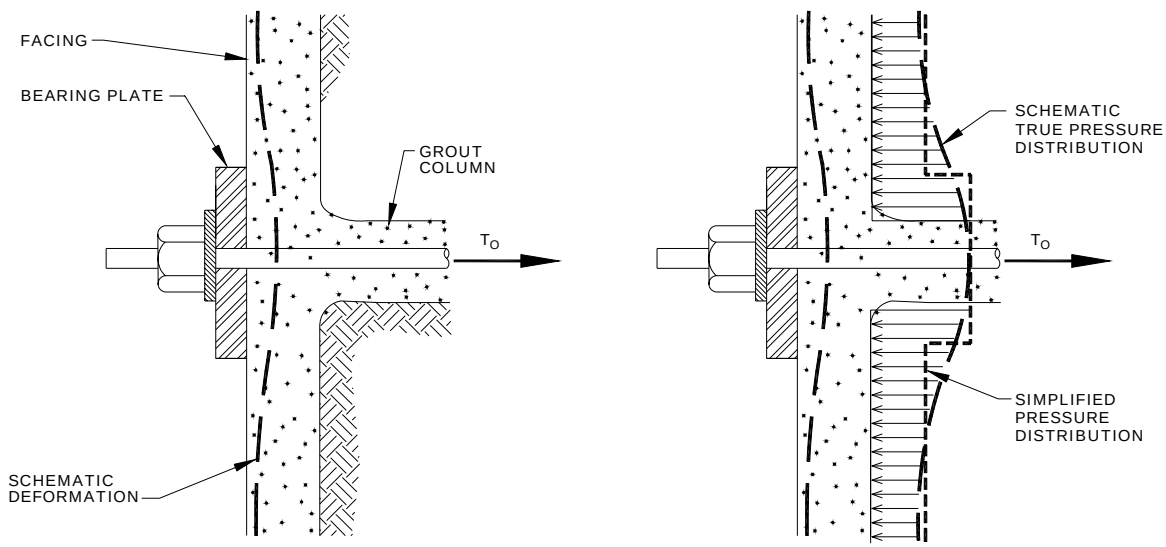
Figure 5.19: Geometry used in Flexural Failure Mode.

The factor C_F takes into account the non-uniform soil pressures behind the facing (Byrne et al., 1998) and represents nominally the ratio of soil pressure behind the nail to soil pressure in the midspan between nails. The soil pressure distribution behind the facing is generally non-uniform. Soil pressure is affected by soil conditions and the facing stiffness, which in turn affects the wall displacement. In the midspan between nails, the displacement of the facing occurs outward and the lateral earth pressure is relatively low. Around the nail heads, the soil pressure is larger than the soil pressure at midspan between nails. Figure 5.20 shows a schematic soil pressure distribution in the vicinity of a soil nail.

The pressure distribution in the facing also depends on the stiffness of the facing. When the facing is relatively thin (as with typical temporary facings), the facing stiffness is relatively low, causing the facing to deform in the midspan sections. As a result, the soil pressure tends to be relatively low in the midspan sections. When the facing is relatively thick, the facing stiffness increases and the resulting wall deformations are smaller than would result from a thin wall facing. As a result of the increased wall stiffness, the soil pressure is more uniform throughout. Table 5.1 shows factors (C_F) for typical facing thickness. For all permanent facings and "thick" [≥ 200 mm (8 in.)] temporary facings, the soil pressure is assumed to be relatively uniform.

Table 5.1: Factors C_F.

Type of Structure	Nominal Facing Thickness mm (in.)	Factor C_F
Temporary	100 (4)	2.0
	150 (6)	1.5
	200 (8)	1.0
Permanent	All	1.0



Modified after Byrne et al. (1998).

Figure 5.20: Soil Pressure Distribution Behind Facing.

In Equations 5.42a and 5.42b, it is assumed that the maximum moments in the facing are around a horizontal axis and the design of reinforcement in the vertical direction is more critical than the design of the horizontal reinforcement. In practice, the cross section area of reinforcement in the horizontal direction is the same as for the vertical direction (i.e., $a_{hm} = a_{vn}$ and $a_{hm} = a_{vm}$); therefore, the most critical case is the one that gives the minimum of S_H/S_V and S_V/S_H.

When the same nail spacing and reinforcement are used in the horizontal and vertical directions, and 420 MPa steel (Grade 60) is used, Equations 5.42 and 5.43 simplify as:

$$R_{FF}\ [\text{kN}] = 1.6 \times C_F \times (a_{vn} + a_{vm})\,[\text{mm}^2/\text{m}] \times h[\text{m}] \qquad \text{(Equation 5.44)}$$

$$R_{FF}\ [\text{kip}] = 230 \times C_F \times (a_{vn} + a_{vm})\,[\text{in}^2/\text{ft}] \times h[\text{ft}] \qquad \text{(Equation 5.45)}$$

These equations can be used for temporary or permanent facing. The thickness of the temporary concrete facing is generally conservatively disregarded when evaluating the flexural capacity of the permanent facing as shown in Figure 5.19. For example, consider a permanent facing of h = 200 mm (8 in.) consisting of rebar mesh with f_y = 420 MPa (60 ksi). The mesh consists of rebar No. 16 @ 300 mm (No. 5 @ 12 in.) each way, with a cross sectional area per unit length of $a_{vn} = a_{vm}$ = 199 mm^2/m (0.31 in^2/ft) (see Table A-2 in Appendix A). For a coefficient C_F = 1.0 for permanent facing (see Table 5.1) and nails equally spaced in the horizontal and vertical directions, it results:

$$R_{FF} = 3.8 \times 1.0 \times (0.31 + 0.31)\ [in^2/ft] \times 60\ [ksi] \times 0.67\ [ft] = 94\ [kips].$$

If (vertical) waler bars are used over the nail heads, the total reinforcement area per unit length in the vertical direction can be calculated as:

$$a_{vn} = a_{vm} + \frac{A_{vw}}{S_H} \qquad \text{(Equation 5.46)}$$

where A_{vw} is the total cross sectional area of waler bars in the vertical direction. Similar concepts can be applied along the horizontal direction. If rebar is used in permanent facings instead of WWM, the total area of reinforcement must be converted to a per unit length basis as:

$$a_{vm} = \frac{A_{vm}}{S_H} \qquad \text{(Equation 5.47)}$$

where A_{vm} is the total cross sectional area of rebar reinforcement in the vertical direction (see Figure 5.19).

Given the tensile force at the soil nail head, T_o, and the facing flexure capacity, the safety factor against facing flexural failure can be defined.

$$FS_{FF} = \frac{R_{FF}}{T_o} \qquad \text{(Equation 5.48)}$$

In general, a minimum factor of safety of 1.35 is adopted for static loads in temporary walls and 1.5 for static loads in permanent walls.

5.6.3.3 Limit Reinforcement in Facing

As with other reinforced concrete structures, the quantity of reinforcement placed in the facing of soil nail wall generally falls within prescribed limits. The amount of reinforcement can be expressed as the reinforcement ratio (ρ or ρ_{ij}) in the "i" direction (vertical or horizontal) and at location "j" (nail head or midspan):

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100 \qquad \text{(Equation 5.49)}$$

where a_{ij} is the cross sectional area of reinforcement per unit width in the "i" direction and "j" location. The minimum reinforcement ratio, expressed in SI and English units, typically occurs in midspan and is defined as:

$$\rho_{min}[\%] = 20\,\frac{\sqrt{f_c^{'}[MPa]}}{f_y[MPa]} \qquad \text{(Equation 5.50)}$$

$$\rho_{min}[\%] = 0.24\,\frac{\sqrt{f_c^{'}[psi]}}{f_y[ksi]} \qquad \text{(Equation 5.51)}$$

The maximum reinforcement ratio is recommended to be:

$$\rho_{max}[\%] = 0.5\,\frac{f_c^{'}[MPa]}{f_y[MPa]}\left(\frac{600}{600 + f_y[MPa]}\right) \qquad \text{(Equation 5.52)}$$

$$\rho_{max}[\%] = 0.05\,\frac{f_c^{'}[psi]}{f_y[ksi]}\left(\frac{90}{90 + f_y[ksi]}\right) \qquad \text{(Equation 5.53)}$$

Therefore, the placed reinforcement must be: $\rho_{min} \leq \rho \leq \rho_{max}$. In addition, the ratio of the reinforcement in the nail and midspan zones should be less than 2.5 to ensure comparable ratio of flexural capacities in these areas.

5.6.4 Punching Shear Capacity

5.6.4.1 Introduction

Punching shear failure of the facing can occur around the nail head and must be evaluated at:

1) bearing-plate connection (used in temporary facings), and

2) headed-stud connection (commonly used in permanent facings).

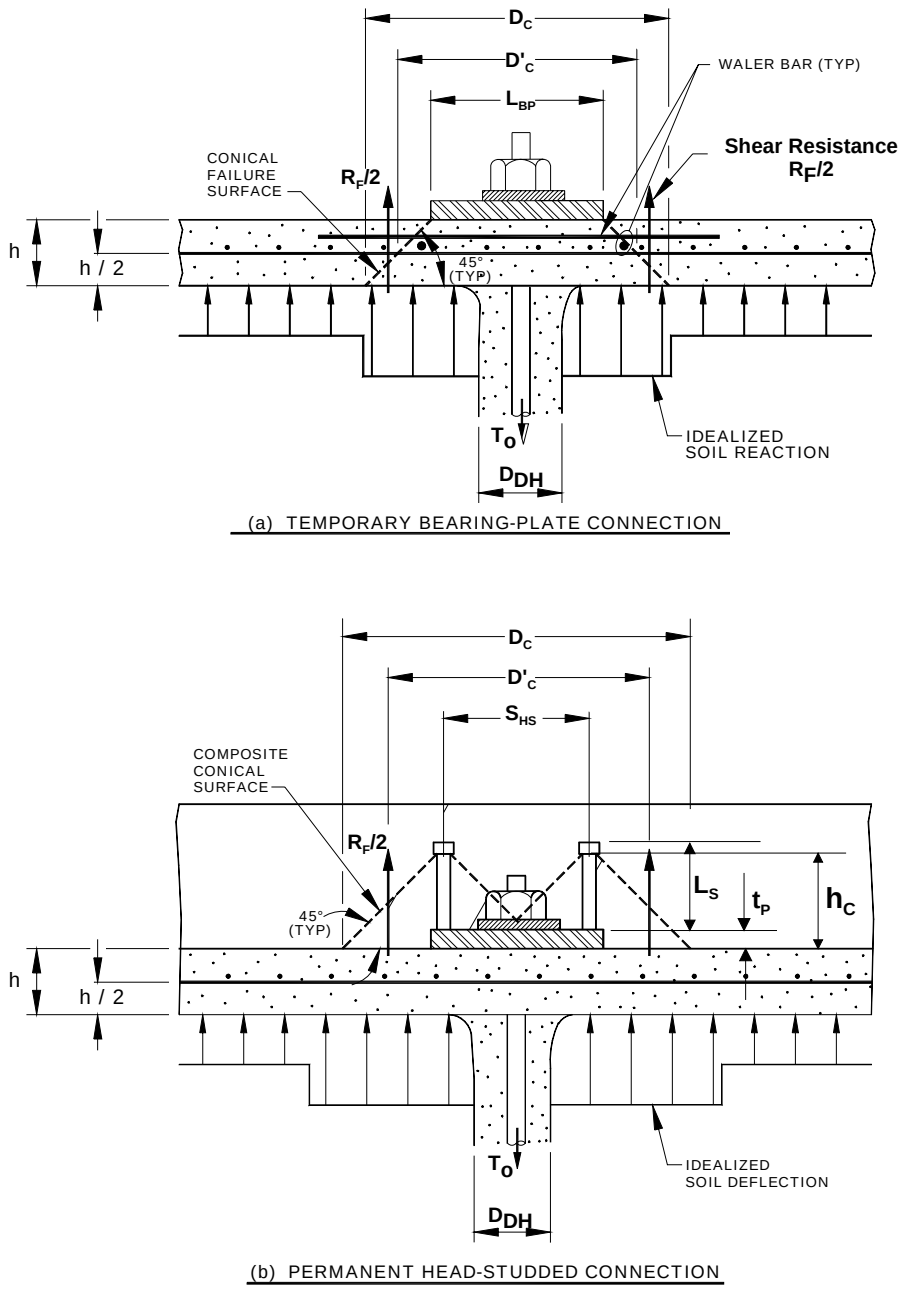
As the nail head tensile force increases to a critical value, fractures can form a local failure mechanism around the nail head. This results in a conical failure surface, as shown in Figure 5.21. This failure surface extends behind the bearing plate or headed studs and punches through the facing at an inclination of about 45 degrees, as shown schematically in Figure 5.21. The size of the cone depends on the facing thickness and the type of the nail-facing connection (i.e., bearing-plate or headed-studs).

5.6.4.2 Facing Punching Shear Capacity

As is common for concrete structural slabs subjected to concentrated loads, the nail-head capacity (Figure 5.21) must be assessed in consideration of the punching shear capacity, R_{FP}, and can be expressed as:

$$R_{FP} = C_P V_F \qquad \text{(Equation 5.54)}$$

where V_F is the punching shear force acting through the facing section and C_P is a correction factor that accounts for the contribution of the support capacity of the soil.



Modified after Byrne et al. (1998).

Figure 5.21: Punching Shear Failure Modes.

The punching shear force can be calculated considering both SI and English units using standard equations for punching shear. These equations consider the size of a conical failure surface (with diameter D'_C at the center of the facing and height h_C, as shown in Figure 5.21) at the level of the concrete slab as:

$$V_F\,[\text{kN}] = 330\,\sqrt{f_c'\,[\text{MPa}]}\;\pi\;D_c'[\text{m}]\,h_c[\text{m}] \qquad \text{(Equation 5.55)}$$

$$V_F\,[\text{kip}] = 0.58\,\sqrt{f_c'\,[\text{psi}]}\;\pi\;D_c'[\text{ft}]\,h_c[\text{ft}] \qquad \text{(Equation 5.56)}$$

where:

D'_C = effective diameter of conical failure surface at the center of section (i.e., an average cylindrical failure surface is considered); and

h_C = effective depth of conical surface.

The correction factor C_P is used to take into account the effect of the soil pressure behind the facing that acts to stabilize the cone. If no subgrade reaction is considered, $C_P = 1.0$. When the soil reaction is considered, C_P can be as high as 1.15. For practical purposes, the correction is usually omitted and this is considered as $C_P = 1.0$.

These equations can be used for both temporary and permanent facing. However, the size of the conical surface (values of D'_C and h_C) must be adjusted to consider the specific type of facing. For the temporary facing, the dimensions of the bearing plate and facing thickness must be considered. For the permanent facing, the dimensions of the headed-studs (or anchor bolts) must be considered. Figure 5.22 shows details of a typical headed-stud connector. Table A.6 in Appendix A contains characteristics of typical headed-stud connectors.

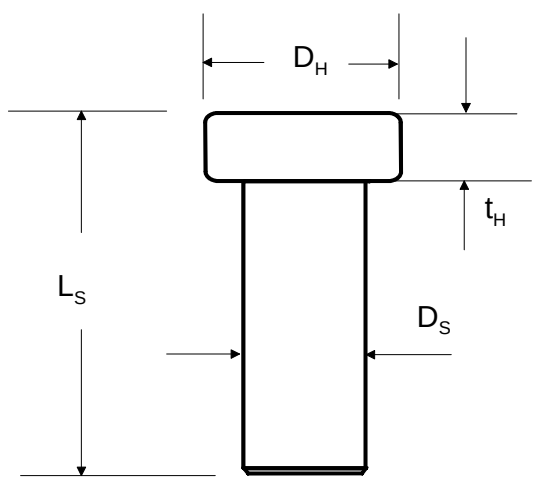


Figure 5.22: Geometry of a Headed-Stud.

The effective size of the conical surfaces must be considered as follows:

Temporary facing (Figure 5.21a)

$$D'_C = L_{BP} + h \qquad \text{(Equation 5.57a)}$$

$$h_C = h \qquad \text{(Equation 5.57b)}$$

where L_{BP} = bearing plate length, h = temporary facing thickness.

Permanent facing (Figure 5.21b)

$$D'_C = \text{min of } (S_{HS} + h_C \text{ and } 2h_C) \qquad \text{(Equation 5.58a)}$$

$$h_C = L_S - t_S + t_P \qquad \text{(Equation 5.58b)}$$

where: S_{HS} = headed-stud spacing, L_S = headed-stud length, t_H = headed-stud head thickness, and t_P = bearing plate thickness.

Given the tensile force at the soil nail head, T_o, (Figure 5.21) and the punching shear capacity of the facing, R_{FP}, the safety factor against facing punching shear (FS_{FP}) can be defined as:

$$FS_{FP} = \frac{R_{FP}}{T_o} \qquad \text{(Equation 5.59)}$$

In general, a minimum factor of safety of 1.35 is adopted for static loads in temporary walls and 1.5 for static loads in permanent walls.

5.6.5 Headed-Stud Tensile Capacity for Permanent Facing

The tensile capacity of the headed-studs (or anchor bolts) connectors providing anchorage of the nail into the permanent facing must be verified, as shown in Figure 5.16. The nail head capacity against tensile failure of the headed-studs, R_{HT}, is computed as:

$$R_{HT} = N_H \; A_{SH} \; f_y \qquad \text{(Equation 5.60)}$$

where:

N_H = number of headed-studs (or anchor bolts) in the connection (usually 4);
A_{SH} = cross-sectional area of the headed-stud shaft; and
f_y = tensile yield strength of the headed-stud.

Given the tensile force at the soil nail head T_o (Figure 5.21) and the tensile capacity of the headed-studs, R_{HT}, the factor of safety against tensile failure of the headed-studs (FS_{HT}) can be defined as:

$$FS_{HT} = \frac{R_{HT}}{T_o} \qquad \text{(Equation 5.61)}$$

For static loads, the minimum factors of safety are 1.8 in temporary walls for steel A307 (or 1.5 for A325), and 2.0 in permanent walls (or 1.7 for A325) (Byrne et al., 1998). The headed studs may also exert excessive compressive stress on the concrete bearing surface. The compression on the

concrete behind the head of the headed-stud is assured to be within tolerable limits if the following geometric constraints are met (ACI, 1998):

$$A_H \geq 2.5\ A_{SH} \qquad \text{(Equation 5.62)}$$

$$t_H \geq 0.5\ (D_H - D_S) \qquad \text{(Equation 5.63)}$$

where:

A_H = cross-sectional area of the stud head;
t_H = head thickness;
D_H = diameter of the stud head; and
D_S = diameter of the headed-stud shaft.

To provide sufficient anchorage, headed-stud connectors should be extended at least to the middle of the section, while maintaining 50 mm (2 in.) minimum cover. To provide additional anchorage capacity, the headed studs should be long enough that the head is located behind the reinforcement.

When threaded bolts are used in lieu of headed-stud connectors, the effective cross-sectional area of the bolts must be employed in the equations above. The effective cross-sectional area, A_E, of threaded anchors is computed as follows:

$$A_E = \frac{\pi}{4}\left[D_E - \left(\frac{0.9743}{n_t}\right)\right]^2 \qquad \text{(Equation 5.64)}$$

where:

D_E = effective diameter of the bolt core; and
n_t = number of threads per unit length.

The evaluations contained in this section as related to facing failure modes are presented in a step-by-step methodology in Chapter 6.

5.7 DEFORMATION BEHAVIOR OF SOIL NAIL WALLS

5.7.1 Wall Displacements

During construction and after its completion, a soil nail wall and the soil behind it tend to deform outwards. The outward movement is initiated by incremental rotation about the toe of the wall, similar to the movement of a cantilever retaining wall. Most of the movement occurs during or shortly after excavation of the soil in front of the wall. Post construction deformation is related to stress relaxation and creep movement, which are caused by post-construction moderate increases in tensile force in the soil nail described previously. Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements (i.e., settlements) of the wall at the facing are generally small, and are on the same order of magnitude as the horizontal movements at the top of the wall. In general, horizontal and vertical displacements of the facing depend on the following factors:

- wall height, H, (deformation increases approximately linearly with height);
- wall geometry (a vertical wall produces more deformation than a battered wall);
- the soil type surrounding the nails (softer soil will allow more deformation);
- nail spacing and excavation lift heights (larger nail spacing and thicker incremental excavation lifts generate more deformation);
- global factor of safety (smaller FS_G's are associated with larger deformation);
- nail-length-to wall-height ratio (shorter nail lengths in relation to the wall height generates larger horizontal deformation);
- nail inclination (steeper soil nails tend to produce larger horizontal deformation because of less efficient mobilization of tensile loads in the nails); and
- magnitude of surcharge (permanent surcharge loading on the wall increases deformation).

Empirical data show that for soil nail walls with typical L/H between 0.7 and 1.0, negligible surcharge loading, and typical global factors of safety (FS_G) values of 1.5, the maximum long-term horizontal and vertical wall displacements at the top of the wall, δ_h and δ_v, respectively, can be estimated as follows:

$$\delta_h = \left(\frac{\delta_h}{H}\right)_i \times H \qquad \text{(Equation 5.65)}$$

where:

$(\delta_h/H)_I$ = a ratio dependant on the soil conditions "i" indicated in the table below; and
H = wall height.

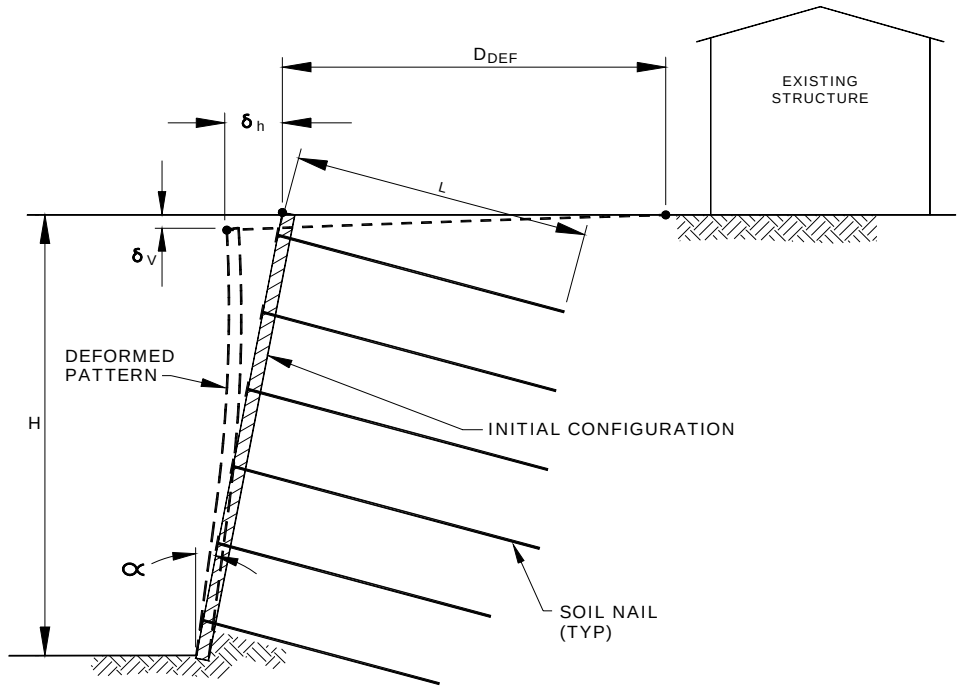
The size of the zone of influence (Figure 5.3), where noticeable ground deformation may take place, is defined by a horizontal distance behind the soil nail wall (D_{DEF}) and can be estimated with the following expression:

$$\frac{D_{DEF}}{H} = C\,(1 - \tan\alpha) \qquad \text{(Equation 5.66)}$$

where α: is the wall batter angle; and C coefficient indicated in Table 5.2.

Table 5.2: Values of $(\delta_h/H)_i$ and C as Functions of Soil Conditions.

Variable	Weathered Rock and Stiff Soil	Sandy Soil	Fine-Grained Soil
δ_h/H and δ_v /H	**1/1,000**	**1/500**	**1/333**
C	**1.25**	**0.8**	**0.7**



Modified after Byrne et al. (1998).

Figure 5.23: Deformation of Soil Nail Walls.

The movements shown above are considered to be relatively small and comparable to those obtained with braced systems and anchored walls. These estimates of deformations have essentially become recommended design values. The adopted tolerable deformation criterion is project-dependent and should consider not only the magnitude of deformation but also the extent of the area behind the wall that may be affected by wall movements. As a first estimate, horizontal deflections greater than 0.005 H during construction should be a cause for concern, as they generally represent an upper limit of acceptable performance.

When excessive deformations are considered to be likely with a certain wall configuration, some modifications to the original design can be considered. Soil nail wall deformations can be reduced by using a battered wall, installing longer nails in the top portion of the wall, using a higher safety factor, or even using ground anchors in conjunction with the soil nails. Additionally, some contractors have used soil nails that are grouted partially along their length and then partially tensioned to mobilize some of the nail tensile strength without soil mass deformation near the wall face. In these cases, after the tensioning is complete, the nails are fully grouted and the shotcrete is applied before the next lift is excavated.

Post-construction monitoring of soil nail wall displacements indicates that movements tend to continue after wall construction, sometimes up to 6 months, depending on ground type. Typically, the post construction deformation increases up to 15 percent of the deformations observed soon after construction. As a result of this movement, additional tension is developed in the nails. In

general, fine-grained soils of high-plasticity (i.e., approximately PI > 20) and high water contents (such that LI > 0.2) tend to incur deformation for longer periods of time.

5.7.2 Permanent Seismic Deformations

Experience has shown that permanent deformations tend to be small when using the typical range of seismic coefficient described earlier (i.e., $0.5 \leq k_h/A_m \leq 0.67$) in the design of relatively flexible soil nail wall systems. However, this is only true as long as the strength of the soil behind the wall is not reduced significantly during seismic events. Permanent deformations of soil nail walls due to a seismic event may be estimated using the Newmark (1965) procedure. As introduced previously, this method consists of calculating the displacement of a potential sliding mass of soil in a manner analogous to the sliding of a rigid block on a ramp subjected to cyclic loading. Additional details of this method can be found elsewhere (Kramer, 1996; Kavazanjian et al., 1997).

5.8 COMPUTER PROGRAMS FOR SOIL NAIL WALLS

5.8.1 Introduction

Several computer programs have been developed for the analysis and design of soil nail walls. Also, numerous general slope stability programs have added the capability to model various types of soil reinforcement (including the use of soil nails and ground anchors). Although soil nails can be considered in the majority of in-use general slope stability programs, the design of soil nail walls and, in particular, the nail lengths, is not as straightforward in these programs as it is in programs specifically developed for soil nail applications.

The two computer programs most commonly used in the United States for the design of soil nail walls are SNAIL and GOLDNAIL. The main features of these programs are described below. Appendix E contains a user's guide for data input and use of SNAIL.

5.8.2 SNAIL Computer Program

SNAIL is a DOS-based (Version 3.09) computer program developed by the California Department of Transportation (CALTRANS) in 1991. The program is based on two-dimensional limit equilibrium that considers force equilibrium only. The failure surface is bi-linear (with the failure surface originating at the toe) or tri-linear (with the failure surface originating at the bottom of the excavation at a point away from the toe). For the case of a tri-linear failure surface, the resisting forces in the lower wedge beneath the wall are calculated assuming passive earth pressure conditions, with the inclination of the passive force fixed at the mobilized friction angle. The methods included in SNAIL only consider force equilibrium. Therefore, in general, although interblock forces are in equilibrium, moment equilibrium is generally not simultaneously achieved with this method. The program allows the user to specify an area in which the program searches for the most critical surface. The search routine is performed at 10 nodes of the search width previously defined and subsequently tries 56 surfaces at each node. While the total number of searched surfaces is 560, SNAIL reports the 10 most critical on the screen and in an output file.

Although the program is not Windows-based, data entry is easy and efficient. SNAIL can model up to seven soil layers. Up to two slope segments can be modeled at the toe. The phreatic or piezometric surface is defined only by up to three points. SNAIL allows up to two uniform (vertical) surcharge distributions and an internal or external force (horizontal or oblique). Therefore, it is possible to model a soil nail wall with an added ground anchor. The soil layers are modeled as lines with end points, whose coordinates are entered by the user. The program has limitations in modeling complex stratigraphy but is adequate to assist analyze and design for a wide range of simple conditions and geometries. For complex wall and ground geometry and loading conditions, the designer may need to make simplifications due to the limitations of the program.

The necessary input related to the reinforcement includes the location, diameter, inclination, vertical and horizontal spacing, and tensile strength of the nails. These data can be easily assigned to all nails at once (i.e., uniform nail characteristics) or individually for each nail. The necessary input data for the soil parameters are the unit weight, the ultimate shear strength, the bond strength, and the bond strength reduction factor (BSRF), which was developed to easily and selectively reduce values of bond strength for some nails. The BSRF may be interpreted as the inverse of the factor of safety for the bond strength or pullout capacity (FS_P). When associated to the FS_P, BSRF varies typically between 0.5 and 0.7. Other input parameters considered in the program include the nail bar cross-sectional area, yield strength of the nail bar, and the facing punching shear capacity. Pseudo-static seismic analysis can be performed in SNAIL by entering the horizontal and vertical seismic coefficients discussed earlier in this chapter.

The soil strength criterion used in SNAIL is the conventional linear Mohr-Coulomb envelope. Parameters can be entered in either English or Metric units. Bond strength input is associated with the soil input, not with the nail input. Hence, if different bond strengths need to be modeled in an otherwise homogeneous soil profile, a new soil layer must be defined. Although the program has been developed specifically for soil nail walls, it can also calculate safety factors for the stability of unreinforced and MSE walls and slopes, as well as ground anchor walls, as described earlier. Also, because the program allows for failure surfaces to go beneath the wall and daylight in the excavation, a rough estimate of the bearing stability can be performed with this program when failure surface beneath the wall are considered. Sliding analyses with SNAIL are approximate because the program is limited to only one planar surface beneath the wall and the soil's passive resistance at this location is calculated simplistically with the Rankine formulation.

The designer can change the input parameters interactively and rerun the analysis until the design criteria for nail locations, diameters, and lengths are met. Results are presented graphically on the screen and in output files. The output screen contains minimal information. The output files provide the forces for each nail and for each of the 10 most critical failure surfaces analyzed. A report of the most critical failure modes is provided in the output file. The program also provides the average nail force corresponding to FS_G =1, which is used in the calculation of the design nail tensile forces. This average is calculated by the program for the case with FS_G =1 (i.e., full mobilization of soil strength) by dividing the sum of all soil nail forces by the total number of nails.

In summary, the program calculates the global factor of safety, F_{SG} and determines the controlling failure mode (either global stability, facing, or nail tensile failures) if the actual facing capacity (i.e., minimum of R_{FF}, R_{FP}, or R_{HT}) and nail bar tensile strength (FS_T) are entered.

The great advantage of this program is that it is public domain, it can be readily downloaded from the Internet (http://www.dot.ca.gov/hq/esc/geotech/request.htm), and is free of charge. There is, however, limited technical support available from CALTRANS to support the program.

5.8.3 GOLDNAIL Computer Program

GOLDNAIL version 3.11 is a versatile Windows-based proprietary program developed in 1993-1998 by Golder Associates. Data can be input with relative ease because they are organized in tables, which facilitates error checking on the screen. The program works in three modes: (1) design, (2) factor of safety, and (3) nail service load modes. In the design mode, a trial run is initiated; subsequently, the program can modify the nail properties (i.e., bond strength, length, diameter, and spacing) until a desired safety factor is achieved. In the factor of safety mode, the global factor of safety is calculated for specified input parameters. In the nail service load design mode, the program provides the maximum in-service tensile forces in the soil nails [according to the Allowable Stress Design (ASD) method] to design the nail bar diameter and facing characteristics.

The program can model up to 13 soil layers, complex slopes and subsurface geometries, horizontal and vertical surcharge distributions, groundwater, and pseudo-static horizontal coefficients. Nail and soil parameters are similar to those described in the previous section. A limitation of the program is that all nails must have the same spacing and inclination. Although this scenario is typical for most designs, this assumption may be too restrictive for some cases. Another limitation of the program is that block/wedge geometries are not considered; only circular failure surfaces are available. In addition, the failure circles can only pass at or above the toe. Therefore, sliding and bearing capacity cannot be assessed with this program, and the analyst must resort to other procedures or computer programs to evaluate sliding and bearing capacity, if deemed necessary.

The soil strength criterion is a linear Mohr-Coulomb envelope with the option of using a bi-linear strength envelope. This option is useful because it may result in a closer modeling of typical non-linear soil behavior as it allows the user to specify a lower friction angle under higher confinement pressure, compared to the values anticipated at lower confining pressures. GOLDNAIL can also be used to analyze unreinforced slopes and walls and ground anchor walls. For the unreinforced case, the program can handle various pressure distributions acting on the wall face.

This program satisfies moment and force equilibrium. Similar to conventional slope stability methods, GOLDNAIL divides the potential sliding mass into vertical slices. The program modifies iteratively the normal stresses distribution at the base of the slices until force and moment equilibrium is obtained. Input data can be entered in English, SI, or any compatible unit system. The program allows the consideration of factored (reduced) strengths when the Load and Resistance Factor Design (LRFD) method is selected and safety factors when the ASD method is used.

Results are presented on the screen and in output files. The graphical information is limited to the minimum calculated factor of safety and its associated critical circle. Output files provide the maximum nail forces and report the controlling type of soil nail failure mode. The cost of the GOLDNAIL program is around $1,000 (December 2000) and a fee is charged for technical support.

5.8.4 Comparison of SNAIL and GOLDNAIL

For the purposes of this document, two cases have been analyzed using SNAIL and GOLDNAIL to compare their performances. Figure 5.24 shows the geometry and soil stratigraphy used for the design of two soil nail walls.

Soil unit weight	γ	120 pcf
Soil friction angle	ϕ'	32 degrees
Soil cohesion	c	100 psf
Drillhole Diameter	D_{DH}	4 inch
Bond Ultimate Strength	q_u	50 psi
Pullout Safety Factor	FS_P	2.0
Nail Bar Length	L	30 ft
Nail Bar Cross-Sectional Area	A_t	1.0 in^2
Steel yield strength	f_Y	60 ksi
Critical Facing Capacities	R_F	100 kip

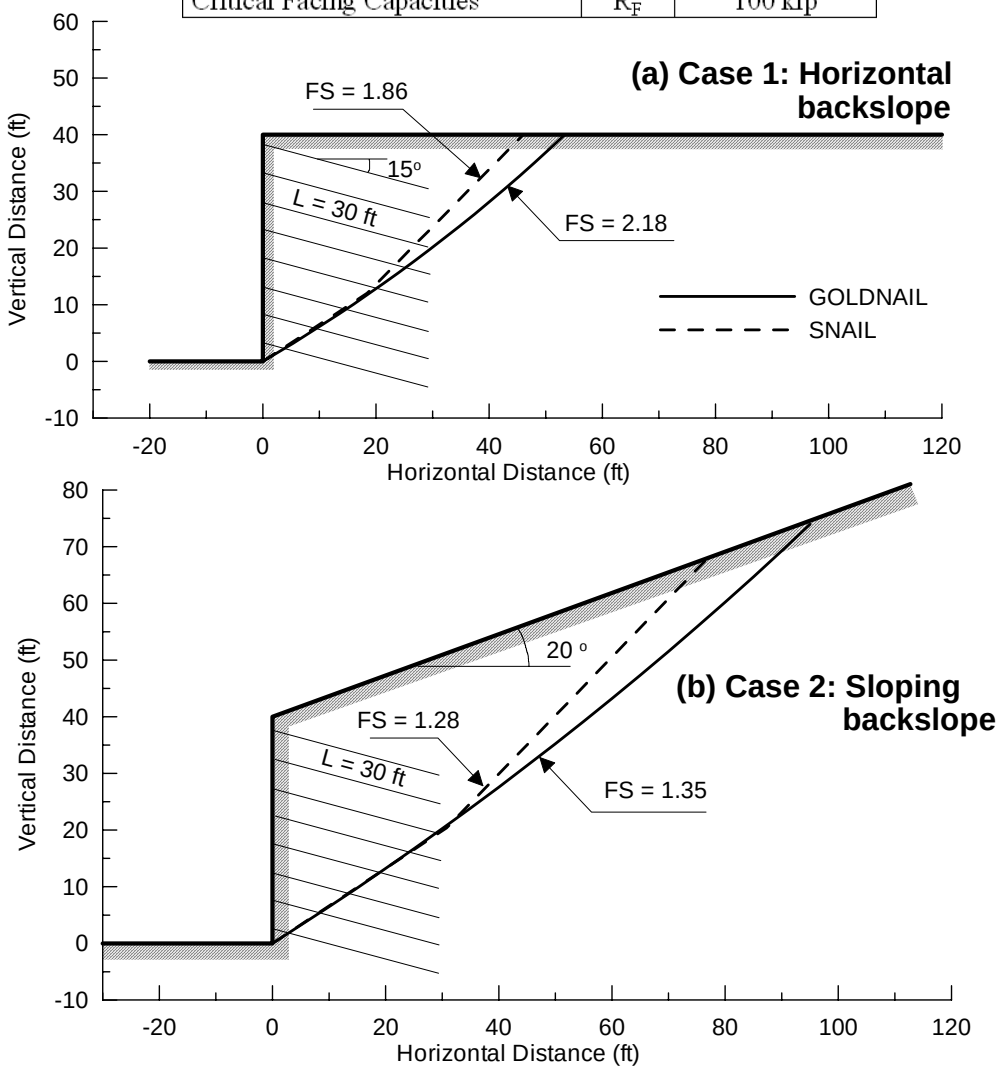


Figure 5.24: Stability Analysis – Comparison of Results Between SNAIL and GOLDNAIL.

The bond strength for this comparison was selected deliberately high to obtain, at least in one of the cases, relatively high safety factors. This strategy provides a wide variation of the calculated factor of safety for the conditions analyzed.

In the first case (Figure 5.24a), a relatively stable wall section was analyzed using relatively high bond strengths and a horizontal backslope. For this case, the computed global factor of safety was 1.86 and 2.18 using SNAIL and GOLDNAIL, respectively. In the second case (Figure 5.24b), a wall section with a sloping backside was considered and the computed factor of safety was 1.28 and 1.35 with SNAIL and GOLDNAIL, respectively. Additional analyses performed using both computer programs and project experience indicate that the factor of safety calculated with these computer programs typically differs by 5 to 10 percent, for the typical range of safety factors between 1.35 and 1.5. Greater variations, up to about 20 percent, in the calculated FS_G occur for factors of safety greater than 1.75, trend which is similar to that observed in earlier comparative studies of soil nailing analysis methods (Banerjee et al., 1998). These results are also comparable to trends that were observed in a more recent study, where the performance of various commercially available software packages for slope stability analysis was evaluated (Pockoski and Duncan, 2000). SNAIL does not always give safety factors that are lower than those obtained with GOLDNAIL, although this was not the case in the comparison shown in Figure 5.24.

One of the reasons attributed to the difference in the calculation results from the two programs is the different assumption of the force distribution along the nail. SNAIL assumes a linear distribution of nail forces, starting from zero at the end of the nail to a maximum value that remains constant from the critical failure surface and the wall facing. In GOLDNAIL, a reduction of the nail force near the wall facing is assumed. However, the factor of safety for global stability is not very sensitive to these distributions as shown by comparative analysis of reinforced soil using different force distributions along the reinforcement (Wright and Duncan, 1992). Both computer programs allow a fast and thorough design of typical situations encountered in soil nail wall applications. The major difference in performance resides in the ability of the programs to model increasingly more complex geometry and soil properties and the ease with which data are input and results interpreted. Despite the difference between the bi-linear (or tri-linear) critical failure surface in SNAIL and the circular failure surfaces considered in GOLDNAIL, the calculated critical surfaces exhibit similar locations as shown in Figure 5.24. For more complex subsurface conditions (e.g., a soft foundation or a weak seam under the soil nail wall), GOLDNAIL cannot be used because the failure surfaces are restricted to locations at or above the excavation. Neither of these programs can analyze composite failure surfaces, which might be applicable when multiple soil layers with dissimilar strengths exist. Also, neither of these programs can simulate cracks at the ground surface.

As mentioned earlier, the great advantage of SNAIL is that it can be readily downloaded from Internet and is free of charge. However, technical assistance is limited.

5.9 RECOMMENDED FACTORS OF SAFETY FOR THE ASD METHOD

This section summarizes and provides recommendations for the minimum safety factor to be used in the design of soil nail walls, for each of the various failure modes described earlier using the ASD method. Recommended factors of safety, modified after Byrne et al. (1998), are given for static and seismic loads, and for temporary and permanent structures (Table 5.3). The

recommended factors of safety are only applicable to the ASD method where loads are unfactored (see Section 5.10).

Table 5.3: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls using the ASD Method.

Failure Mode	Resisting Component	Symbol	Minimum Recommended Factors of Safety		
			Static Loads(1)		Seismic Loads(2) (Temporary and Permanent Structures)
			Temporary Structure	Permanent Structure	
External Stability	Global Stability (long-term)	FS_G	1.35	1.5(1)	1.1
	Global Stability (excavation)	FS_G	1.2-1.3 (2)		NA
	Sliding	FS_{SL}	1.3	1.5	1.1
	Bearing Capacity	FS_H	2.5 (3)	3.0 (3)	2.3 (5)
Internal Stability	Pullout Resistance	FS_P	2.0		1.5
	Nail Bar Tensile Strength	FS_T	1.8		1.35
Facing Strength	Facing Flexure	FS_{FF}	1.35	1.5	1.1
	Facing Punching Shear	FS_{FP}	1.35	1.5	1.1
	H.-Stud Tensile (A307 Bolt)	FS_{HT}	1.8	2.0	1.5
	H.-Stud Tensile (A325 Bolt)	FS_{HT}	1.5	1.7	1.3

Notes: (1) For non-critical, permanent structures, some agencies may accept a design for static loads and long-term conditions with FS_G = 1.35 when less uncertainty exists due to sufficient geotechnical information and successful local experience on soil nailing.

(2) The second set of safety factors for global stability corresponds to the case of temporary excavation lifts that are unsupported for up to 48 hours before nails are installed. The larger value may be applied to more critical structures or when more uncertainty exists regarding soil conditions.

(3) The safety factors for bearing capacity are applicable when using standard bearing-capacity equations. When using stability analysis programs to evaluate these failures modes, the factors of safety for global stability apply.

5.10 LOAD COMBINATIONS

Soil nail walls used on typical highway projects are subjected to different loads during their service life. Typical applied loads are dead loads (e.g., weight of the soil nail wall system, lateral earth pressure, weight of a nearby above-ground structure), traffic loads, impact loads (e.g., vehicle collision on barriers above soil nail wall), and earthquake loads. Table 5.4 presents a list of the load types considered for highway structures by AASHTO (1996).

Several load combinations that include two or more load types must be considered in design to assess the most critical loading condition. Each load combination (or "load group" per AASHTO, 1996) takes into consideration the occurrence of simultaneous loads of different types that a structure may be subjected to during its service life. The effect of a load group (N) can be expressed, in its most general form, as:

$$N = \gamma_n \left(\sum \beta_{in} Q_{in} \right) \qquad \text{(Equation 5.67)}$$

where:

γ_n = load factor for a load group "n";
n = subscript indicating the load group number (defined below);
β_{in} = load combination coefficient for load type "i" and load group "n";
i = subscript indicating the load type (Table 5.4); and
Q_{in} = the load type.

Table 5.4: Typical Load Types.

i	Load Type (Q_i)	Symbol
1	Dead Load (i.e., permanent load)	D
2	Live Load (i.e., transient frequent loads)	L
3	Live Load Impact (i.e., transient infrequent loads)	I
4	Centrifugal Forces on Bridge Deck	CF
5	Earth Pressure	E
6	Buoyancy	B
7	Stream Current Pressure	SF
8	Wind Loads on Structure	W
9	Wind Loads on Live Load	WL
10	Longitudinal Forces on Bridge Deck	LF
11	Rib Shortening, Shrinkage, and Temperature	R+ S+ T
12	Earthquakes	EQ
13	Ice Pressure	ICE

In the ASD method, the loads are unfactored; therefore, the load factors are γ_n =1 regardless of the load type (Table 5.4). Typical combination coefficients are also listed in Table 5.5. For most soil nail wall applications, load groups with static or quasi-static loading (i.e., Load Groups I, II, V) and seismic loading (e.g., Load Group VII), are considered. For seismic loads (Load Group VII), AASHTO (1996) allows increasing the allowable stresses 133 percent from the values obtained with factors of safety for static loads as indicated in the last column of Table 5.5. This approach was followed to develop safety factor for seismic conditions included in Table 5.5.

Table 5.5: Coefficients γ and β[1] for the Allowable Stress Method.

GROUP (N)	γ_n	β_{in} (i = load type, n = Group Number)													% [3]
		D	$(L+I)_n$ [2]	$(L+I)_p$ [2]	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE	
I	1.0	1	1	0	1	1	1	1	0	0	0	0	0	0	100
IA	1.0	1	2	0	0	0	0	0	0	0	0	0	0	0	150
IB	1.0	1	0	1	1	1	1	1	0	0	0	0	0	0	**[4]
II	1.0	1	0	0	0	1	1	1	1	0	0	0	0	0	125
III	1.0	1	1	0	1	1	1	1	0.3	1	1	0	0	0	125
IV	1.0	1	1	0	1	1	1	1	0	0	0	1	0	0	125
V	1.0	1	0	0	0	1	1	1	1	0	0	1	0	0	140
VI	1.0	1	1	0	1	1	1	1	0.3	1	1	1	0	0	140
VII	1.0	1	0	0	0	1	1	1	0	0	0	0	1	0	133
VIII	1.0	1	1	0	1	1	1	1	0	0	0	0	0	1	140
IX	1.0	1	0	0	0	1	1	1	1	0	0	0	0	1	150
X[6]	1.0	1	1	0	0	1[5]	0	0	0	0	0	0	0	0	100

Notes:
(1) Source: AASHTO (1996).
(2) (L + I)n = Live load plus impact for AASHTO Highway H or HS loading.
(L + I)p = Live load plus impact consistent with the overload criteria of the operation agency.
(3) Percentage of increment of allowable stresses (determined with recommended factors of safety).
(4) $\text{Percentage} = \dfrac{\text{Maximum Stress (Operating Rating)}}{\text{Allowable Stress}} \times 100$
(5) $\beta_E = 1.00$ for vertical and lateral loads on all other structures.
(6) Load Group X is for culverts.

5.11 OTHER DESIGN CONSIDERATIONS

5.11.1 Drainage Design Considerations

5.11.1.1 Introduction

As with many other geotechnical projects, surface water runoff and groundwater can cause difficulties during construction, increase the cost and duration of construction, impair long-term integrity, and weaken the performance of soil nail walls. To minimize these complications, surface water runoff and groundwater must be controlled both during and after construction of the soil nail wall. Additionally, it has been shown that soil nail walls perform significantly better when an effective drainage system is installed to control water levels behind the wall. A brief description of the control systems commonly used in soil nail walls is presented below.

5.11.1.2 Surface Water and Groundwater Control

Dewatering measures during construction include, as a minimum, the control of surface water runoff and subsurface flow associated with either perched water or localized seepage areas. A surface water interceptor ditch, excavated along the crest of the excavation and lined with concrete, applied during the shotcreting of the first excavation lift, is a recommended element for controlling surface water flows. Additionally, if the design engineer believes that the groundwater impacts are

localized or short-term conditions, wells or well points installed beyond the length of the nails may be used temporarily to lower the groundwater table. However, this approach may result in much higher construction costs and delays.

5.11.1.3 Long-Term Groundwater and Surface Water Control

Geocomposite Drain Strips. These elements are strips of synthetic material approximately 300 to 400 mm (12 to 16 in.) wide. They are placed in vertical strips against the excavation face along the entire depth of the wall (Figures 4.5 and 5.25). The horizontal spacing is generally the same as the nail horizontal spacing. The lower end of the strips discharges into a pipe drain that runs along the base of the wall or through weep holes at the bottom of the wall. For highly irregular excavation faces, the placement of prefabricated drain strips against the excavated face is difficult and often impractical. In some cases, the prefabricated drain strips may be sandwiched between the shotcrete construction facing and the permanent CIP facing, with the drain placed over 50 to 75 mm (2 to 3 in.) diameter weep holes passing through the construction facing. The design engineer needs to provide explicit construction and inspection guidance for this type application, to assure that the performance of the drainage system is not impacted during installation of the shotcrete. If appropriate performance cannot be guaranteed, the effect of the groundwater table needs to be considered in the analysis.

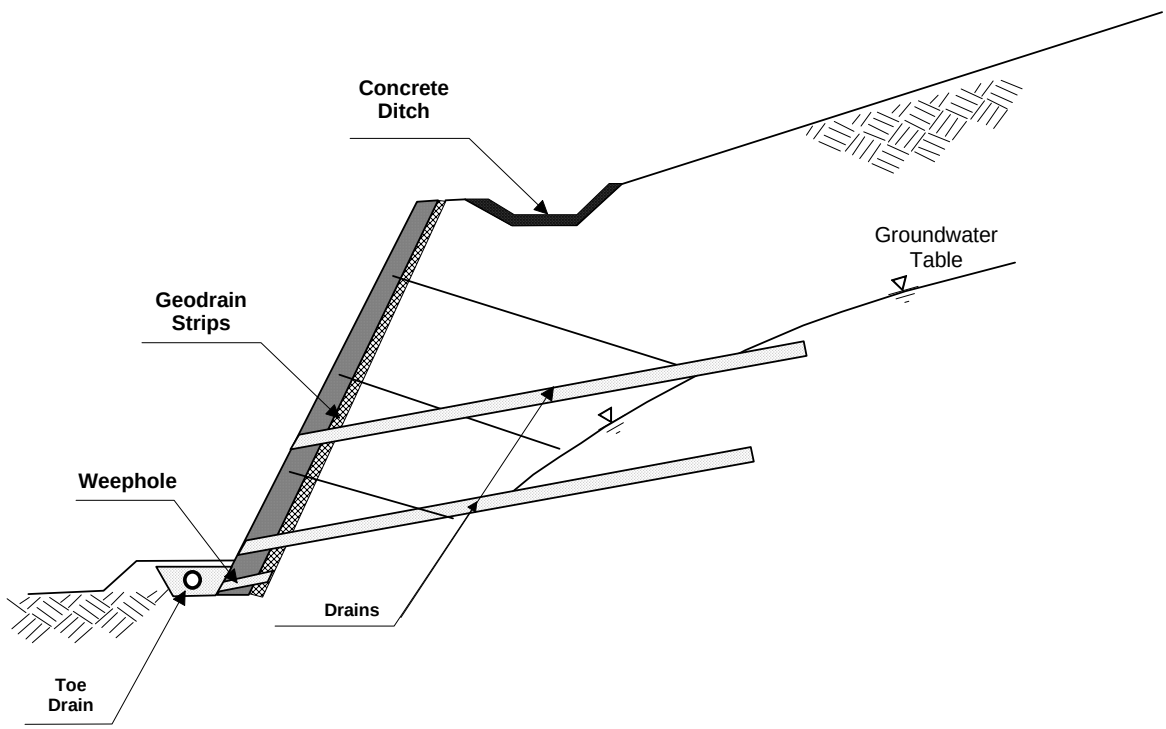


Figure 5.25: Drainage of Soil Nail Walls.

Shallow Drains (Weep Holes). These are typically 300- to 400-mm (12- to 16-in.) long, 50- to 100-mm (2- to 4-in.) diameter PVC pipes discharging through the face and located where localized

seepage is encountered or anticipated. Weep holes are also used as the terminating point of the vertical strip drains to allow any collected water to pass through the wall.

Drain Pipes. Horizontal or slightly inclined drain pipes may be installed where it is necessary to control the groundwater pressures imposed on the retained soil mass. Drain pipes typically consist of 50-mm (2-in.) diameter PVC slotted or perforated tubes, inclined upward at 5 to 10 degrees to the horizontal. Drain pipes are typically longer than the length of the nails and serve to prevent groundwater from being in contact with the nails or the soil nail wall mass, as shown in Figure 5.25. The lengths of the drains depend on the application. To provide drainage of shallow or perched groundwater occurring erratically close to the facing, drain pipes with lengths varying from 0.3 to 0.5 m (1 to 1.6 ft), and in some cases, up to 1 m (3.2 ft) can be installed. They are installed at a density of approximately one drain per 10 square meters (100 square ft) of face. Drain pipes are typically deployed after nail installation to prevent potential intrusion of nail grout into the slotted pipes. The pipes typically exit through the face of the wall.

The PVC pipe should be slotted, as shown in Figure 5.26. Although drain pipes are typically installed after nails are in place and the shotcrete is applied to avoid either grout or shotcrete from entering the drain, they can be applied prior to shotcrete application. In this case, a plug of dry-pack and temporary PVC caps must be used to prevent the shotcrete from coming into the drain hole and obstructing the drain slots or perforations.

Permanent Surface Water Control. Permanent surface water control measures include installing an interception ditch behind the wall to prevent surface water runoff from infiltrating behind the wall or flowing over the wall edge. A vegetative protective cap may be also be used to reduce or retard water infiltration into the soil.

5.11.1.4 Design Considerations

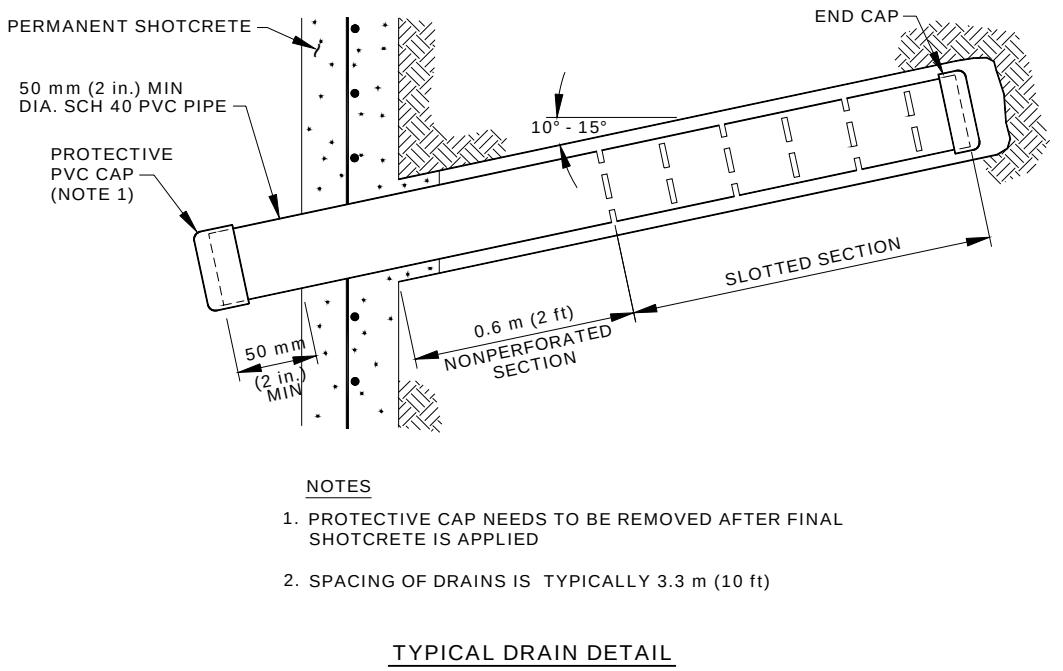
Drain pipes require long-term maintenance. Analysis of soil nail walls for long term conditions may need to take into consideration the potential for clogging. Clogging of horizontal drains and a corresponding increase in water pressure will reduce the factor of safety against global stability and/or sliding, and may adversely impact the internal stability by affecting soil/nail interaction.

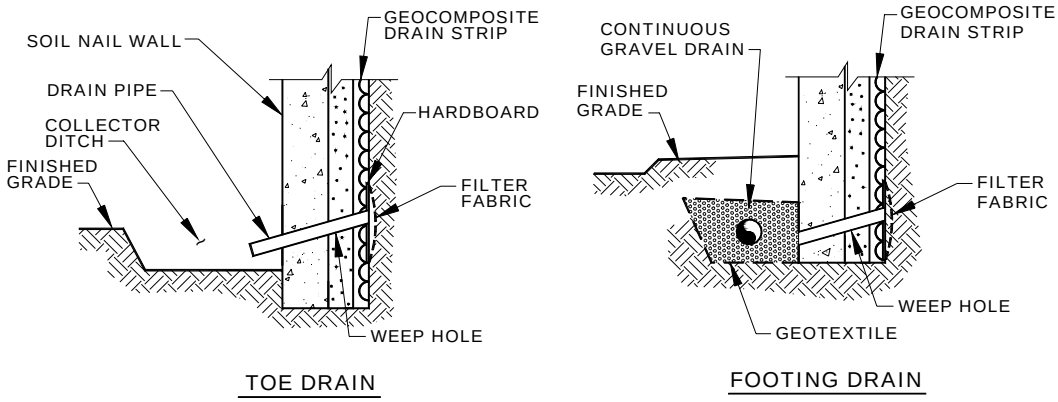
5.11.2 Frost Protection

The formation of ice lenses in the vicinity of the soil nail wall facing in frost-susceptible soils may lead to the development of high loads on both the facing and the head of the nail. This phenomenon may result in damage to the facing (Byrne et al., 1998). In situations where the facing is designed to resist frost damage, the nail or to the connection between the nail and the facing can still be impacted by frost.

The magnitude of the impact to the facing/nail depends on the depth of frost penetration, the intensity and duration of the freeze period, the availability of water, and the stiffness of the facing. Kingsbury et al. (2002) report that the force in the nail head caused by front action can be as high as 2.5 times larger than the maximum seasonal nail force without frost action. Increases in nail and facing loads should be anticipated in areas where frost durations are generally greater than one

week, where frost susceptible soils are encountered near the face, and where the face is in close proximity to a source of water. Soils susceptible to frost action are those exhibiting the following characteristics: (1) more than 3 percent of the solids fraction is smaller than 0.02 mm for non uniform soils (i.e., $C_u > 5$), or (2) more than 10 percent of the solids fraction smaller than 0.02 mm for uniform soils (i.e., $C_u \leq 5$) (Casagrande, 1931). C_u is the uniformity coefficient, which can be obtained from grain size gradation tests (see Table 3.4 in Section 3.5 and ASTM, 2002).

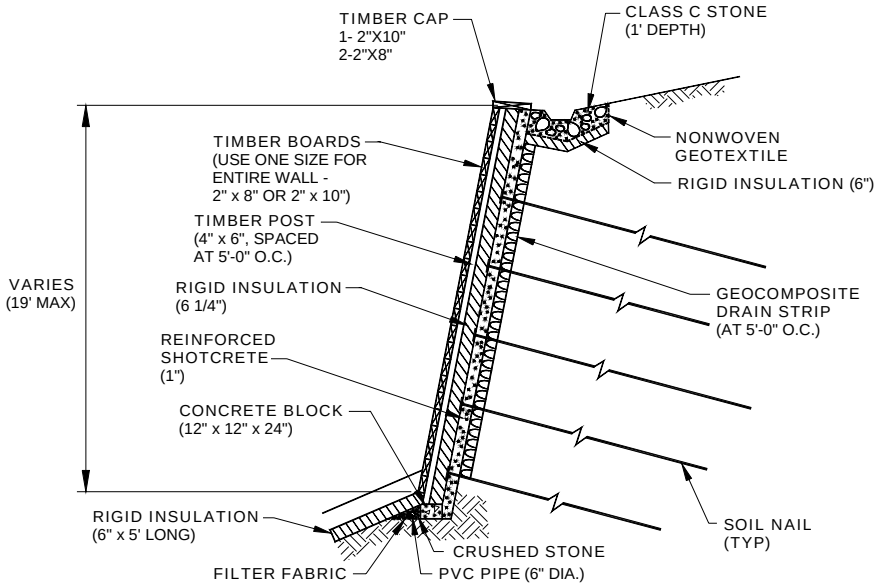




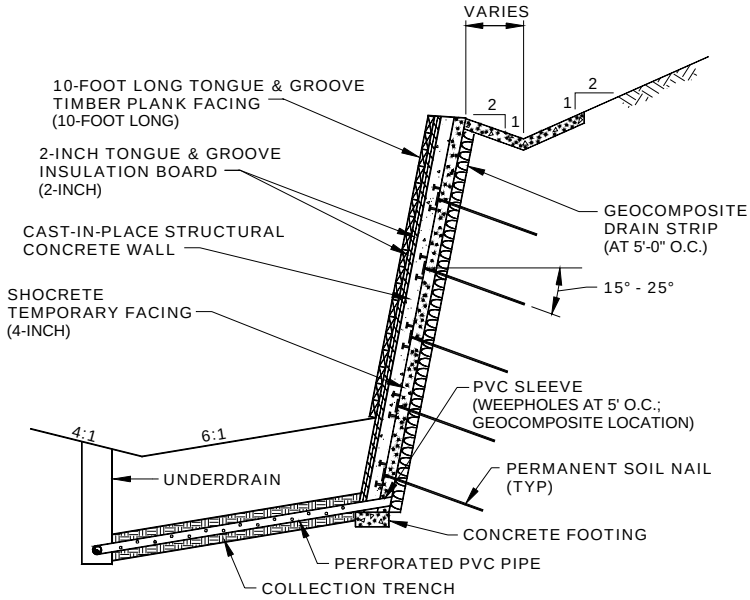
Modified after Byrne et al. (1998).

Figure 5.26: Typical Drain Pipe Details to Provide Groundwater Control in Soil Nail Walls.

In general, frost-loading effects can be eliminated or significantly mitigated by increasing the wall thickness to approximately the depth of frost penetration. This can be done by placing porous backfill (e.g., gravel) or insulating material either between the temporary facing and permanent facing, or outside the permanent facing. Figure 5.27 shows examples of proposed frost protection details using Styrofoam insulation. Because a 25-mm (1-in.) thick Styrofoam insulation board is generally considered to be equivalent to 0.3-m (1-ft) thickness of gravel, appropriate protection against frost action can be obtained with a relatively thin layer of this material.



a) Example of Soil Nail Wall with Frost Protection



b) Example of soil nail wall with Frost Protection

Modified after Byrne et al. (1998).

Figure 5.27: Frost Protection.

5.11.3 External Loads

External loads may be applied at the top of the soil nail wall and may vary from relatively light highway appurtenance loads (e.g., roadway lighting supports) to significant loads (e.g., loads resulting from the integration of a relatively large cantilever retaining structure on top of the wall). For relatively light loading conditions, the external loads can be used to define additional shear forces and flexural moments in the section of the wall above the first row of nails. These loads are then added to the calculated facing loads for subsequent analysis.

For more significant loads (e.g., loads applied by bridge abutments), it may be necessary to perform a full soil-structure interaction analysis to define how the additional facing and nail loads are distributed throughout the entire soil nail structure. The magnitude and distribution of the load transferred to the wall depends on the distance of the load to the wall and the type of load foundation (shallow or deep). The magnitude of these loads can be significantly increased if the structure is subject to seismic forces.

5.11.4 Design of Support for Facing Dead Load

The weight of temporary facing must be supported by the installed nails or other supplementary means until compressive stresses develop at the facing-nail contact. This is particularly important for the facing of the initial excavation lifts that becomes unsupported when the next excavation lift is performed. For typical construction facings consisting of 100-mm (4-in.) thick shotcrete, experience has shown that the soil nails will support the weight of the facing without major difficulties. For thicker applied shotcrete facings, support for the shotcrete facing weight by considering the shear capacity of the nails and the bearing capacity of the soils beneath the nails should be formally evaluated. The maximum thickness of shotcrete facing that can be supported in this manner is dependent on the strength of the soils. In competent ground, shotcrete facings up to 200- to 250-mm (8- to 10-in.) thick have been successfully supported.

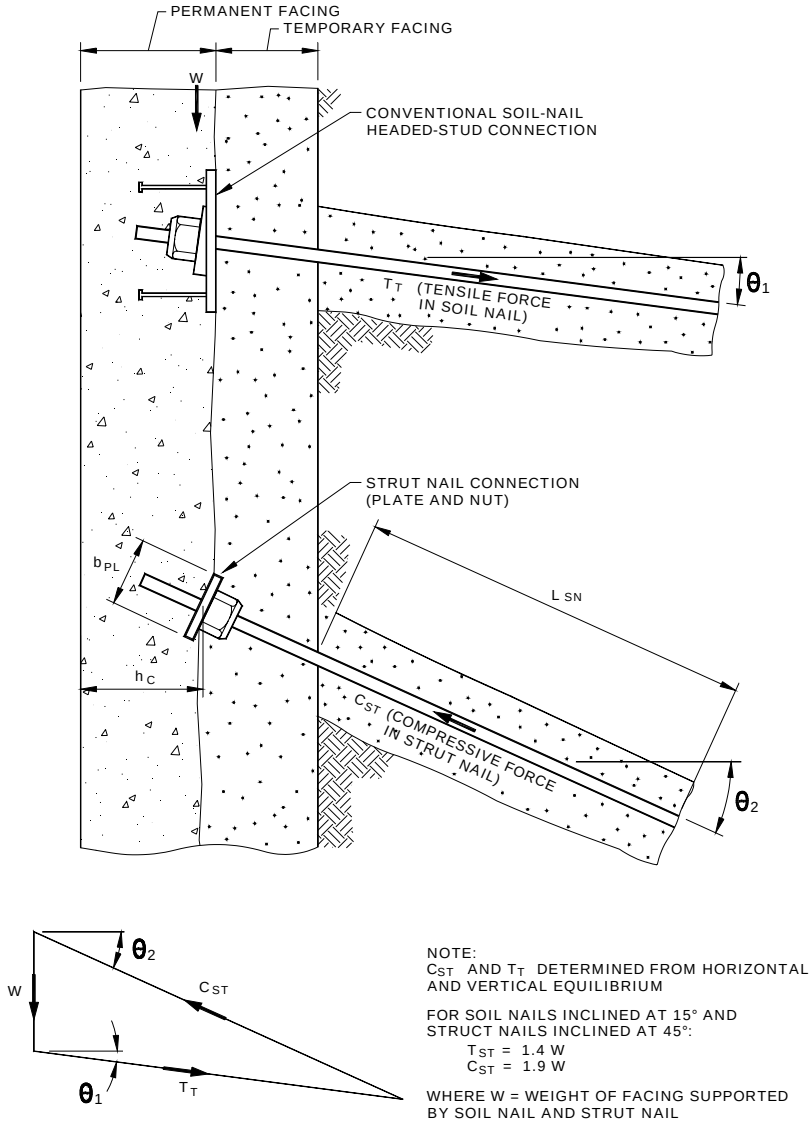
If necessary, support of the shotcrete facing weight may be achieved by the installation of additional short, steeply inclined reinforcing elements acting as compression struts. Figure 5.28 shows a soil nail/strut nail system supporting a facing panel of self-weight W. The soil nail wall has a conventional headed-stud connection to the facing. The strut nail has a bearing plate and washer connection system, as shown. Figure 5.28 also shows the method for calculating the service loads (associated with self weight support of the facing) within both the soil nail and the strut nail. Note that shear resistance along the facing-soil interface is disregarded.

5.12 SPECIAL DESIGN CONSIDERATIONS

5.12.1 Stepped Structures

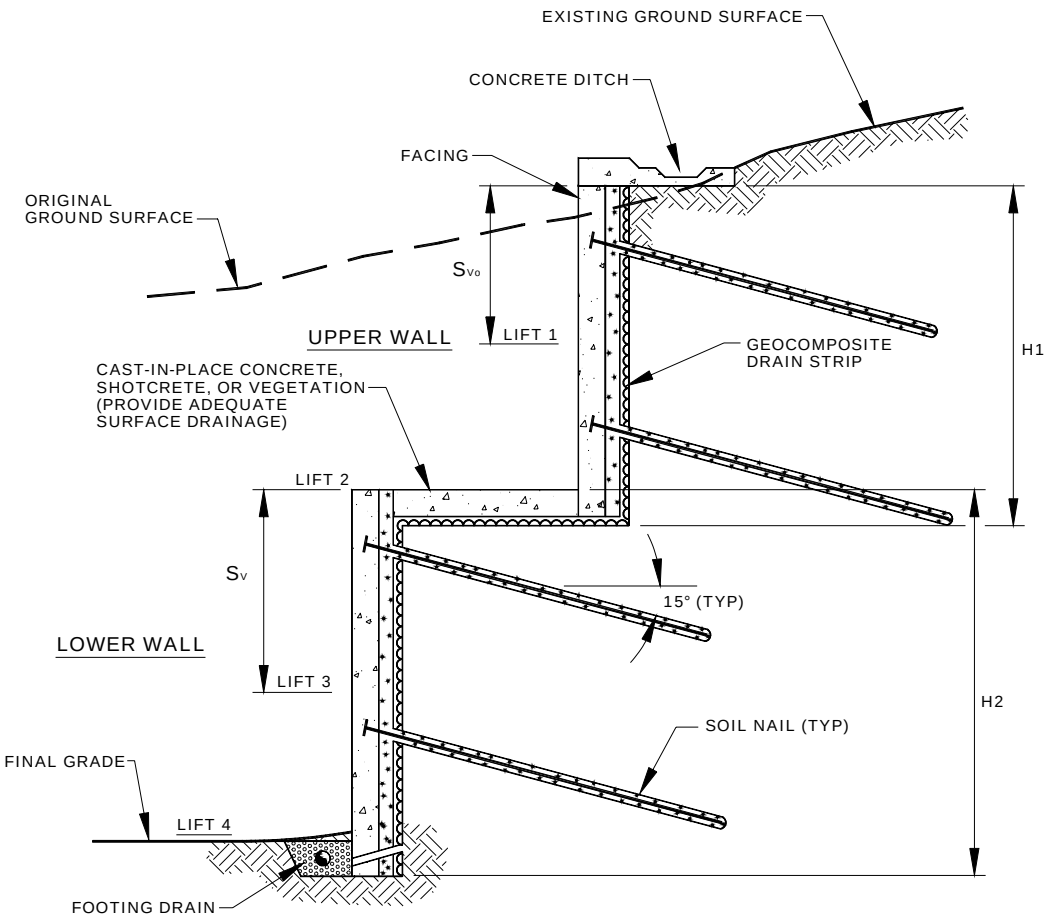
In some cases, it may be appropriate to use stepped or benched facing for a soil nail wall, with horizontal setbacks between individual wall sections (Figure 5.29). The structure acts as an equivalent battered face wall when the horizontal setback is small in relation to the height of the individual benches. Using guidance from MSE walls, this situation arises when the setback is less than about 1/20 of the combined height of the stepped wall, which results in an equivalent batter of

3 degrees. When the horizontal setback is larger than the height of the lower wall (H_2), each individual wall will act independently and each wall must be analyzed and designed as two independent soil nail wall structures. If the horizontal setback is smaller than the height of the lower wall, the lower wall must be analyzed considering the upper wall as a surcharge.



From Byrne et al. (1998).

Figure 5.28: Strut Nail Concept.



Source: Byrne et al. (1988).

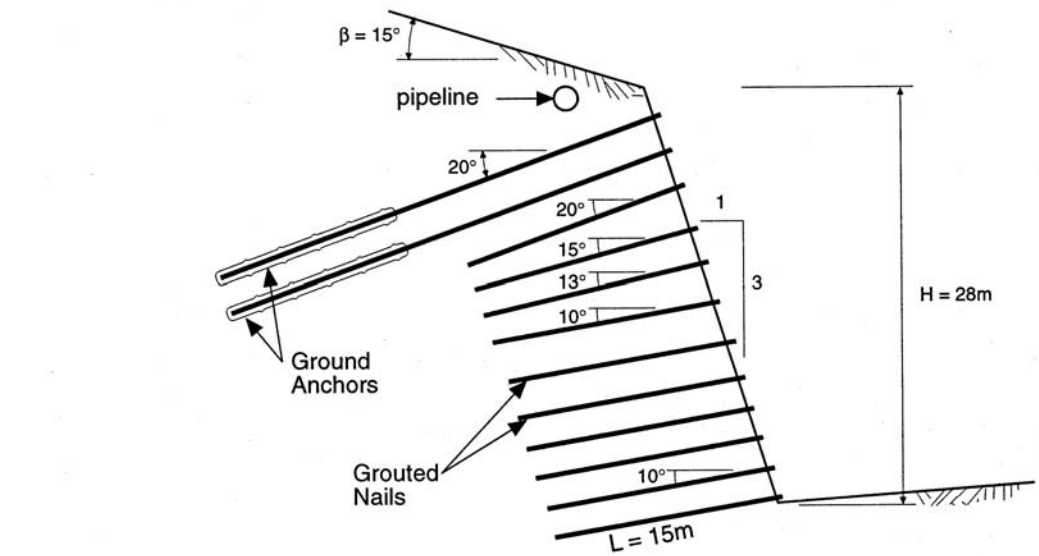
Figure 5.29: Example of Stepped Soil Nail Wall.

5.12.2 Composite Structures

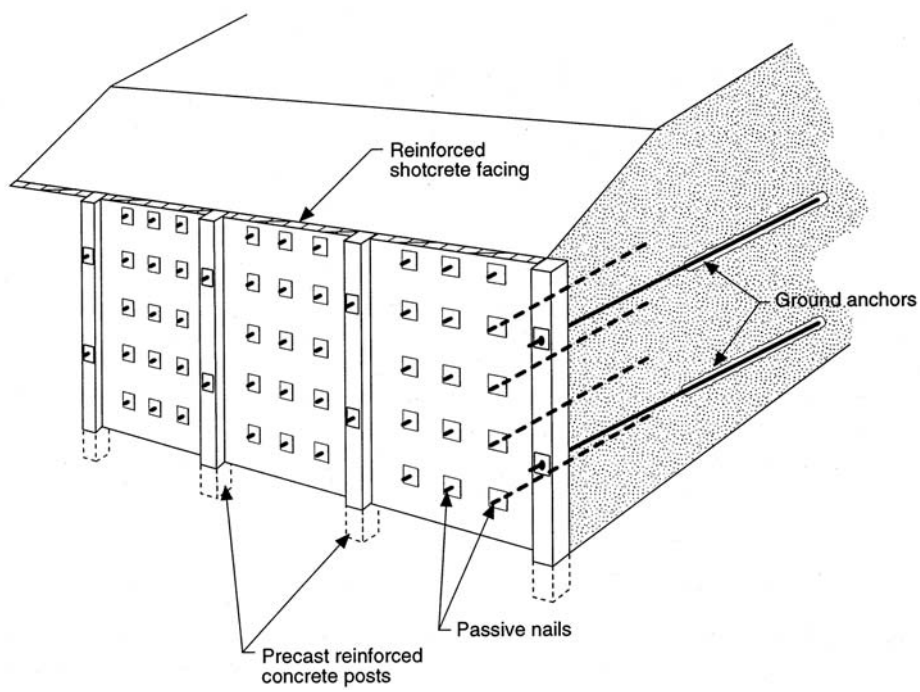
In this manual, composite structures refer to the combination of soil nail walls and another ground stabilization technology. For instance, a composite system may consist of soil nails in conjunction with ground anchors (Figure 5.30). Both nails and ground anchors are installed as excavation proceeds from top-down. The main objective of using ground anchors is to contribute significantly to global stability. In addition, wall deformation can be greatly reduced, particularly if the ground anchors are installed in the top portion of the wall (Figure 5.30a). Ground anchors can be also installed along the full height of the wall by means of precast concrete posts (Figure 5.30b). High walls, up to 25 m (82 ft), have been constructed with composite systems.

The design methodology depends on the configuration of the support system, particularly on the relative contribution and intended function of the nails and ground anchors. For instance, for the system shown in Figure 5.30a (where soil nails provide comparatively more stability for shallower failure surfaces than ground anchors do; while the latter provide more stability against deep-seated failures), the recommended design approach is to analyze the wall with soil nails only, but limiting

the slip surfaces considered in the analysis to shallow surface (i.e., at a distance behind the top of the wall no greater than 1.5 times the proposed nail length). The soil nailed zone is then considered a rigid block. The ground anchors must provide stability against deeper failure surfaces. Therefore, the length of the ground anchors will be controlled by the stability requirements of the soil nailed block. The reader is referred to GEC No. 4 (Sabatini et al., 1999) for complete details of ground anchor design.



(a)



(b)

Source: Byrne et al. (1988).

Figure 5.31: Composite Wall Structures.

CHAPTER 6 DESIGN OF SOIL NAIL WALLS

6.1 INTRODUCTION

The purpose of this chapter is to present a step-by-step generalized method for soil nail wall design. The intent is to provide the reader with a "prescriptive" approach for the design of soil nail walls. The five major steps and their substeps in this design method are outlined in Table 6.1. In the remainder of this chapter, each of these steps is presented.

Table 6.1: Design Steps For Soil Nail Walls.

Step 1.	**INITIAL SOIL NAIL WALL DESIGN CONSIDERATIONS** A. Wall layout (e.g., wall height, wall length, face batter) B. Soil nail vertical and horizontal spacing C. Soil nail pattern on wall face (e.g., square, staggered, other irregular patterns) D. Soil nail inclination E. Soil nail length and distribution F. Soil nail material type (e.g., selection of steel bar grade) G. Selection of relevant ground properties for design (e.g., soil shear strength; location of groundwater table, etc.) H. Other initial considerations
Step 2.	**PRELIMINARY DESIGN USING SIMPLIFIED CHARTS** These charts are used to preliminarily evaluate nail length and maximum nail force.
Step 3.	**FINAL DESIGN** A. External Failure Modes 1) Global stability 2) Sliding stability 3) Bearing capacity B. Seismic Considerations C. Internal Failure Modes 1) Nail pullout resistance 2) Nail tensile resistance D. Facing Design 1) Nail head load 2) Wall facing type and thickness 3) Facing materials 4) Flexural resistance 5) Facing punching shear resistance 6) Facing head stud resistance 7) Other design facing considerations
Step 4.	**ESTIMATE MAXIMUM WALL DEFORMATIONS**
Step 5.	**OTHER DESIGN CONSIDERATIONS** A. Drainage B. Frost protection C. External loads D. Support for facing dead load (e.g., foundation support for precast wall facing)

Before performing any of the steps outlined in Table 6.1, the soil nail wall design engineer should:

- Establish project requirements, including construction constraints (e.g., topography, site access, ROW, utilities, adjacent structures).
- Evaluate aesthetic requirements; and performance criteria (e.g., deformation, stability, durability).

After completing the design, the design engineer will prepare soil nail wall specifications (see Chapter 7) and recommendations for construction monitoring (see Chapter 8).

6.2 STEP 1: INITIAL SOIL NAIL WALL DESIGN CONSIDERATIONS

A. Wall Layout

Establish the layout of the soil nail wall, including: (1) wall height; (2) length of the wall; and (3) wall face batter (inclination typically ranges from 0º to 10º). The evaluation of the wall layout also includes developing the wall longitudinal profile, locating wall appurtenances (e.g., traffic barriers, utilities, and drainage systems), and establishing ROW limitations.

Battered wall face can be selected to improve temporary face stability, as a battered face exerts smaller forces on the wall, thus requiring shorter soil nails. The material savings resulting from the use of shorter nails may offset the increased cost of soil excavation incurred to create the batter. A mild batter (i.e., less than 10 degrees) is usually provided for aesthetic reasons, especially around horizontal curves, and may be enough to ensure temporary face stability. A batter greater than 10 degrees can enhance stability.

B. Soil Nail Vertical and Horizontal Spacing

Horizontal nail spacing, S_H, is typically the same as vertical nail spacing, S_V (Figure 6.1). Nail spacing ranges from 1.25 to 2 m (4 to 6.5 ft) for conventional drilled and grouted soil nails, and may be as low as 0.5 m (1.5 ft) for driven nails. This reduced spacing for driven nails is required because driven soil nails develop bond strengths that are lower than those for drilled and grouted nails. A soil-nail spacing of 1.5 m (5 ft) is routinely used and is preferred for conventional drilled and grouted soil nails. Soil nail spacing may be affected by the presence of existing underground structures.

Soil nail spacing in horizontal and vertical direction must be such that each nail has an influence area $S_H \times S_v \leq 4\ m^2$ ($\leq 40\ ft^2$ ft). The design engineer should specify a minimum horizontal soil nail spacing of about 1.0 m (3.3 ft). Design forces from global stability analysis and facing design are affected by soil nail spacing. In general, the larger the spacing, the greater the design forces. The purpose of the minimum nail spacing is to reasonably ensure that group effects between adjacent soil nails are minimized due to potential nail intersection as a result of drilling deviations. Group effects reduce the load-carrying capacity of individual soil nails. The maximum soil nail spacing should also be specified. The purpose of a maximum spacing [usually about 2 m (6.5 ft)] is to provide for a soil nail system that is relatively easy to construct and that effectively supports the lateral earth pressures and imposed surcharge loads.

C. Soil Nail Pattern on Face

The soil nail pattern is commonly one of the following (see Figure 6.1): (1) square (rectangular); (2) staggered in a triangular pattern; and (3) irregular (at limited locations).

A square pattern results in a column of aligned soil nails, and facilitates easier construction of vertical joints in the shotcrete facing (or easier installation of precast concrete panels). Also, a square pattern enables a continuous vertical installation of geocomposite drain strips behind the facing to be easily constructed. In practice, a square pattern is commonly adopted.

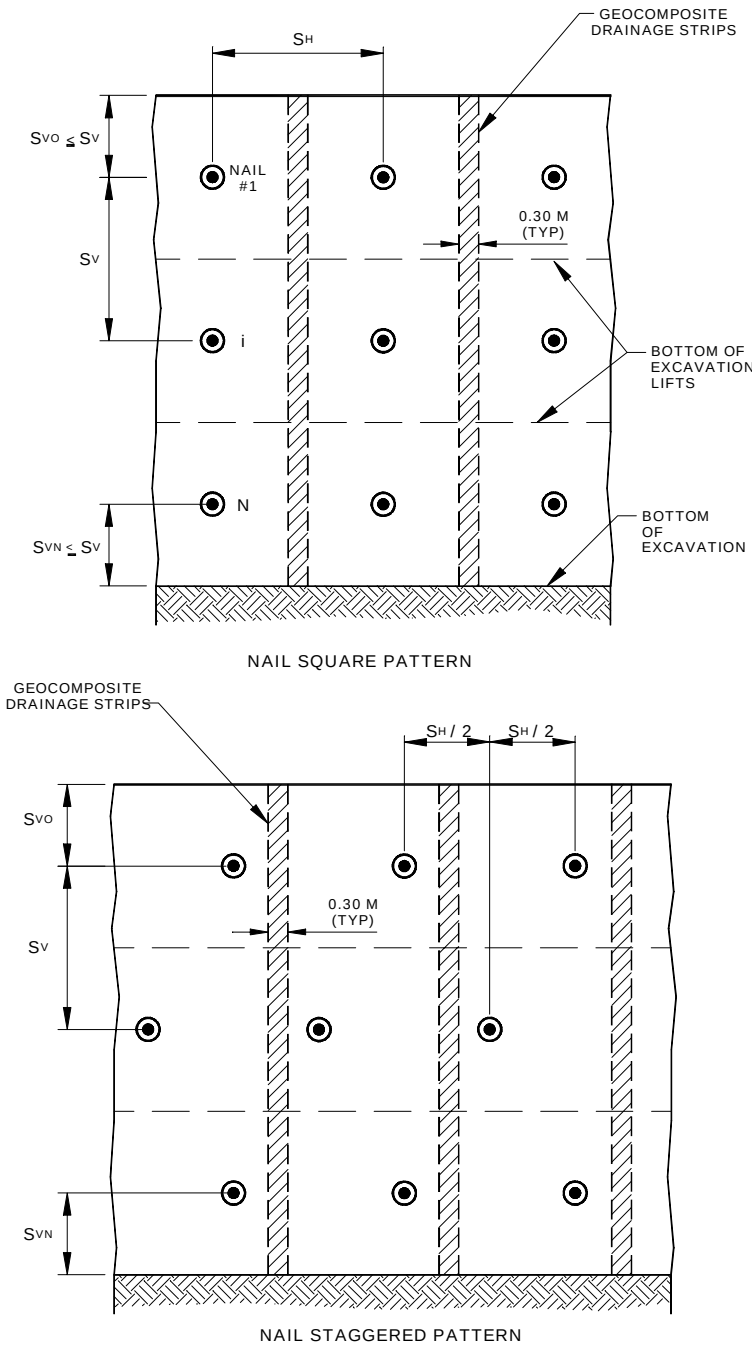


Figure 6.1: Soil Nail Patterns on Wall Face.

A staggered soil nail pattern results in a more uniform distribution of earth pressures in the soil mass. This effect is beneficial because an enhanced soil arching effect is achieved. This method should be considered in cases where marginally stable soils are present because such soils have less margin to redistribute loads. The main disadvantage of the use of a triangular pattern is that it makes installation of geocomposite drain strips more complicated. In particular, it can be difficult to establish a vertically continuous drain system to the footing drain, especially for higher walls.

The use of uniform nail spacing is beneficial because it simplifies construction and quality control. However, due to project-specific geometric constraints, nail spacing may need to be irregular, with reduced spacing at some locations; for instance, in areas where the bottom of the excavation or the top of the wall is not horizontal. In such cases, it is more convenient to install one or two nail rows parallel to the non-horizontal edge and then establish a transition zone where nails have a closer vertical spacing until a horizontal nail row is achieved (Figure 6.2a). It is also customary to reduce horizontal spacing at the vertical edges of the wall to accommodate transition zones (Figure 6.2a).

D. Soil Nail Inclination

Soil nails are typically installed at an inclination ranging from 10 to 20 degrees from horizontal with a typical inclination of 15 degrees. This recommended range of soil nail inclination assures that grout will flow readily from the bottom of the hole toward the nail head for typical borehole and soil nail dimensions and conventional grout mixtures. Steeper nail inclinations may be required, particularly for the upper row of nails, if a significantly stronger soil zone is located at a greater depth and a more effective anchorage in the stiffer layer is desired. Such evaluations can be readily made during design. Nail inclination smaller than about 10 degrees should not be used because the potential for creating voids in the grout increases significantly. Voids in the grout will affect the load capacity of the nail and reduce the overall corrosion protection provided by the grout.

Project conditions may, however, require that other nail inclinations be used. For example, Figure 6.2b shows a case in which utilities or other underground structures are located within the proposed soil nail zone. In most cases, this situation only occurs for the upper first and second rows of nails. Another situation where different nail inclinations may be used is at exterior wall corners. To avoid intersecting nails behind exterior corners of a wall, nail inclination on one side of the corner could be installed with a different inclination. An alternative layout for exterior corners is to splay the nails on a plan view (Figure 6.2c). Overhead space restrictions may require that the nail inclination be smaller than 15 degrees. This might be the case for road widening at embankment bridge abutments. Logistical limitations due to location of nailing equipment (i.e., operating at the bottom of a narrow excavation) may require a steeper nail inclination.

The effect of nail inclination should be considered in global and local stability analyses of the soil nail wall system because stability factors of safety for the system, particularly for sliding wedge analyses in the upper portion of the wall, can decrease significantly as the nail inclination increases below the horizontal.

E. Soil Nail Length and Distribution

The distribution of soil nail lengths in a soil nail wall can be selected as either uniform (i.e., only one nail length is used for the entire wall), or variable, where different nail lengths may be used for individual soil nail levels within a wall cross section. Additional information on nail distribution is provided below.

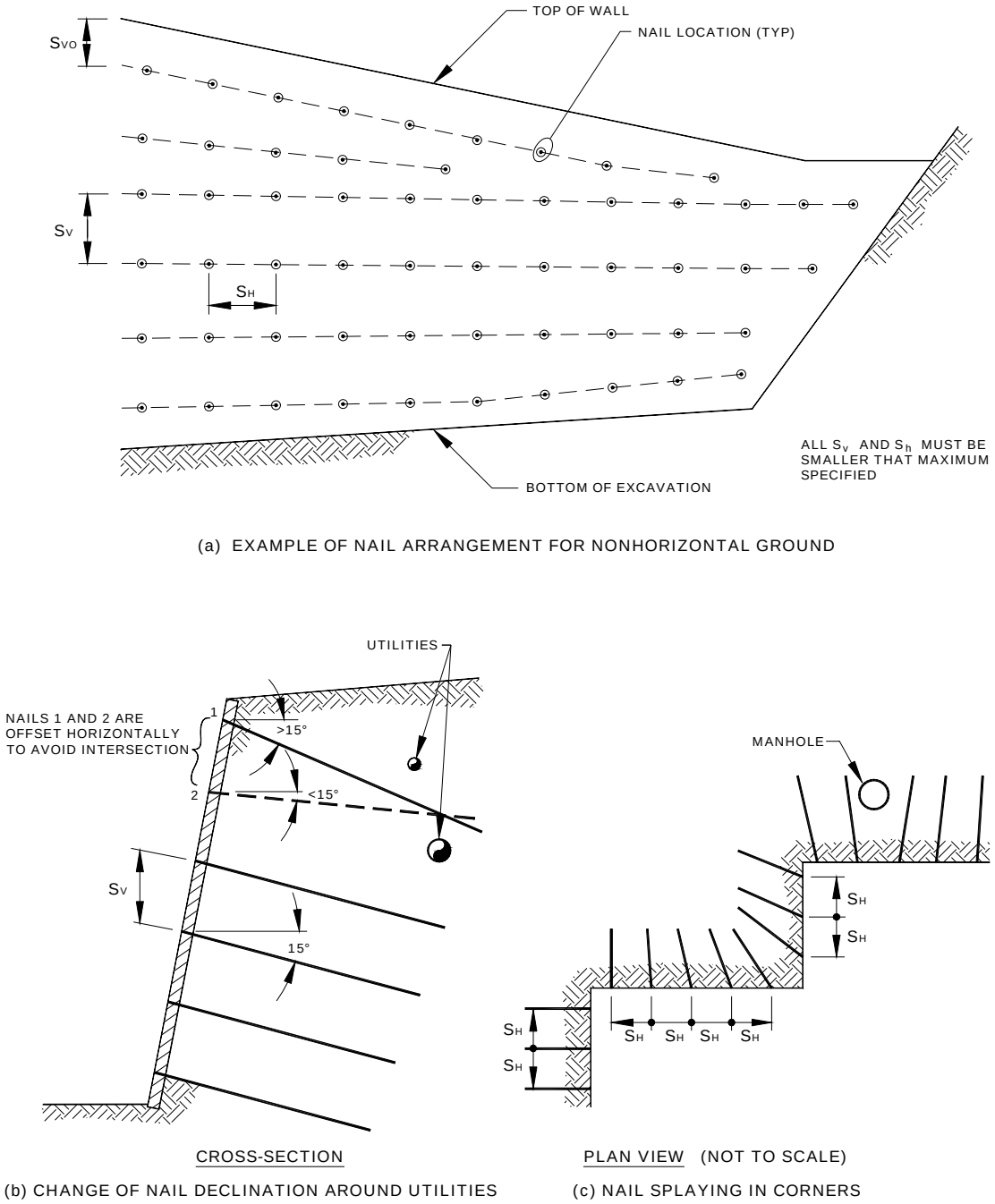


Figure 6.2: Varying Nail Patterns.

- *Uniform Nail Length*: When the potential for excessive wall deformation is not a concern (e.g., soil nail walls constructed in competent ground or in an area without nearby structures), it is beneficial to select a uniform length distribution because it simplifies construction and quality control. Additionally, a slightly smaller total length of nails is obtained with a uniform soil length pattern. This pattern provides commonly a high sliding stability safety factor. Uniform patterns should be used in most projects.

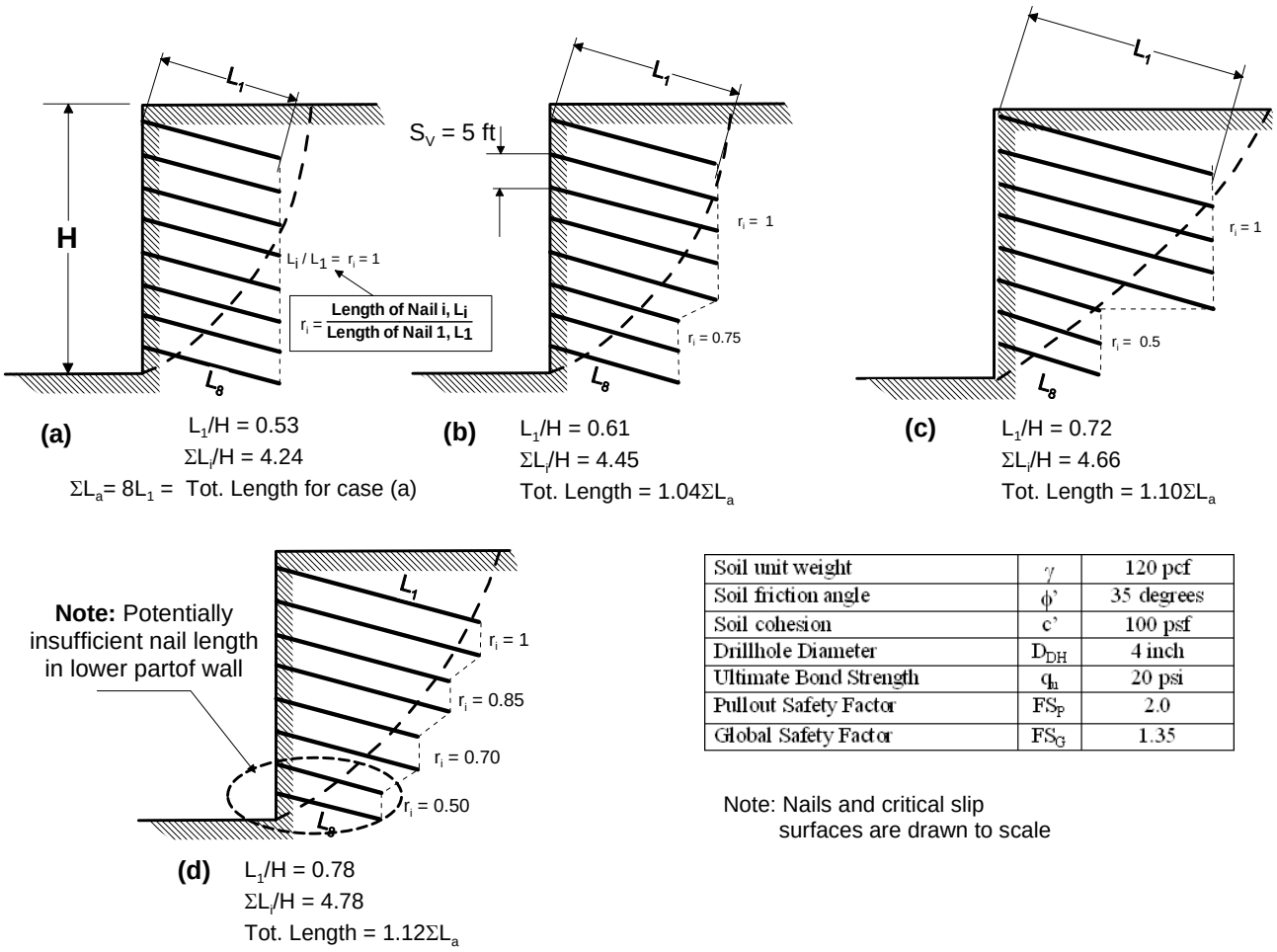
- *Variable Nail Lengths*: Occasionally, a variable nail length distribution may be used if wall deformations need to be controlled. The global equilibrium and deformation pattern of a system with different nail lengths will be different from a system with uniform nail lengths. Field measurement data from constructed soil nail walls indicate that wall displacements can be significantly reduced if the nail lengths in the upper two-thirds to three-quarters of the wall height are greater than those in the lower portion. Placing additional reinforcement (i.e., length of soil nail) near the top of the wall will provide more resistance to wall movement at the critical areas near the top of the wall. As lower nails are shorter in non-uniform length patterns, this distribution tends to produce a lower sliding stability safety factor.

Figure 6.3 shows different nail length distributions for the same height wall and the required total nail length to obtain a factor of safety of 1.35. The maximum calculated total length (corresponding to Figure 6.3d) is 12 percent greater than that required for the uniform length pattern (base case, Figure 6.3a). For this particular set of examples, the comparison indicates that factors of safety are not very sensitive to nail distribution with depth. However, certain nail length distributions may result in less wall deformation than other nail layouts despite having similar factors of safety. In addition, some nail length distributions may have too short nails in the lower portion of the wall; this unfavorable condition may lead to a sliding stability failure.

Performance of soil nail walls has shown that larger displacements are observed when the upper nails are too short. The deformations in soil nail walls can be significantly reduced when nails at the top of the structure are longer than required by stability analysis. In general, the higher the global factor of safety of a soil nail wall, the smaller the wall deformations. Therefore, all other variables being equal, the nail layouts shown in Figures 6.3c and 6.3d are likely to result in smaller wall deformations, especially near the top of the wall.

Nail lengths have been installed successfully with a uniform nail length in the upper two-thirds to three-quarters of the wall, with progressively shorter nails to a minimum value, not smaller than 0.5 H (H is the wall height), at the bottom of the wall in dense cohesionless soils that provide relatively large sliding stability. In general practice, nail length in the lower rows should never be shorter than 0.5 H. Nail lengths less than 0.5 H will not likely satisfy sliding stability requirements. As an example, the nail distribution shown in Figure 6.3d may not meet sliding stability requirements. In all cases, and especially where reducing the nail lengths in the lower reaches of the wall are considered, stability analysis considering sliding need to be performed as part of a detailed design.

In general, variable nail lengths result in a more complicated installation and require more nail materials. Nevertheless, as many soil nail projects are specified based on performance criteria, contractors may prefer to use longer nails in the upper rows to reduce deflections. Project specifications must provide ROW constraints, locations of underground utilities and substructures (or requirements that the contractor locate these), and specific deformation criteria (i.e., maximum wall deflection and location where this deflection is to be measured).



Soil unit weight	γ	120 pcf
Soil friction angle	ϕ'	35 degrees
Soil cohesion	c'	100 psf
Drillhole Diameter	D_{DH}	4 inch
Ultimate Bond Strength	q_u	20 psi
Pullout Safety Factor	FS_P	2.0
Global Safety Factor	FS_G	1.35

Note: Nails and critical slip surfaces are drawn to scale

Figure 6.3: Effect of Different Nail-Length Patterns.

Based on the discussion presented in this section, the following recommendations are made concerning soil nail length and distribution.

- Select uniform length pattern whenever possible.
- Select longer nails than required by the target factor of safety as a means to reduce wall deformations in the upper portions of the wall.
- Avoid the use of "short" nails in top portion of wall.
- Avoid the use of too "short" nails in lower portion of wall. Evaluate if shorter nails in bottom rows installed in competent ground satisfy sliding stability requirements. Shorter nails at the bottom should be not smaller than 0.5 H.

- Non-uniform nail length patterns may be used if soil layers with very dissimilar conditions are encountered.

For feasibility evaluations, soil nail length can be initially assumed to be 0.7 H, where H is the height of the wall. The length of the nails may be greater than 0.7 H if large surcharge loads are expected or if the wall is very high [greater than 10 m (approximately 30 ft) high]. In Step 2 of the design method presented herein (see Table 6.1), simplified design charts are used to select the length of the nails.

F. Soil Nail Materials

Select appropriate grade of steel for the soil nail bar. Information on the selection of steel grade is presented in Chapter 4; however, for most applications Grade 420 MPa (Grade 60) steel is used.

G. Soil Properties

The procedures and methods used to select soil properties for the analysis and design of soil nail walls is provided in Chapter 3. The ultimate bond strength for the grout-ground interface can be selected using Table 3.10.

H. Other Initial Considerations

- Evaluate corrosion potential (see Section 3.9 and Appendix C).
- Evaluate drilling methods likely to be used by prospective contractors for the project. This information is used to select a design ultimate bond strength value.
- Estimate drillhole diameter based on previous experience in similar ground and diameter restrictions imposed by selected level of corrosion protection.
- Select factors of safety (see Section 5.9) for the different failure modes (e.g., global stability, sliding, tensile strength, pullout).
- Define loads

6.3 STEP 2: PRELIMINARY DESIGN USING SIMPLIFIED CHARTS

6.3.1 Introduction

Nail length, diameter, and spacing typically control external and internal stability of a soil nail wall. Therefore, these parameters may be adjusted during design until all external and internal stability requirements are satisfied [i.e., calculated factors of safety for each failure mode are larger than the minimum values selected for the design (e.g., Table 5.3)]. A series of charts was developed in this document as a design aid to provide preliminary nail length and maximum tensile forces (see Appendix B). The charts were developed using the computer program SNAIL, which was selected because it is public domain software, readily available, and free of charge. In preparing these charts, the following main assumptions were made:

- homogenous soil;
- no surcharge;

- no seismic forces;
- uniform length, spacing and inclination of nails; and
- no groundwater.

When the conditions of a new analysis case, do not match the assumptions listed above, it is recommended that interpolations or extrapolations be made to estimate the soil lengths from these charts. Alternatively, the use of a preliminary nail length between 0.7 to 1.0 times the wall height can be made. The upper range of soil nail length is used for less favorable soil conditions, wall heights greater than 10 m (30 ft), and where large surcharge loads need to be resisted by the wall.

The charts were developed for different values of face batter (α), backslope (β), effective friction angle (ϕ'), and ultimate bond strength (q_u). Table 6.2 presents the set of values used for the development of the design charts.

Table 6.2: Variable Parameters.

Parameter	Units	Values
Face Batter	Degrees	0, 10
Backslope	Degrees	0, 10, 20, 30
Effective Friction Angle	Degrees	27, 31, 35, 39
Ultimate Bond Strength	KPa	52, 104, 172, 276, 689

These values encompass a wide range of soil nail wall geometries and ground conditions. If intermediate parameter values are required for a particular project application, then it is acceptable to evaluate preliminary values of nail length and maximum nail tensile force by interpolating between values on the charts.

The first type of charts was developed to evaluate the nail length (Figures B.1a through B.6a in Appendix B) for combinations of α and β. Using these charts, the required nail length, L, (normalized with respect to the wall height, H) to achieve a global safety factor FSG = 1.35 is obtained as a function of the normalized allowable pullout resistance (μ). The normalized allowable pullout resistance is defined as:

$$\mu = \frac{q_u \, D_{DH}}{FS_P \gamma \, S_H \, S_V} \qquad \text{(Equation 6.1)}$$

where FS_P is the factor of safety against pullout (typically 2.0); D_{DH} is the drillhole diameter; γ is the total unit weight of the soil behind the wall; and S_H and S_V are the horizontal and vertical nail spacing, respectively. The nail lengths in these charts were computed based on the most critical failure surface (i.e., considering base and toe failures) for the selected geometry and material properties, and assuming that failure of the nail (i.e., tensile breakage) and/or failure of the facing would not take place. Therefore, the pullout failure is implicitly assumed. Equation 6.1 is based on a drillhole diameter of 100 mm (4 in.). Also, the use of Equation 6.1 inherently assumes that the soil has a cohesion intercept c' such that $c^* = c/\gamma H = 0.02$. If the drillhole diameter or cohesion

intercept values being considered are different than the assumptions stated here, then adjustments to the calculated nail length and maximum tensile forces are made in the final step. These adjustments are discussed subsequently.

The second type of charts (Figures B.1b through B.6b in Appendix B) provides the corresponding maximum normalized design tensile force of all nails (t_{max-s}) as a function of μ calculated for a global safety factor of 1.0. The maximum normalized design tensile force in the bar is defined as:

$$t_{max-s} = \frac{T_{max-s}}{\gamma\ H\ S_H\ S_V} \qquad \text{(Equation 6.2)}$$

With t_{max-s} read from the design charts, the maximum nail tensile force, T_{max-s} can be calculated using Equation 6.2. These design charts are developed for the case in which all nail bars are the same length. These design charts do not provide information on the distribution of tensile load in individual soil nails or the maximum load in any particular nail. A preliminary design with the charts is not necessary if a full, final design using computer programs will be used. However, even a final design is performed, the charts can still provide preliminary values.

6.3.2 Preliminary Design Procedure

A step-by-step procedure for preliminary design using the charts provided in Appendix B is presented in this section. An example of the use of the charts is provided in the example presented in Section 6.7.

1. For a specific project application, evaluate batter (α), backslope (β), effective friction angle (ϕ'), and ultimate bond strength (q_u). Calculate normalized pullout resistance (μ) using Equation 6.1.
2. Obtain normalized length (L/H) from the first set of charts (Figures B.1a through B.6a in Appendix B).
3. Obtain normalized force (t_{max-s}) from the second set of charts (Figures B.1b through B.6b in Appendix B).
4. Using Figure B.7, evaluate correction factors for: (a) normalized length to account for a drillhole diameter other than 100 mm (4 in.) (correction factor C_{1L}), (b) a c* value other 0.02 (correction factor C_{2L}), and (c) a global factor of safety other than 1.35 (correction factor C_{3L}).
5. Using Figure B.7, evaluate correction factors for normalized maximum nail force to account for: (a) a drillhole diameter other than 100 mm (4 in.) (correction factor C_{1F}), and (b) a c* value other 0.02 (correction factor C_{2F}).
6. Apply correction factors to normalized length and/or normalized force. Calculation method is provided on Figure B.7.
7. Multiply the normalized length by the wall height to obtain the soil nail length.
8. Calculate the maximum design load in the nail T_{max-s} using the value of t_{max-s} and Equation 6.2.

9. Calculate the required cross-sectional area (A_t) of the nail bar according to:

$$A_t = \frac{T_{max-s} FS_T}{f_y} \qquad \text{(Equation 6.3)}$$

where f_y is the steel yield strength and FS_T is the factor of safety for nail bar tensile strength (see Table 5.3).

10. Select closest commercially available bar size using Table A.1 that has a cross-sectional area of at least that evaluated in the previous step.

11. Verify that selected bar size fits in the drillhole with a minimum grout cover thickness of 25 mm (1 in.).

12. If the length and/or nail diameter are not feasible, select another nail spacing and/or drillhole diameter, recalculate the normalized pullout resistance, and start the process again.

6.4 STEP 3: FINAL DESIGN

It is necessary to perform a final design in which the actual wall geometry, stratigraphy, loads, variation of engineering parameters (if present), and other conditions are considered. The preliminary design procedure described in section 6.3 should not replace the findings and results obtained with the final design presented herein. The steps of the final design are described as follows.

A. External Failure Modes

The following external failure modes are considered for final design of a soil nail wall:

1) Global Stability (Figure 5.3a)

- Select a well-established computer program for design of soil nail walls that considers heterogeneous soils, groundwater, general loading conditions, seismic forces, and diverse nail characteristics. In this section, the computer program SNAIL (see Section 5.8.2 and Appendix F for a description of input and output capabilities of SNAIL) is selected.

- Select the factor of safety against pullout failure (FS_P) from Table 5.3 (SNAIL requires the value of bond strength reduction factor (BSRF) equal to $1/FS_P$ (e.g., for $FS_P = 2$, the corresponding BSRF is 0.5).

- Use the nail diameter calculated in preliminary design (if available) as the input nail hole diameter for SNAIL. If no preliminary design was performed, assume a relatively large nail diameter [e.g., > 100 mm (4 in.)] to ensure that pullout failure controls the design. Also, select a high facing punching shear capacity [e.g., R_{FP} > 2,000 kN (at least 220 kip)] to ensure that pullout failure controls the design. It is noted that the tensile strength and punching failure mechanisms are explicitly considered subsequently in this chapter.

- For the first SNAIL analysis, use nail length estimated in Step 1.E (or calculated in the preliminary design) and perform global stability analysis using SNAIL.

- After selecting an initial nail length, perform the following iterative procedure using SNAIL: (1) calculate the global factor of safety using the selected nail length; (2) compare the calculated global factor of safety to the recommended minimum factor of safety; and (3) increase or decrease the nail length if the calculated factor of safety is lower or higher than the recommended value (Table 5.3) and start the process again.

- If the length of the nail needs to be reduced without reducing the factor of safety, then increase the nail hole diameter or reduce the nail spacing.

2) Sliding Stability (Figure 5.3b)

Evaluate the potential for sliding failure using the equations and procedures outlined in Section 5.4.3 and summarized below.

- Calculate the horizontal resisting forces (ΣR) using Equation 5.8 (see Figure 5.5).

- Calculate the horizontal driving forces as follows:
  - If necessary, calculate the equivalent backslope angle (β_{eq} in Figure 5.5);
  - Evaluate the active earth pressure coefficient K_A using Equations 5.11 to 5.13 in Section 5.4.3 and/or the charts presented in Figures 5.6 and 5.7;
  - Calculate the active lateral earth force (P_A) using Equation 5.10;
  - Assume that the active lateral earth force is applied a distance of $H_1/3$ from the elevation of the bottom of the soil nail wall (Figure 5.5); and
  - Calculate the horizontal driving force (ΣD) using Equation 5.9.

- Evaluate the factor of safety against sliding failure (FS_{SL}) using Equation 5.7.

- If the factor of safety against sliding is lower than the specified minimum (see Table 5.3), increase the length of the lower nails and reevaluate sliding stability using the method presented herein.

3) Bearing Capacity (Figure 5.3c)

- If soil nail wall is constructed in soft soils, evaluate the potential for bearing capacity failure using the equations outlined in Section 5.4.4.

- Evaluate the factor of safety against heave (bearing capacity) failure (FS_H) using Equation 5.14.

- If the factor of safety against bearing capacity is lower than the specified minimum see Table 5.3), then the site may not be suitable for construction of soil nail wall structures.

B. Seismic Considerations

Consider seismic loads to ensure that nail lengths calculated in the previous step for static loading condition provide adequate factors of safety for seismic loading conditions.

1) Define seismic loads

   a. Determine the seismic zone of the project site.

   b. Use national seismic maps (e.g., 1988 national seismic maps with contours of maximum horizontal acceleration contained in Division I-A "Seismic Design" in AASHTO (1996)). Alternative sources such as NEHRP (1997) and IBC (2000) can be used.

   c. Read maximum ground acceleration coefficient, A_I, from maps.

   d. Establish soil profile type at the site (this step requires results from the site investigation).

   e. Determine site coefficient S from Table 6.3.

   f. Account for potential soil amplification by modifying the reference ground acceleration coefficient (A_I) corresponding to a Soil Profile I by a site coefficient (S), as follows:

$$A = S\,A_I \qquad \text{(Equation 6.4)}$$

   g. Obtain site coefficient S from Table 6.3:

Table 6.3: Seismic Site Coefficients.

Soil Profile	Description	S
I	1. Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 760 m/s (2,500 ft/s), or by other appropriate means of classification); or 2. Stiff soil conditions where the soil depth is less than 60 m (200 ft) and the soil types overlying bedrock are stable deposits of sands, gravels, or stiff clays.	1.0
II	Stiff clay or deep cohesionless conditions where the soil depth exceeds 60 m (200 ft) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.	1.2
III	Soft to medium-stiff clays and sands, characterized by 9 m (30 ft) or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.	1.5
IV	Soft clays or silts greater than 12 m (40 ft) in depth. (Such materials may be characterized by a shear wave velocity less than 150 m/s (500 ft/s) and might include loose natural deposits or synthetic, non-engineered fill.)	2.0

(Based on AASHTO, 1996).

h. Calculate design acceleration coefficient at center of gravity:

$$A_m = (1.45 - A)\,A \qquad \text{(Equation 6.5)}$$

2) Calculate the horizontal seismic coefficient, k_h

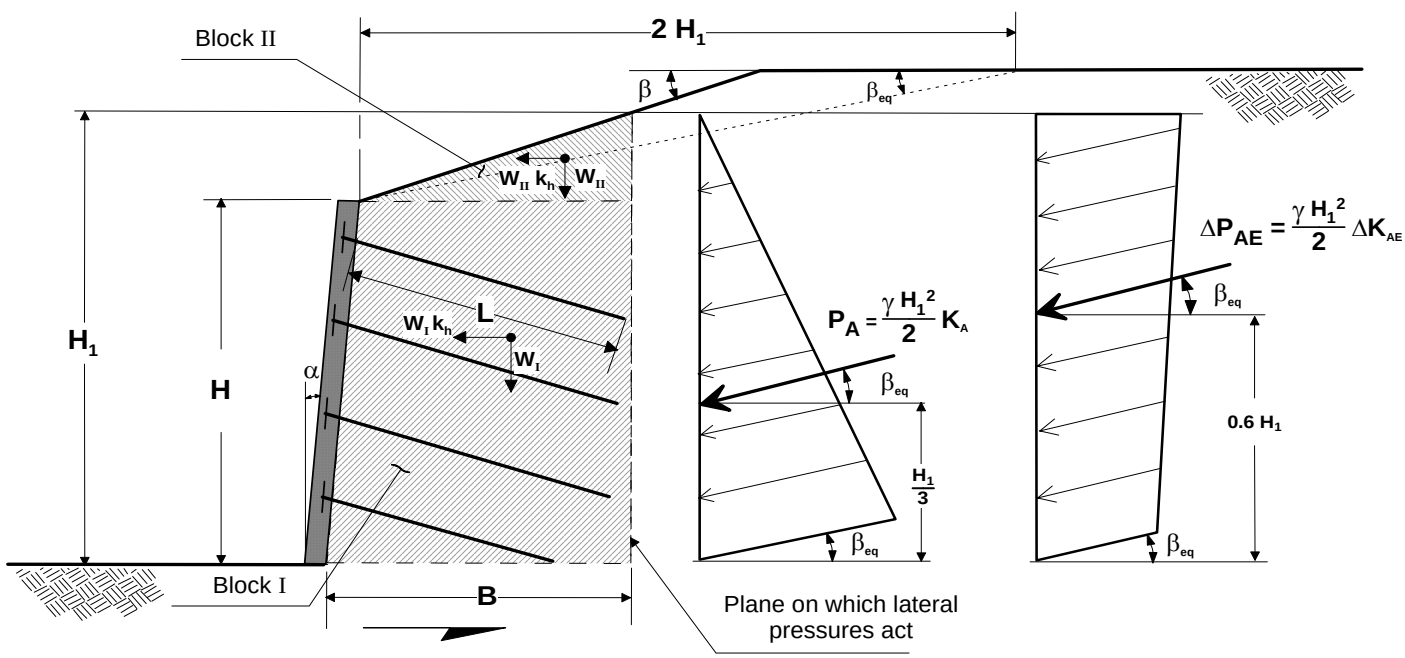
Select a value for the horizontal coefficient (k_h) between 0.5 A_m and 0.67 A_m. The lower values correspond to stiffer soils. See Section 5.4.5.2 for a more comprehensive discussion of seismic coefficient.

3) Perform global stability analysis

- Perform a global stability analysis using all the parameters selected for the static case along with the calculated seismic coefficient. This analysis may be performed using SNAIL.
- Compute global factor of safety for seismic conditions.
- Verify that calculated FS_G is greater than minimum FS_G (see Table 5.3) for seismic conditions.
- If minimum criterion is not met, increase the nail length and perform analysis again.

4) Evaluate sliding stability with seismic forces

Calculate the seismically induced inertia forces within a soil nailed block and calculate the increased active earth thrust acting behind the soil block as follows (see Figure 6.4):



a. Calculate inertia forces

- Limit the mass of soil subjected to inertia forces to a block extending from the wall facing to the end of the nails and having a height H_1 and a width B (Figure 6.4). Calculate H_1 and B as:

$$H_1 = \left[1 + \left(\frac{L}{H}\right)\tan\beta\right]H \qquad \text{(Equation 6.6)}$$

$$B = \left[\left(\frac{L}{H}\right) + \tan\alpha\right]H \qquad \text{(Equation 6.7)}$$

where:

H = wall height;
L/H = nail length to height ratio (minimum value to use in equation above is 1);
β = backslope angle; and
α = wall batter angle.

- Divide the soil mass subjected to inertia forces into a lower block (with weight W_I) and an upper block (with weight W_{II}). See Figure 6.4 for definitions. Calculate the equivalent, pseudo-static inertia forces (F_I and F_{II}) from blocks I and II as:

$$F_{in} = F_I + F_{II} = \frac{H^2\gamma}{2}A_m\left[0.5\tan\alpha + \left(\frac{L}{H}\right) + 0.5\left(\frac{L}{H}\right)^2\tan\beta\right] \qquad \text{(Equation 6.8)}$$

The inertia force calculated above neglects the vertical seismic coefficient (i.e., $k_V = 0$) and assumes a horizontal seismic coefficient equal to one-half of the design coefficient of acceleration (i.e., $k_h = 0.5\ A_m$). This 50 percent reduction of A_m is based on results from seismic deformation analyses on translational slope failures. These results indicate that cumulative permanent seismic deformation is relatively small (e.g., no more than several centimeters) for a system with a yield acceleration coefficient (i.e., acceleration coefficient resulting in $FS_G = 1.0$) equal to half the peak ground acceleration (PGA) for earthquakes of all magnitudes.

b. Calculate Seismic Active Forces

- Evaluate the dynamic active thrust increment as described below.
  - Calculate total coefficient of active pressure (K_{AE}) using Mononobe-Okabe (Equation 5.22 in Section 5.4.5.4).
  - Calculate coefficient of dynamic increment (ΔK_{AE}) as the difference between K_{AE} and K_A (coefficient of active pressure calculated previously in Step 3A).
  - Calculate dynamic active thrust increment directly as:

$$\Delta P_{AE} = \frac{\gamma H_1^2}{2}\Delta K_{AE} \qquad \text{(Equation 6.9)}$$

- For horizontal backfill, calculate dynamic active thrust increment directly as:

$$\Delta P_{AE} = 0.375 \frac{\gamma H_1^2}{2} A_m \qquad \text{(Equation 6.10)}$$

The coefficient of dynamic increment (ΔK_{AE}) in the equation above is calculated based on a simplified method recommended by Seed and Whitman (1970) for horizontal backfills:

$$\Delta K_{AE} = \frac{3}{4} k_h \qquad \text{(Equation 6.11)}$$

and assuming a horizontal seismic coefficient equal to one-half of the design coefficient of acceleration (i.e., $k_h = 0.5A_m$).

- Assume that dynamic active thrust increment (ΔP_{AE}) acts parallel to the equivalent backslope angle (i.e., $\delta = \beta_{eq}$) (see Figure 6.4).
- Assume that the point of application of the dynamic active thrust increment is at $0.6H_1$ from the base of the wall (see Figure 6.4).

- Calculate the total active force as the combination of the initial, static component (P_A), calculated in Step 3A, and a dynamic active thrust increment (ΔP_{AE}) as:

$$P_{AE} = P_A + \Delta P_{AE} \qquad \text{(Equation 6.12)}$$

c. Seismic sliding stability

- Evaluate sliding stability considering all forces acting on the soil nail wall (e.g., inertia forces and active forces).
- Compute sliding factor of safety using Equation 5.7.
- Verify that calculated FS_{SL} is greater than minimum recommended in Table 5.3 for seismic conditions.
- Redesign if minimum criterion is not met

C. Internal Failure Modes

1) Nail Pullout Resistance (R_P)

The procedure described in Step 3.A.1 "Global Stability" automatically satisfies the condition for pullout resistance (i.e., $T_{max} \leq R_P / FS_P$). In a SNAIL analysis, the calculated nail lengths (corresponding to an acceptable global factor of safety for a given critical failure surface) are based on pullout capacity values, which have already been reduced by the factor of safety with respect to pullout, FS_P.

2) Nail Tensile Resistance (R_T)

- The SNAIL analysis provides (at the end of the output file) the average nail tensile force calculated for a case with $FS_G = 1.0$ ($T_{avg\text{-}s}$).
- Calculate the average nail load (T_{avg}) as the sum of the individual nail forces calculated by SNAIL divided by the number of nails in the analyzed cross section.
- The SNAIL analysis provides the maximum nail tensile force (T_{max}).
- Calculate the maximum nail tensile force ($T_{max\text{-}s}$) as:

$$T_{max-s} = \left(\frac{T_{avg-s}}{T_{avg}}\right) T_{max} \qquad \text{(Equation 6.13)}$$

- Select the nail yield strength (f_y).
- With $T_{max\text{-}s}$, f_y and the factor of safety against tensile failure (FS_T), calculate the required cross sectional area of steel nail bar (A_t) according to:

$$A_t \geq \frac{T_{max-s} FS_T}{f_y} \qquad \text{(Equation 6.14)}$$

- Select the closest commercially available nail bar size (Table A.1).
- Verify that the bar fits in the drillhole subject to a minimum grout cover thickness of 25 mm (1 in.) and the required corrosion protection.

D. Facing Design

1) Calculate design nail head tensile force at the face (T_o) as:

$$T_o = T_{max\text{-}s} \left[0.6 + 0.2\,(S_v[m] - 1)\right] \qquad \text{(Equation 6.15)}$$

2) Select wall facing thickness.

Temporary facing thickness (h): [e.g., 100, 150, 200 mm (4, 6, 8 in.)].
Permanent facing thickness (h): [e.g., 200 mm (8 in.)].

3) Select soil nail wall materials.

Steel reinforcement:
Grade (f_y): 420 MPa (Grade 60), 520 MPa (Grade 75).
WWM features (refer to Appendix A, Table A.2).
Rebar features (refer to Appendix A, Table A.3).
Concrete Strength (f_c'): 21 and 28 MPa (3,000 and 4,000 psi).
Select headed-stud characteristics (Table A.6).
Select bearing plate geometry: min. 200 × 200 mm (8 × 8 in.) and 19 mm (0.75 in.) thick.

4) Verify facing flexural resistance (R_{FF}) for temporary and permanent facing.

a. The minimum reinforcement ratio is calculated as:

$$\rho_{min}[\%] = 20\,\frac{\sqrt{f'_c\,[MPa]}}{f_y[MPa]} \qquad \text{(Equation 6.16)}$$

The maximum reinforcement ratio is calculated as:

$$\rho_{max}[\%] = 0.5\,\frac{f'_c\,[MPa]}{f_y[MPa]}\left(\frac{600}{600 + f_y[MPa]}\right) \qquad \text{(Equation 6.17)}$$

b. Select reinforcement area per unit length of WWM for temporary/permanent facing (Table A.3) at the nail head (a_n) and at mid-span (a_m) in both the vertical and horizontal directions. Typically, the amount of reinforcement at the nail head is the same as the amount of reinforcement at the mid-span (i.e., $a_n = a_m$) in both vertical and horizontal directions.

For temporary facing, if waler bars are used at the nail head in addition to the WWM, recalculate the total area of reinforcement at the nail head in the vertical direction (see Equation 6.18) and horizontal direction (change Equation 6.18 appropriately).

$$a_n = a_m + \frac{A_{vw}}{S_H} \qquad \text{(Equation 6.18)}$$

c. Calculate the reinforcement ratio (ρ) at the nail head and the mid span as (see Section 5.6.3.3):

$$\rho_n = \frac{a_n}{b\,h/2}\;100 \qquad \text{(Equation 6.19)}$$

$$\rho_m = \frac{a_m}{b\,h/2}\;100 \qquad \text{(Equation 6.20)}$$

d. Verify that the reinforcement ratio of the temporary and permanent facing at the mid-span and the nail head are greater than the minimum reinforcement ratio (i.e., $\rho_{min} \le \rho$), otherwise increase the amount of reinforcement (a_n and/or a_m) to satisfy this criterion.

e. Verify that the reinforcement ratio of the temporary and permanent facing at the mid-span and the nail head are smaller than the maximum reinforcement ratio (i.e., $\rho \le \rho_{max}$), otherwise reduce the amount of reinforcement (a_n and/or a_m) to satisfy this criterion.

f. Using Table 5.1, select factor C_F (typically 1 for permanent facings) to take into account the non-uniform soil pressures behind facing.

g. Calculate facing flexural resistance (R_{FF}) for the temporary and permanent facing as:

$$R_{FF}\,[\text{kN}] = \text{minimum of} \begin{pmatrix} 1.6 \times C_F \times (a_{vn} + a_{vm})[\text{mm}^2/\text{m}] \times h[\text{m}] \\ \\ 1.6 \times C_F \times (a_{hn} + a_{hm})[\text{mm}^2/\text{m}] \times h[\text{m}] \end{pmatrix} \quad \text{(Equation 6.21)}$$

Alternatively, calculate the total reinforcement ratio ρ_{tot} as:

$$\rho_{tot} = \rho_n + \rho_m \quad \text{(Equation 6.22)}$$

and use Table 6.4a (interpolate for ρ_{tot} if necessary) and calculate R_{FF} for the temporary/permanent facing.

h. Using the recommended factor of safety for facing flexure (FS_{FF}) listed in Table 5.3, verify that the temporary and permanent facing flexural resistance is higher than nail head tensile force (T_o):

$$R_{FF} \geq FS_{FF}\, T_o \quad \text{(Equation 6.23)}$$

i. If the capacity of the temporary and/or permanent facing is insufficient, increase the thickness of facing, steel reinforcement strength, concrete strength, and/or amount of steel and repeat the facing flexural resistance calculations.

5) Verify Facing Punching Shear Resistance (R_{FP})

a. *Temporary Facing*: With the values of concrete strength (f_c'), facing thickness (h), and bearing plate length (L_{BP}), use Table 6.4b to obtain the punching shear resistance (R_{FP}) for the temporary facing.

b. *Permanent Facing*: With the values of concrete strength (f_c'), headed-stud geometric characteristics and spacing, use Table 6.4c to obtain the punching shear resistance (R_{FP}) for the permanent facing.

Alternatively, use the equations presented in Sections 5.6.4.2 to calculate the punching shear resistance for the temporary and permanent facing.

c. Using the recommended factor of safety for punching shear (FS_{FP}) listed in Table 5.3, verify that that capacity for the temporary/permanent facing is higher than the nail head tensile force:

$$R_{FP} > FS_{FP}\, T_o \quad \text{(Equation 6.24)}$$

d. If capacity for the temporary/permanent facing is not adequate, then implement larger elements or higher material strengths and repeat the punching shear resistance calculations.

6) Facing Headed Stud Resistance (R_{FH}) – Permanent Facing

a. Calculate the maximum tensile resistance due to headed-stud tensile failure (R_{HT}) using Table 6.4d, or alternatively as:

$$R_{FH} = N_H \; A_S \; f_y \qquad \text{(Equation 6.25)}$$

b. Verify that that capacity is higher than nail head tensile force:

$$R_{FH} > FS_{HT} \; T_o \qquad \text{(Equation 6.26)}$$

c. Verify that compression on the concrete behind headed-stud is within tolerable limits by assuring that:

$$A_H \geq 2.5 \; A_S \qquad \text{(Equation 6.27)}$$

$$t_H \geq 0.5 \; (D_H - D_S) \qquad \text{(Equation 6.28)}$$

where:

A_H = cross-sectional area of the stud head;
A_S = cross-sectional area of the stud shaft;
t_H = head thickness;
D_H = diameter of the stud head; and
D_S = diameter of the headed-stud shaft.

d. Provide sufficient anchorage to headed-stud connectors and extended them at least to the middle of the facing section and preferably behind the mesh reinforcement in final facing.

e. Provide a minimum 50 mm (2 in.) of cover over headed-studs.

f. If capacity is not enough, adopt larger elements or higher strengths and recalculate.

7) Other Facing Design Considerations

To minimize the likelihood of a failure at the nail head connection (see Section 5.6.5), use the recommended minimum specifications for the hardware elements provided below. Additional are included in section 5.6.5.

- *Bearing Plates:* Bearing plates should be mild steel with a minimum yield stress, f_y, equal to 250 MPa (ASTM A-36/A36M).
- *Nuts:* Nuts should be the heavy-duty, hexagonal type, with corrosion protection (oversized when epoxy-coated bars are used).
- *Beveled Washers:* Beveled washers (if used) should be steel or galvanized steel. If the plate and other hardware elements are not within the ranges recommended, a formal calculation of capacities should be performed. Note that some proprietary systems employ spherical seat nuts that do not require washers.

TABLE 6.4: FACING RESISTANCE FOR VARIOUS FAILURE MODES (SI UNITS).

(a) FACING RESISTANCE FOR FLEXURE, R_{FF} (TEMPORARY FACING)

h(1)	Nail Spacing Ratio(2)	ρ_{tot} (%) (4)		
		0.5	1.0	2.0
(mm/in.)	[-]	R_{FF} in kN (kip)		
100/4	0.67	50 (12)	105 (24)	210 (48)
	1	75 (18)	155 (36)	315 (71)
150/6	0.67	90 (20)	175 (40)	355 (81)
	1	130 (30)	265 (60)	535 (120)
200/8	0.67	105 (24)	210 (48)	425 (95)
	1	155 (36)	315 (71)	635 (143)

Based on a reinforcement yield strength, f_y, of 420 MPa (Grade 60). For f_y = 520 MPa (Grade 75), multiply the values in the table by 1.24. For permanent facing, for h = 100 mm (4 in.), divide R_{FF} by 2; for h = 150 mm (6 in.), divide R_{FF} by 1.5; for h = 200 mm (8 in.), use same R_{FF}.

(b) FACING RESISTANCE FOR SHEAR PUNCHING, R_{FP} (TEMPORARY FACING)

h(1)	f'c(3)	Bearing Plate Length, L_{BP} (mm/in.)		
		200/8	225/9	250/10
(mm/in.)	(MPa)	R_{FF} in kN (kip)		
100/4	21	140 (32)	150 (35)	165 (37)
	28	165 (37)	175 (40)	190 (43)
150/6	21	245 (56)	265 (60)	285 (64)
	28	288 (65)	305 (69)	325 (74)
200/6	21	380 (85)	400 (91)	425 (96)
	28	435 (99)	465 (105)	495 (111)

(c) FACING RESISTANCE FOR SHEAR PUNCHING, R_{FP} (PERMANENT FACING)

hc(5)	f'c(3)	Headed-Stud Spacing, S_{HS} (mm/in.)		
		100/4	125/5	150/6
(mm/in.)	(MPa)	R_{FP} in kN (kip)		
100/4	21	95 (21)	95 (21)	95 (21)
	28	110 (25)	110 (25)	110 (25)
125/5	21	130 (30)	145 (33)	145 (33)
	28	150 (35)	170 (39)	170 (39)
150/6	21	175 (40)	195 (44)	210 (48)
	28	205 (46)	225 (51)	245 (55)

(d) FACING RESISTANCE FOR HEADED STUD, TENSILE FAILURE, R_{FH}, (PERMANENT FACING)

Headed-Stud Shaft Diameter, D_S		R_{FH}
mm	in.	kN (kip)
9.7	3/8	120 (28)
12.7	1/2	210 (48)
15.9	5/8	330 (75)
19.1	3/4	480 (108)
22.2	7/8	650 (146)

Based on 4 headed-studs and a yield strength, fy, of 420 MPa (Grade 60). For f_y = 520 MPa (Grade 75), multiply the values in the table by 1.24

Notes:

(1) h is the facing thickness.

(2) Nail space ratio is the lowest of either S_V/S_H or S_H/S_V, resulting in a value less than or equal to 1.

(3) f'_c is the concrete nominal compressive strength.

(4) ρ_{tot} is the total reinforcement ratio calculated as $\rho_{tot} = \rho_n + \rho_m$, where ρ_n and ρ_m are the nail head and mid-span reinforcement ratios, respectively. $\rho_i = a_{ij}/0.5h$, where a_{ij} = cross sectional area of reinforcement per unit width in "i" direction (vertical or horizontal) and at location "j" (nail head or midspan).

(5) $h_c = L_s - t_H + t_P$ where: L_s is the effective headed-stud length (Table A.6); t_P is the bearing plate thickness [typically 19 mm (0.75 in.)]; t_H is the headed-stud head thickness (Table A.6).

6.5 STEP 4: DEFORMATION ESTIMATES

1) Use Figure 5.23 as a guide to estimate the magnitude of vertical and horizontal displacements.
2) Obtain wall height (H) and batter angle (α) (see Figure 5.23 for a description of variables).
3) Identify ground conditions (i.e., weathered rock/stiff soil, sandy soil, clayey soil).
4) Estimate horizontal and vertical displacements δ_h and δ_v at the top of the wall

Variable	Weathered Rock and Stiff Soil	Sandy	Clayey
δ_h/H and δ_v /H	**1/1000**	**1/500**	**1/333**
C	**1.25**	**0.8**	**0.7**

5) Calculate zone of influence, D_{DEF}, where noticeable ground deformations occur:

$$\frac{D_{DEF}}{H} = C\,(1 - \tan\alpha) \qquad \text{(Equation 6.29)}$$

6) Verify estimated displacement against maximum deformation criteria.
7) If in a highly seismic area, determine the need to perform seismic deformation analysis (i.e., Newmark analysis). One potential scenario for these types of analyses is when soft sites and there exist a potential for site amplification (Kramer, 1996; Kavazanjian et al., 1996).

6.6 STEP 5: OTHER DESIGN CONSIDERATIONS

A. Drainage

- Evaluate the surface water and groundwater conditions at the site.
- Consider the installation of surface water control (e.g., channels) and groundwater control (e.g., geocomposite drains, weep holes, pipe drains) systems to avoid construction difficulties and/or long-term performance deterioration (See Section 5.11.1 and Figures 5.25 and 5.26 for details).
- At sites where drains will be installed but they cannot be properly inspected and/or maintained, consider performing analysis of soil nail walls assuming a high groundwater table for long-term conditions.

B. Frost Penetration

- Evaluate the depth of frost penetration.
- Evaluate the soil susceptibility to frost action as follows:
  - evaluate coefficient of uniformity (C_u) from grain size distribution curves;

- calculate the fraction of soil smaller than 0.02 mm; and
- soils are frost susceptible if fraction is greater than 3 percent for non-uniform soils (i.e., $C_u > 5$), or greater than 10 percent for uniform soils (i.e., $C_u \leq 5$).

- If soil is susceptible to frost action, provide a facing thickness greater than the frost penetration depth. Use insulators to reduce thickness requirements (see Figure 5.27 and Chapter 5.11.2 for details).

C. External Loads

- Identify appropriate loading conditions, including permanent loads (e.g., self-weight, nearby foundations); and transient loads (e.g., construction traffic).
- Assume a typical surcharge load due to traffic of 12 kPa (75 psf). To take into consideration the effect of storage and heavy construction equipment. Also, consider a load of 15 kPa (about 100 psf) for temporary conditions.

D. Support for Facing Dead Load

- For the initial lifts, for temporary facing thicker than 100 mm (4 in.), evaluate the ability of the nails to support the weight of temporary facing by shear and friction between the facing and the soil.
- If the nails cannot provide adequate support of the temporary facing, install additional short, steeply inclined reinforcing elements acting as compression struts (see Figure 5.28).

6.7 EXAMPLE PROBLEM

The following section presents a step-by-step example problem that illustrates the recommended design procedures described previously in this chapter using only the simplified design charts of Appendix B. A complete and more detailed design example is presented in Appendix.

The project consists of a wall near the access of a non-critical, lightly trafficked road. The site is rural. No buildings are located near the proposed location of the soil nail wall. The area is flat and the elevation of the groundwater table is significantly below the bottom of the proposed excavation.

- **Initial Project Considerations**

1) Construction restrictions
   a. Topography: no sloping terrain behind or in front wall (assume β=0).
   b. Site access: easy site access.
   c. ROW: no limitations.
   d. Utilities: no disturbance of utilities.
   e. Adjacent structures: none; extent of deformed zone behind wall is not an issue.
   f. Other geometric constraints: no headroom limitations; nails can be installed without difficulties behind wall.

- **Other Requirements**
  1) Project design life: wall is for a temporary excavation expected to be open a few months. Assume temporary structure.
  2) Aesthetics requirements: none.
  3) Performance criteria: conventional.
  4) Deformation performance: conventional

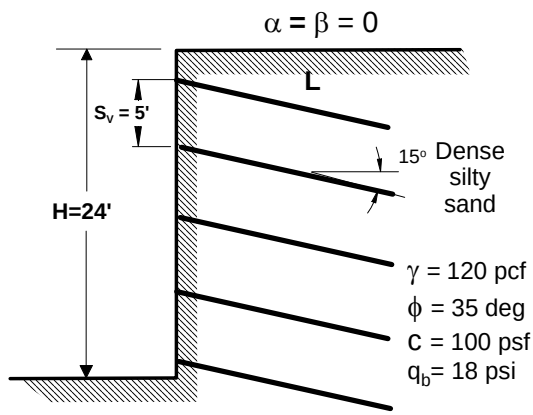


Figure 6.6: Example.

STEP 1: INITIAL CONSIDERATIONS AND PARAMETERS

A. Wall layout: Height, H = 8 m (24 ft); wall length >> H; face batter: $\alpha = 0$; slope: $\beta = 0$;

B. Soil nail spacing: $S_H = S_V = 1.5$ m (5 ft)

C. Soil nail pattern on wall face: uniform

D. Soil nail inclination: i = 15 degrees

E. Soil nail length distribution: uniform

F. Soil nail materials: Nail bar: Grade 420, $f_y = 420$ MPa (60 ksi)

G. Soil properties

From site investigations, dense to very dense silty sands with the following properties were encountered:

$\phi' = 35$ degrees, $c' = 5$ kPa (100 psf), $\gamma = 18.9$ kN/m^3 (120 pcf)

Ultimate bond strength is selected as:

$q_u = 125$ kPa = 18 psi

Select $D_{DH} = 150$ mm (6 in.) in step H below.

Select FS_P = 2.0 in step H below.

Calculate normalized bond strength:

$$\mu = \frac{q_u \; D_{DH}}{FS_P \gamma \, S_H \; S_V} = \frac{18\,\text{psi} \times (144\,\text{in.}^2/\text{ft}^2) \times 6\ \text{in.} \times (1\ \text{ft}/12\ \text{in.})}{2\ \times\ 120\,\text{pcf} \times 5\,\text{ft} \times 5\,\text{ft}} = 0.22$$

H. Other design considerations

1) Drilling method: For the existing ground conditions, conventional rotary drilling is possible (Chapter 3).

2) Drill hole diameter: Select a minimum diameter compatible with these conditions: D_{DH} = 150 mm (6 in.)

3) Corrosion Potential: The ground corrosion potential is unknown. Per Appendix C guidelines, for temporary structures and ground with unknown aggressivity, provide Class II corrosion protection as a minimum. Provide a minimum grout thickness of 25 mm (1 in.) over nail bars.

4) Minimum Factors of Safety: For the following conditions: Temporary structure, static loads only, non-critical structure, use:

Failure Mode	**Resisting Component**	**Symbol**	**Minimum Recommended Factors of Safety**
External Stability	Global Stability	FS_G	1.35
	Global Stability	FS_G	1.2-1.3
	Sliding	FS_{SL}	1.3
	Bearing Capacity	FS_H	2.5
Internal Stability	Pullout Resistance	FS_P	2.0
	Nail Bar Tensile Strength	FS_T	1.5
Facing Strength	Facing Flexure	FS_{FF}	1.35
	Facing Punching Shear	FS_{FP}	1.35
	Headed-Stud Tensile (A307 Bolt)	FS_{HT}	1.8

- **Loading Conditions**

1) From Table 5.5:

a. Dead load (self-weight is the only dead load)

b. Live Load (minimal during construction, and neglected beyond construction)

c. No wind

d. No impact

e. No seismic load

STEP 2: PRELIMINARY DESIGN

Nail Length and Diameter using simplified charts

For $\alpha = 0$, $\beta = 0$, use Figure B.1. With $\phi' = 35$ degrees and $\mu = 0.22$

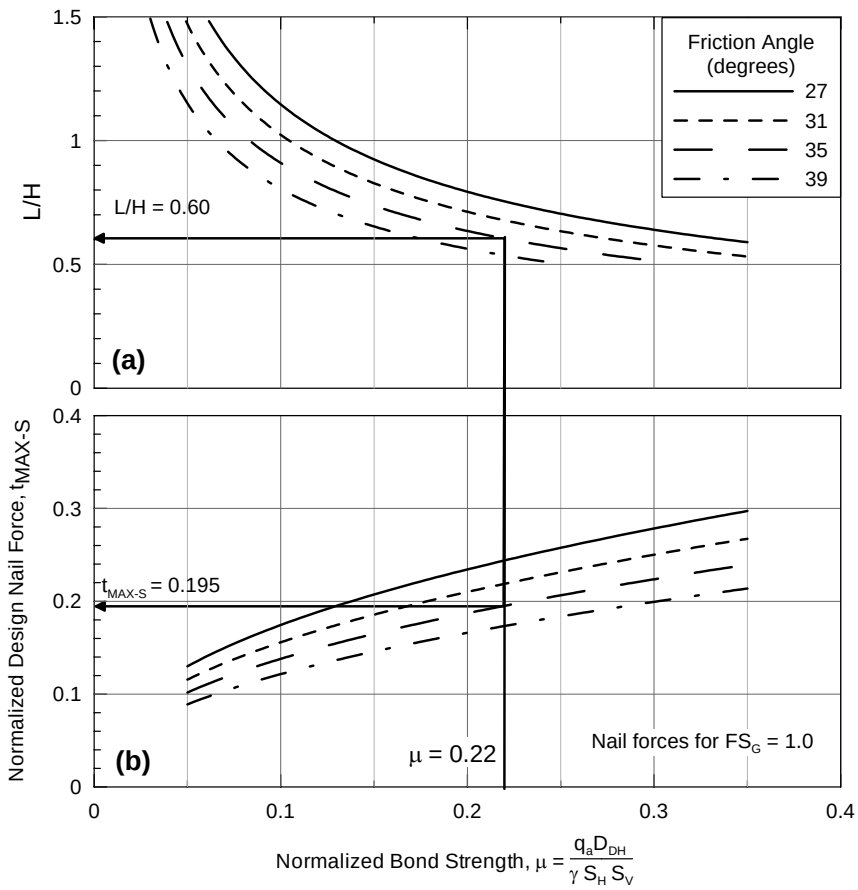
$L/H = 0.60$

$t_{max\text{-}s} = 0.195$

Nail Length

Correction for drillhole diameter, $C_{1L} = 0.83$ (from Figure B.7)

L = 0.60 x 24 ft x 0.83 = 12.0 ft. Check: 12.0/24 = 0.50. This value is on the low side. Adopt L = 16 ft (L/H = 0.67) to reduce potential wall deformations.



Calculate Design Nail Tensile Load

Correction for drillhole diameter, $C_{1F} = 1.48$

$$T_{max\text{-}s} = t_{max\text{-}s}\ \gamma\ S_H\ S_V\ H\ C_{1F} = 0.195 \times 120 \times 5 \times 5 \times 24 \times 1.48 / 1{,}000 = 20.8 \text{ kip}$$

Required cross sectional area of steel

$$A_t = \frac{T_{max-s}\ FS_T}{f_y} = \frac{20.8 \times 1.5}{60} = 0.52 \ \text{in}^2$$

Select a No. 25 (No. 8) threaded bar with cross sectional area A = 510 mm^2 (0.79 in^2) and diameter = 25 mm (1.12 in.) (see Table A.1).

Bar installation: This bar can be installed with no difficulty in the drillhole. Available cover is at least (6 - 1.12) / 2 = 2.44 in. = 62 mm > minimum cover = 25 mm.

STEP 3: FINAL DESIGN

A. External Failure Modes

Only the preliminary design developed in Step 2 is considered in this example.

B. Seismic Considerations

No seismic considerations are necessary in this example.

C. Internal Failure Modes

Only the preliminary design developed in Step 2 is considered in this example.

D. Facing Design

1) Maximum design tensile force at the face:

$$T_o = T_{max\text{-}s}\left[0.6 + 0.057\,(S_{max}[\text{ft}] - 3)\right] =$$
$$= 20.8\left[0.6 + 0.057\,(5-3)\right] = 19.5 \times 0.71 = 14.9 \text{ kip}$$

2) Facing Features

a) Wall facing type:

Temporary: Shotcrete + WWM

Permanent: Cast-in-Place Reinforced Concrete

b) Facing geometry

Temporary facing thickness, h = 100 mm (4 in.)

Permanent facing thickness, h = 200 mm (8 in.)

Steel reinforcement

f_y = 420 MPa (60 ksi)

Temporary facing reinforcement: Select WWM: 4x4 – W2.9 x W2.9

Select Rebar: horizontal and vertical waler bars: 2 × No. 3, Grade 60

A_s = 2 × 0.11 = 0.22 $in.^2$) in both directions)

Permanent facing reinforcement: No. 5 @ 12 in., each way

Concrete Strength, f_c' = 21 MPa (3,000 psi)

Select headed-stud characteristics:

4 H-Studs: $^1/_2 \times 4^1/_8$, Grade 60, L_s = 100 mm (4 in.), D_H = 25 mm (1 in.), D_S = 13 mm (0.5 in.), t_H = 8 mm (0.3 in.), S_S = 150 mm (6 in.), headed-stud spacing, S_{HS} = 150 mm.

Select bearing plate geometry

Bearing Plate, Grade 36, Length: L_{BP} = 225 mm (9 in.), thickness: t_P = 25 mm (1 in.).

3) Facing Reinforcement

Temporary Facing

a) Reinforcement in vertical and horizontal directions in midspan

Area per unit length (WWM for temporary facing) (Table A.2)

$a_{vm} = a_{hm}$ = 0.087 in.^{2}/ft for WWM 4 × 4 – W2.9 × W2.9.

b) Reinforcement in vertical and horizontal directions around soil nail head

$$a_{vn} = a_{hn} = a_{vm} + \frac{A_{vw}}{S_H} = 0.087 + \frac{0.22}{5} = 0.13 \text{ in}^2/\text{ft}$$

c) C_F = 2.0 (Table 5.1)

d) $\rho_n = a_{vn} / 0.5\ h$ = (0.13 in.^{2}/ft/12)/(0.5 × 4 in.) × 100 = 0.54 %

e) $\rho_m = a_{vn}/ 0.5\ h$ = (0.087 in.^{2}/ft/12)/(0.5 × 4 in.) × 100 = 0.36 %

f) $$\rho_{min}[\%] = 0.24 \frac{\sqrt{f'_c[\text{psi}]}}{f_y[\text{ksi}]} = 0.24 \frac{\sqrt{3{,}000}}{60} = 0.22\ \%$$

g) $$\rho_{max}[\%] = 0.05 \frac{f'_c[\text{psi}]}{f_y[\text{ksi}]}\left(\frac{90}{90 + f_y[\text{ksi}]}\right) = 0.05 \frac{3{,}000}{60}\left(\frac{90}{90+60}\right) = 1.5\ \%$$

h) ρ_m = 0.36 % > ρ_{min} = 0.22 % ✓

ρ_m = 0.36 % < ρ_{max} = 1.5 % ✓

i) ρ_n = 0.54 % > ρ_{min} = 0.22 % ✓

ρ_n = 0.54% < ρ_{max} = 1.5% ✓; ρ_n/ρ_m = 1.5 < 2.5 ✓

j) $\rho_{tot} = \rho_n + \rho_m$ = 0.54% + 0.36 % = 0.90 %

Permanent Facing

a) Area per unit length for No. 5 @ 12 in., each way (Table A.3)

$a_{vn} = a_{vm}$ = 0.31 in.^{2}/ft.

b) Total reinforcement in vertical direction (no waler bars)

$a_{vn} = a_{vm} = 0.31 \text{ in.}^2/\text{ft}$

c) $C_F = 1.0$ (Table 5.1)

d) $\rho_{min}[\%] \geq 0.24 \dfrac{\sqrt{f'_c[\text{psi}]}}{f_y[\text{ksi}]} = 0.24 \dfrac{\sqrt{3{,}000}}{60} = 0.22\ \%$

e) $\rho_{max}[\%] \geq 0.05 \dfrac{f'_c[\text{psi}]}{f_y[\text{ksi}]}\left(\dfrac{90}{90 + f_y[\text{ksi}]}\right) = 0.05 \dfrac{3{,}000}{60}\left(\dfrac{90}{90+60}\right) = 1.5\%$

f) $\rho_n = \rho_m = 0.31/12/(0.5 \times 8) \times 100 = 0.64\%$ (satisfies both ρ_{min} and ρ_{max})

g) $\rho_{tot} = 1.29\%$

4) Facing Tensile flexural Resistance (R_{FF}) - Temporary and Permanent Facing

a) Facing flexural resistance (R_{FF})

Temporary: From Table 6.4a, R_{FF} = 140 kN (32 kip)

Permanent: From Table 6.4a, R_{FF} = 412.8 kN (93 kip)

b) Verify: $FS_{FF}\ T_o < R_{FF}$

Temporary: 1.35 × 14.9 = 20.1 kip < 32 kip ✓

Permanent: 1.5 × 14.9 = 22.4 kip < 93 kip ✓

5) Facing Punching Shear Resistance (R_{FP}) – Temporary and Permanent Facing

a) Facing flexural resistance (R_{FP})

Temporary: From Table 6.4b, R_{FP} = 150 kN (35 kip)

Permanent: h_c = 4 in. - 0.3 in. + 1 in. = 4.7 in. (120 mm)

From Table 6.4c, with S_{HS} = 150 mm (6 in.)

R_{FP} = 145 kN (32 kip)

b) Verify: $FS_{FP}\ T_o < R_{FP}$

Temporary: 1.35 × 14.9 = 20.1 kip < 35 kip ✓

Permanent: 1. 5 × 14.9 = 22.4 kip < 32 kip ✓

6) Facing Head Stud Resistance (R_{HT}) – Permanent Facing

a) Maximum tensile resistance (headed-stud tensile failure) (R_{HT})

$$R_{HT} = N_H\ A_{SH}\ f_y = 4 \times 0.196 \text{ in.}^2 \times 60 \text{ ksi} = 47 \text{ kip}$$

b) Verify: $FS_{FP}\ T_o$ = 1.8 × 14.9 = 26.7 < 47 kip ✓

CHAPTER 7 CONTRACTING APPROACHES AND SPECIFICATIONS

7.1 INTRODUCTION

Two types of contracting methods are used to develop contract drawings and specifications for permanent and temporary soil nail wall systems. Some agencies prefer one approach to the other or a mix of approaches for soil nail wall systems based on the criticality and complexity of a particular project, experience of the owner and their engineering consultants, and the availability of specialty contractors. The selection of a contracting method may also depend on the experience of the agency with soil nail wall technology. Both contracting approaches are valid if properly implemented and each has advantages and disadvantages. These contracting methods include:

- *Procedural or Method Approach.* All details of design, construction materials and methods are specified in the contract documents. A variant to this method allows the contractor to select nail installation methods required to achieve specified nail capacities, while specifying requirements for nail diameter, length, and corrosion protection.
- *Performance or End-Result Approach.* Lines and grades with specific design criteria and methods and performance requirements are provided in the contract documents. With this approach, a project-specific review and detailed plan submittal occurs in conjunction with the submittal of working drawings.

Because specialty contractors often introduce innovative, cost-competitive solutions, it is recommended that the contract documents for soil nail projects be structured to allow specialty contractors to make use of the latest available construction techniques. Thus, contract documents that are performance-based, with respect to drilling and grouting methods, are recommended. Either the method or performance approach allows this flexibility.

In this section, contracting procedures and guideline information are summarized. Specifications are provided in Appendix E for both method- and performance-based approaches. These specifications can be readily modified for a specific project and can be referenced whenever possible to the agency standard specifications. To facilitate tailoring the following guidelines specifications to specific projects, comments have been added throughout Appendix E and are shown in italics.

7.2 CONTRACTOR'S PRE-QUALIFICATION

The procedures for soil nail construction need to be followed closely to provide a high quality product and avoid undesirable construction conditions that could jeopardize the structure integrity and represent a hazard to workers. The likelihood of obtaining a satisfactory soil nail installation increases when an experienced and qualified soil nail contractor is selected to construct the wall. The likelihood of success further increases when comprehensive construction quality assurance procedures are developed and enforced by the owner or their inspection agent. Agencies or Owners with no previous soil nailing experience should seek a qualified third-party quality assurance provider.

Contract documents for soil nailing projects should clearly define: (1) the contractor pre-qualification requirements, (2) submittal procedures for this qualification information, and (3) the means by which the owner will enforce these requirements. Such pre-qualification requirements are commonly being used by many transportation agencies for other specialty construction techniques, such as permanent ground anchor walls, micropiles, drilled shaft foundations, and shotcreting. However, for some public agencies, it is prohibited by law to "pre-qualify" contractors and any "pre-qualifications" of the more experienced contractors must be disregarded. To circumvent this limitation, agency officials can require bidders to demonstrate that they meet specific soil nail experience requirements.

Typical soil nail contractor pre-qualification requirements are included in the specifications presented in Appendix E.

7.3 PROCEDURAL (METHOD) SPECIFICATIONS

The procedural (or method) contracting approach includes the development of a detailed set of plans and specifications to be provided in the bidding documents. The advantage of this approach is that complete design details and specifications are developed and reviewed by experienced representatives. This approach further empowers agency engineers to examine options that may be available during design but requires engineering staff trained in soil nail wall design and construction methods. The staff then also becomes a valuable asset during construction, when questions and/or design modifications are required.

Under this contracting procedure, the agency is fully responsible for the design and performance of the soil nail system, as long as the contractor has installed each component (e.g., nails, facing, drainage) in strict accordance with the contract documents. The agency assumes all risks and is responsible for directing the work if changes to the design are required based on actual field conditions.

The use of a variant to this method, in which the contractor is responsible for developing the required nail capacity by varying the drilling and grouting methods, drill hole diameter, and length of nails from specified minimums, has several advantages. It empowers contractors to maximize the use of their experience and specialized equipment and allows the agency to share the major risk, (i.e., nail capacity for a specified length), with the contractor. To implement this approach, the following information must be included in a special provision to the contract:

- results of the geotechnical investigations, including all laboratory test results;
- submittal requirement for the contractor to provide information outlining its drilling and grouting methods;
- minimum drillhole diameters and nail length (alternatively, this information can be provided by the contractor); and
- required soil nail design loads at each level or location.

The use of a method specification is recommended only for agencies that have developed sufficient in-house expertise and consider soil nail wall design and construction control as a conventional or standard method for earth retention.

7.4 PERFORMANCE SPECIFICATIONS

This approach is often called "line and grade", "conceptual plans", or "design-build." Under this approach, the agency: (1) prepares drawings defining the geometric and aesthetic requirements for the structure and material specifications for the components, (2) defines performance requirements including factors of safety, and (3) indicates the range of acceptable construction and design methods.

This approach, when coupled with sound specifications and pre-qualification of contractors, offers several benefits. Design of the structure is performed by trained and experienced staff and can utilize contractors' proprietary equipment and methods along with material components successfully and routinely used in the past. Also, the performance specification approach lessens engineering costs and manpower requirements for the agency and transfers some of the project design costs to construction. The disadvantage is that agency engineers must have adequate expertise in soil nailing to perform a design review, approve construction modifications, and engage a consultant with demonstrated proficiency in this technology. It is noted that the limitations previously described under the method approach for pre-qualification of contractors for certain public agencies also apply to pre-qualification with the performance contracting approach.

Bid quantities are obtained from specified pay limits denoted on the "line and grade" drawings and can be bid on a lump sum or unit-price basis per square meter (or square feet) of wall face. The basis for detailed designs to be submitted after contract award is detailed as a special provision, as would construction control and monitoring requirements. The special provision should clearly identify the required submittals, schedule for submittals, and schedule for review and approval of these submittals. Items to be contained in the special provision are submittals of: (1) calculations, (2) shop drawings, and (3) product information.

Plans furnished by the owner as part of the contract documents should contain the geometric, geotechnical, and design-specific information listed below:

- plan and elevation of the areas to be retained, including beginning and end stations, top of wall, bottom of wall and original ground line;
- typical cross-section indicating, as a minimum, face batter, pay limits, drainage requirements, and excavation limits;
- elevation view of each structure showing original ground line, minimum foundation level, finished grade at ground surface and top of wall or slope line;
- location of utilities, signs, and any other structures and the loads imposed by each such appurtenance, if any;
- construction constraints such as staged construction, ROW, and construction easements;

- magnitude, location, and direction of external loads due to bridges, overhead signs and lights, and traffic surcharges;
- limits and requirements of drainage features beneath, behind, above, or through the structure;
- reference to specific governing sections of an agency design manual (materials, structural, hydraulic and geotechnical), construction specifications, special provisions, and performance criteria for the soil nail wall; the agency may specify as part of the special provisions, acceptable design methods by referencing one or more of the methods outlined in this GEC, or qualify methods that the contractor uses as part of its pre-qualification process;
- results of all geotechnical investigations on site;
- submittals of drawings and construction methods;
- safety factors and material properties;
- level of corrosion protection required;
- finished face aesthetic requirements;
- wall alignment tolerances and allowable horizontal movements;
- percentage of nails to be tested, testing procedures, and acceptance criteria; and
- wall construction monitoring requirements.

Performance specifications are recommended for agencies with limited experience with this technology or for complicated projects where a specialty contractor's specific or local knowledge can be maximized.

CHAPTER 8 CONSTRUCTION INSPECTION AND PERFORMANCE MONITORING

8.1 INTRODUCTION

The purpose of this chapter is to provide guidance regarding construction inspection and performance monitoring of soil nail walls. Inspection is the primary mechanism to assure that the soil nail wall is constructed in accordance with the project plans and specifications. Short-term and long-term performance monitoring is conducted to assess the performance of the soil nail wall. The owner agency, the contractor, or a combination of both can carry out the construction inspection activities, depending on the contracting approach [i.e., procedure (method) or performance). Performance monitoring is usually conducted by the owner agency. Inspection and monitoring of permanent soil nail walls that are constructed using performance specifications are described in this chapter.

Inspection activities, if properly conducted, play a vital role in the production of a high-quality soil nail wall because conformance to project plans and specifications should result in a soil nail wall that will perform adequately for the intended service life. Inspection may involve evaluation of the following: (1) conformance of system components to material specifications; (2) conformance of construction methods to execution specifications; (3) conformance to short-term performance specifications (i.e., load testing); and (4) long-term monitoring, if required by contract. A valuable source of information on proper soil nail construction inspection practices is provided in the "*Soil Nailing Field Inspector's Manual*" (Porterfield et al., 1994).

Monitoring activities may include short-term or long-term measurements of soil nail wall performance. Short-term monitoring is usually limited to monitoring measurements of soil nail wall performance during load testing (i.e., proof, verification, and creep tests). In some cases, short-term monitoring may include monitoring lateral wall movements and ground surface settlements. Oftentimes this monitoring is motivated by performance requirements. Long-term monitoring of the soil nail wall usually includes a continuation of measurements from short-term monitoring.

8.2 INSPECTION ROLES

For a soil nail wall contracted using the method approach, inspection activities are carried out by the owner agency based on comprehensive material and procedural requirements of owner-provided plans and specifications. The Contractor's responsibility is to follow the project plans and specifications. The owner's inspection is conducted to assure strict compliance with each component of the plans and specification. Prior to construction of a soil nail wall, personnel responsible for construction control and monitoring should become thoroughly familiar with the following items:

- plans, specifications, and testing requirements;
- site conditions relevant to soil nail wall construction conditions;
- material requirements and allowable tolerances;

- construction sequencing; and
- pre-qualification requirements for specialty contractors and necessary data to comply with this requirement.

Quality assurance measures must be implemented during construction to ensure that:

- construction is being performed in accordance with plans and specifications;
- allowable excavation heights are not exceeded;
- nail drillholes have not caved during nail installation;
- nail bars are of the right size and type (i.e., steel grade, length, diameter);
- corrosion protection systems are in compliance;
- grouting, installation of facing rebar and mesh, and shotcrete are in compliance with respect to materials and methods;
- nail pullout testing verifies the design values required by the specifications; and
- required drainage is properly installed.

8.3 INSPECTION OF CONSTRUCTION MATERIALS

8.3.1 Construction Materials

The quality of all materials used is controlled on-site by one or a combination of the following procedures:

- visual examination for defects due to poor workmanship, contamination, or damage from handling;
- certification by the manufacturer or supplier that the materials comply with the specification requirements; and/or
- laboratory testing of representative samples from materials delivered to the site or approved storage area.

Steel components (e.g., nail bars, bearing plates, nuts, washers, reinforcing steel), centralizers, grout components, epoxy coating, sheathing, cement, additives, geocomposite drainage material, and piping are normally accepted based on mill certificates.

Grout for nails and facing shotcrete is initially accepted on the basis of the Engineer's approval of the mix design. Compressive tests are specified to determine the acceptability of the as delivered material.

8.3.2 Storage

Nails, cement, bars, and drainage materials must be kept dry and stored in a protected location. Note that bars should be placed on supports to prevent contact with the ground.

8.3.3 Inspection of Corrosion Protection

The common methods to protect nails from corrosion include encapsulation, epoxy coating, grout protection, or a combination of these measures. Encapsulated bars are usually delivered to the site completely assembled. A representative number of encapsulated nails should be checked for voids in the grout placed in the annular space between the nail and the corrugated tube. This can be done by lightly tapping the encapsulation with a steel rod and listening for hollow sounds indicating the presence of voids. The epoxy coating should be visually examined for damage. Corrosion protection is a critical component of most permanent soil nails. Soil nails with damaged corrosion protection should be either repaired or replaced. Any repairs should be done in strict accordance with manufacturer's recommendations.

8.4 INSPECTION OF CONSTRUCTION ACTIVITIES

8.4.1 Introduction

As previously described, soil nail walls are constructed in staged lifts using "top-to-bottom" construction with each lift completed to closure (i.e., with nail testing and shotcrete facing completed) prior to excavating subsequent lifts. It is the responsibility of the inspection staff to ensure that all required construction activities and testing for each lift has been completed in accordance with the contract specifications and plans. Inspection and Quality Assurance/Quality Control (QA/QC) activities are listed below:

- inspecting nail bars for damage, required length, and checking mill test certificates to certify grade and corrosion protection;
- verifying that the stability of the excavated face is maintained at all stages of construction; if stability cannot be maintained at the initial depth of cut, the depth of subsequent excavation lifts should be reduced and followed by immediate shotcreting;
- verifying that the nails are installed to the correct orientation, spacing, size/grade, and length; in drilling the drillhole, the contractor must maintain an open hole without any loss of ground, otherwise casing must be used; subsidence of ground above the drilling location or large quantities of soil removal with little or no advancement of the drill head should not be permitted; drilling mud should not be permitted because bentonite residue on the drill-hole perimeter will likely reduce the capacity of the nail;
- verifying that centralizers are used to provide proper location of the nails in the drilled hole; insertion of the bar may be done before or after tremie grouting the drillhole; centralizers must be placed along the length of the nail such that flow of grout in the borehole is not impeded;
- inability to achieve the required nail length in uncased holes is usually a sign of caving and may require re-drilling;
- verifying that proper grouting of the borehole around the nail bar is employed; the grouting operation involves injecting grout at the lowest point of the drill hole in order to fill the hole evenly without air voids (i.e., via a tremie pipe);

- verifying that shotcrete is placed to the required thickness, is placed in accordance with standard practice, and that the facing reinforcement is installed in accordance with the specifications and plans;
- verifying proper placement of the bearing plate; deviations of perpendicularity between the plate and nail should be adjusted by using tapered washers below the nut;
- verifying proper installation of drain pipes, weepholes, and prefabricated vertical drains; it is essential that hydraulic continuity of the vertical drains be assured if installed incrementally; and
- verifying that grout cubes and shotcrete cores are taken for strength testing of permanent walls.

Soil nail wall specifications (see Appendix E) require that the contractor select a soil nail installation method and drillhole diameter that will develop the required soil nail bond strength used to develop the design. Because only a percentage of the nails will be tested, it is important that consistent and similar construction methods be used for all nails. Therefore, the inspector must observe and document the installation conditions and method of installation for each nail as well for all verification and proof testing.

The inspection staff should make daily inspections of the area adjacent to the soil nail wall and check for any surface cracking. Tension cracks often form in the ground surface located immediately behind the top of the nail wall excavation. These cracks should be monitored, and the contractor and design engineer contacted, if displacements become excessive, or rapidly increase in size. Water should not be allowed to enter tension cracks.

For some projects, construction monitoring devices and installation of instrumentation, such as slope inclinometers, surface survey points, load cells, or strain gauges, may be required. The installation methods should be covered in the plans and specifications and should be the responsibility of the contractor to maintain these devices during construction.

A discussion of inspection issues for each of the major construction steps is provided below. Note that Porterfield et al. (1994) provides considerably greater details on inspection methods, nail testing, inspection forms, and handling of difficult ground conditions during construction.

8.4.2 Excavation

The two types of excavation that generally occur during construction of a soil nail wall are:

- mass excavation, which is conducted to provide equipment access and general site grading; and
- excavations required for construction of the soil nail wall, (i.e., excavation to the plan finish wall line, or “neat line”).

During mass excavation, the inspection staff must verify that the excavation does not encroach upon the partially completed soil nail wall because uncontrolled excavation near the wall location could affect the stability of the wall. The contractor is responsible for completing the excavation to neat

line and grade, and the inspector should verify compliance. The contractor must address conditions such as overbreaks, or sloughing of the cut face, as well as protrusion of cobbles or boulders beyond the line of the cut face. Limited overbreaks can typically be backfilled with shotcrete, placed at the same time as the facing shotcrete.

Finished excavation and cut face exposures that will not receive shotcrete facing within the specified time limit must be stabilized by placing a soil berm against the exposed face, placing a temporary shotcrete flash-coat, or installing another approved method.

8.4.3 Soil Nail Drillhole Drilling

The soil nail drillholes should be located as shown on the plans and within the specification tolerances. Generally, the angle of the drill mast, (as measured with a magnetic angle tool), is used to check the angle of the drillhole. Most soil nails for permanent applications are installed using small hydraulic, track-mounted drill rigs. These rigs are mostly of the rotary/percussive type that use sectional augers or drill rods. For deeper soil nail excavations requiring longer nail lengths, larger hydraulic-powered track-mounted rigs with continuous-flight augers may be used.

The method of drilling to be used by the specialty contractor will depend on site and ground conditions and available drilling equipment. However, the most frequently used method is open-hole drilling. The most common grouting method used with open-hole drilling is the low-pressure tremie method. Cased-hole methods of drilling are required in more difficult ground conditions (i.e., ground with caving drillholes).

Drillholes in soil should be kept open only for short periods of time. The longer the hole is left open, the greater the risk of caving or destressing of the soil. A mirror or a high intensity light should be used, prior to nail installation, to inspect the hole for cleanliness. Soil that may have sloughed into the hole should be removed either by redrilling or by cleaning with a tool, if feasible.

8.4.4 Nail Installation

The inspection staff should check each nail to ensure that the length, diameter, steel grade, centralizers, and corrosion protection (as required) are in accordance with the plans and specifications. The nail must be inserted into the hole to the minimum specified length. The inability to do so indicates an unacceptable condition caused by caving/sloughing of the hole and/or insufficient drilled length.

Nails must be handled carefully to avoid damage. Single point lifting should only be allowed for short nails that do not bend when handled in this manner. Longer nails and all encapsulated nails should be picked up at two or three equally spaced points. The centralizers should be stiff and large enough to provide space for the minimum specified grout cover. Centralizers should be spaced closely enough to each other to keep the bar from sagging and touching the bottom of the hole, but should not impede the free flow of tremied grout into the hole.

When using hollow-stem augers method in combination with stiffer grout with a slump no greater than 200 mm (8 in.), centralizers may not be required. Nails that are driven or pushed beyond the drillhole length or cut off should not be accepted.

8.4.5 Grouting

The primary inspection activity associated with grouting involves verifying that the entire length of the nail is grouted without any voids or gaps in the grouted column. To minimize the potential for drillhole caving, open-hole tremie grouting should be performed as soon as possible after drilling and immediately following nail insertion. Grout should be injected by tremie pipe inserted to the bottom of the drillhole, to ensure that the grout evenly and completely fills the hole from the bottom to the surface without air voids. The grout should flow continuously as the tremie pipe is withdrawn. The withdrawal rate should be controlled to ensure that the end of the tremie pipe is always below the grout surface. A record of the volume of grout placed should be maintained. When hollow-stem auger methods are used, the contractor should not be allowed to reverse the auger rotation during extraction. This action forces soil to mix with the grout, thus reducing grout/ground bond strength.

In the only case that partial grouting is used is determination of bond strength from nail testing. Test nails require partial grouting of the nail drillhole to develop bonded and unbonded test lengths. Typical methods for establishing the top of the test bonded length include placing, prior to insertion of the nail into the drill hole, a painted centralizer, or a piece of survey ribbon on the nail, at the desired point of grout termination (top of bonded zone). Once the nail test is completed, the actual bonded length can be determined by the marker reference. Subsequent to testing and acceptance of the results, the unbonded length of all test nails must be filled with grout.

8.4.6 Drainage Strips

Once the final wall line excavation and nail installation have been completed for each lift, the geocomposite drain strips are typically placed vertically, at specified intervals. Drain strips must be continuous from the top to the bottom of the wall. The inspector should verify that splices are made with a minimum 300-mm (12-in.) overlap (or per specification or manufacturer's recommendations) to assure that water flow is not impeded. If the geotextile component of the drainage strip becomes torn or punctured, the damaged section should be replaced completely or repaired with a piece of fabric overlapping the damaged area. Maintenance of drainage continuity and capacity is critical to the overall stability of the system and must not be jeopardized.

At the base of the soil nail wall, drains are connected either to a footing drain below the finished grade, or to weepholes that penetrate the finished wall. Weepholes should be located and spaced as shown on the plans, coinciding with the drain locations. A filter fabric is usually placed against weepholes to prevent clogging. If drainage aggregate is used, it must be encapsulated by filter fabric. Footing drains are comprised of perforated pipe embedded in drainage gravel. Pipes should be sloped along the wall alignment as shown on the plans.

8.4.7 Wall Facing

After the geocomposite drain strips are installed, the reinforcing steel is placed and shotcrete is applied to the lines and grades specified. The WWM or reinforcing steel must be installed with the proper dimensions, at the specified locations and with the prescribed overlap length. When required by site conditions (e.g., poor cut face standup time), shotcrete facing may be placed before nails are drilled and installed.

Conventional shotcrete procedures as described in the specifications (see Appendix E) are applicable. During shotcreting, construction equipment that causes excessive ground vibrations should not be operating in the vicinity of the shotcreting operations to reduce shotcrete rebound (i.e., shotcrete slump). The overlying cold joint must be cleaned prior to placement of the overlying lift of shotcrete. Acceptable methods for cleaning the shotcrete joint include washing with a combination of injected water and compressed air, blowing with compressed air, or sand blasting. Care should be taken to avoid eroding the soil cut face below the cold joint.

The most critical factor in ensuring a good quality shotcrete facing is a nozzleman who is experienced in applying (i.e., "gunning") shotcrete. The specifications may require the nozzleman to complete a pre-qualification test panel prior to beginning production work.

Some basic recommended practices for applying shotcrete facings are provided below:

- the nozzle should be held perpendicular to the exposed excavated surface, except when shooting around reinforcing bars;
- optimum nozzle distance from the surface being shot against is: 0.6 to 1.5 m (2 to 5 ft) for wet-mix, 1 to 2 m (3 to 6 ft) for dry-mix;
- placement of shotcrete should start at the bottom;
- voids shall not be allowed to form behind bars, plates, or steel mesh; and
- where sharp edges and accurate lines are required, these should be set out by screen boards, guide wires and/or depth spacers.

Temporary shotcrete facings typically consist of 100-mm (4-in.) thick WWM-reinforced shotcrete, placed directly against the soil, as the excavation proceeds in staged lifts. The steel bearing plate is positioned while the shotcrete is wet. Deviations from perpendicularity are adjusted with tapered washers below the nut. Once the bottom of the excavation is reached, a permanent wall facing is built.

8.5 LOAD TESTING

8.5.1 Introduction

Soil nails are load tested in the field to verify that the nail design loads can be carried without excessive movements and with an adequate factor of safety. Testing is also used to verify the adequacy of the contractor's drilling, installation, and grouting operations prior to and during construction of the soil nail wall. If ground and/or installation procedures change, additional testing may be required to evaluate the influence on soil nail performance. It is typical practice to complete testing in each row of nails prior to excavation and installation of the underlying row. This requirement of completing all testing in the upper row may need to be relaxed, at the direction of the engineer, for very long walls. If test results indicate faulty construction practice or soil nail capacities are less than that required, the contractor should be required to alter nail installation/construction methods. Testing procedures and nail acceptance criteria must be included in the specifications. Load testing can consist of:

Verification or Ultimate Load Tests

Verification or ultimate load tests are conducted to verify the compliance with pullout capacity and bond strengths used in design and resulting from the contractor's installation methods. Verification load tests should be conducted to failure or, as a minimum, to a test load that includes the design bond strength and pullout factor of safety. The number of verification load tests will vary depending on the size of the project and the number of major different ground types in which nails will be installed. As a minimum, two verification tests should be conducted in each soil strata that is encountered. Verification tests are performed on "sacrificial" test nails, which are not incorporated into the permanent work.

Proof Tests

Proof tests are conducted during construction on a specified percentage, typically five (5) percent, of the total production nails installed. Proof tests are intended to verify that the contractor's construction procedure has remained constant and that the nails have not been drilled and grouted in a soil zone not tested by the verification stage testing. Soil nails are proof tested to a load typically equal to 150 percent of the design load.

Creep Tests

Creep tests are performed as part of ultimate, verification, and proof testing. A creep test consists of measuring the movement of the soil nail at a constant load over a specified period of time. This test is performed to ensure that the nail design loads can be safely carried throughout the structure service life.

8.5.2 Equipment for Testing

A center-hole hydraulic jack and hydraulic pump are used to apply a test load to a nail bar. The axis of the jack and the axis of the nail must be aligned to ensure uniform loading. Typically, a jacking frame or reaction block is installed between the shotcrete or excavation face and the jack. The jacking frame should not react directly against the nail grout column during testing. Once the jack is centered and aligned, an alignment load should be applied to the jack to secure the equipment and minimize the slack in the set-up. The alignment load should not be permitted to exceed 10 percent of the maximum test load. Figure 8.1 shows soil nail testing operations.

Movement of the nail head is measured with at least one, and preferably two, dial gauges mounted on a tripod or fixed to a rigid support that is independent of the jacking set-up and wall. The use of two dial gauges provides: (1) an average reading in case the loading is slightly eccentric due to imperfect alignment of the jack and the nail bar, and (2) a backup if one gauge malfunctions. The dial gauges should be aligned within 5 degrees of the axis of the nail, and should be zeroed after the alignment load has been applied. The dial gauges should be capable of measuring to the nearest 0.02 mm (0.001-in.). The dial gauges should be able to accommodate a minimum travel equivalent to the estimated elastic elongation of the test nail at the maximum test load plus 25 mm (1 in.), or at least 50 mm (2 in.).

A hydraulic jack is used to apply load to the nail bar while, a pressure gauge is used to measure the applied load. A center-hole load cell may be added in series with the jack for use during creep tests. For extended load hold periods, load cells are used as a means to monitor a constant applied load

while the hydraulic jack pump is incrementally adjusted. Over extended periods of time, any load loss in the jack will not be reflected with sufficient accuracy using a pressure gauge. Recent calibration data for the jack, pressure gauge, and load cell must be obtained from the contractor prior to testing. Figure 8.2 shows schematically a hydraulic jack typically used in soil nail applications.

The nail bar shall not be stressed to more than 80 percent of its minimum ultimate tensile strength for Grade 525 MPa (Grade 75) steel or more than 90 percent of the minimum yield strength for Grade 420 MPa (Grade 60) steel.

Source Porterfield et al. (1994).

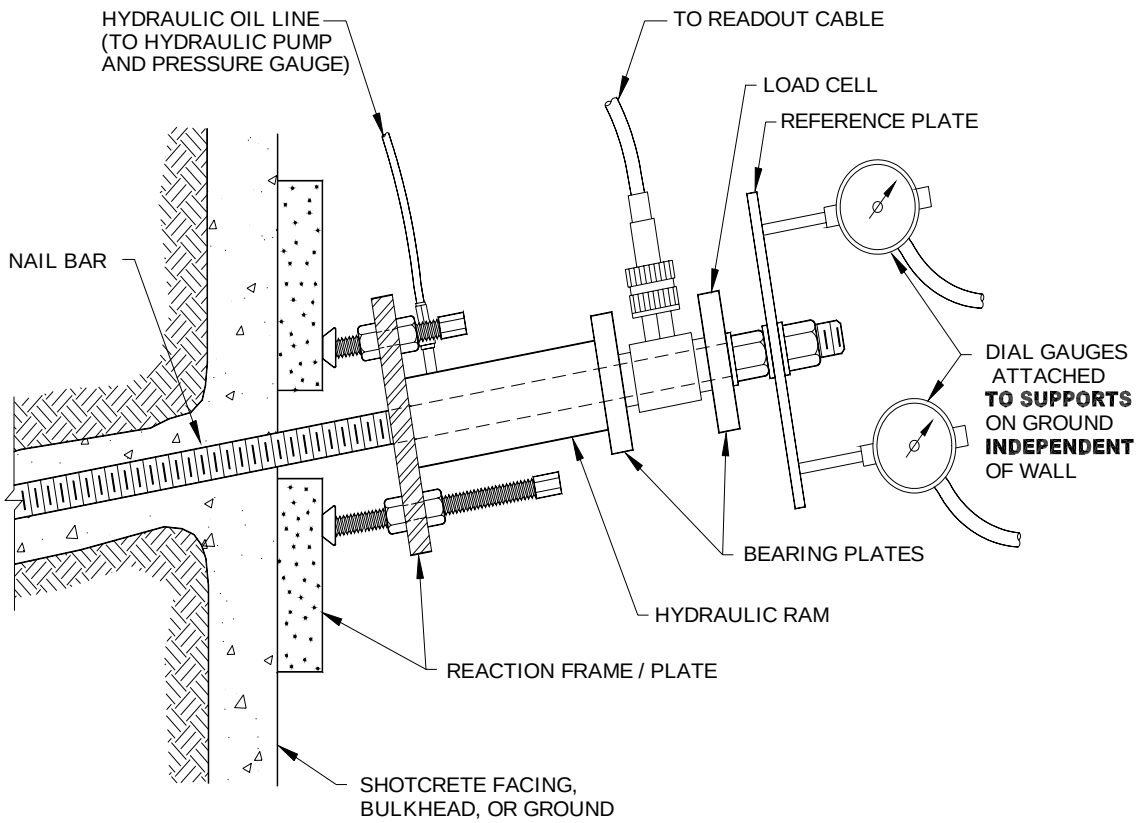
Figure 8.1: Soil Nail Load Testing Setup.

8.5.3 Verification Tests

Verification tests are completed on non-production, "sacrificial" nails prior to construction. In addition, verification testing may be required during production to verify capacities for different in situ conditions encountered during construction and/or different installation methods. Although it would be optimal for verification tests to reach the point of pullout failure, this may not be possible in some cases. Verification tests provide the following information:

- determination of the ultimate bond strength (if carried to pullout failure);
- verification of the design factor of safety; and
- determination of the soil nail load at which excessive creep occurs.

As a minimum, verification test loading must be carried out to a load defined by the pullout factor of safety times the design allowable pullout capacity. If the factor of safety for pullout is 2.0, then the test load must verify 200 percent of the allowable pullout capacity. Test loads in excess of this minimum, and preferably to failure, are recommended as they provide considerably more information and may lead to more economical drilling installation methods. The test acceptance criteria require that:



Source Porterfield et al. (1994).

Figure 8.2: Hydraulic Jack Used for Soil Nail Load Testing.

- no pullout failure occurs at 200 percent of the design load where pullout failure is defined as the load at which attempts to further increase the test load increments simply results in continued pullout movement of the tested nail; and
- the total measured movement (ΔL) at the test load of 200 percent of design load must exceed 80 percent of the theoretical elastic movement of the unbonded length (UL). This criterion is expressed as $\Delta L \geq \Delta L_{min}$, where ΔL_{min} is the minimum acceptable movement defined as:

$$\Delta L_{min} = 0.8 \frac{P\ UL}{E\ A} \qquad \text{(Equation 8.1)}$$

where:

P = maximum applied test load;
UL = unbonded length (i.e., from the back of reference plate to top of the grouted length);
A = cross-sectional area of the nail bar; and
E = Young's modulus of steel [typically 200,000 MPa (29,000 ksi)].

This criterion ensures that load transfer from the soil nail to the soil occurs only in the bonded length and not in the unbonded length. Loading sequences and acceptance criteria are contained in specifications (Appendix E). Figure 8.3 shows a data log sheet that can be used for the load testing of soil nails. Figure 8.4 presents an example of data reduction of soil nail load testing to calculate elastic movement.

8.5.4 Proof Test

A proof test is performed on a specified number (typically up to 5 percent) of the total number of production soil nails installed. This test is a single cycle test in which the load is applied in increments to a maximum test load, usually 150 percent of the design load capacity. Proof tests are used to ascertain that the contractor's construction methods and/or soil conditions have not changed and that the production soil nails can safely withstand design loads without excessive movement or long-term creep over the service life.

Production proof test nails have both bonded and (temporary) unbonded lengths. Specifications (see Appendix E) require that the temporary unbonded length of the test nail must be at least 1 m (3 ft). The maximum bonded length is based on the nail bar grade and size such that the allowable bar tensile force is not exceeded during testing. The typical minimum bonded length is 3 m (10 ft). Production proof test nails shorter than 4 m (12 ft) may be tested with less than the minimum 3-m (10-ft) bond length.

The acceptance criteria require that no pullout failure occurs and that the total movement at the maximum test load of 150 percent of design load must exceed 80 percent of the theoretical elastic movement of the unbonded length. Again, the measured movement must be $\Delta L \geq \Delta L_{min}$, where ΔL_{min} has been defined in Equation 8.1

8.5.5 Creep Test

Creep tests are typically performed as part of a verification or proof test. Creep testing is conducted at a specified, constant test load, with displacements recorded at specified time intervals. The deflection-versus-log-time results are plotted on a semi-log graph, and are compared with the acceptance criteria presented in the construction specification.

Acceptance criteria typically requires that creep movement between the 1- and 10-minute readings, at maximum test load, must be less than 1 mm (0.04 in.), or that the creep movement between the 6- and 60-minute readings must be less than 2 mm (0.08 in.) at maximum test load. The creep criterion is based largely on experience and current practice with ground anchors and has been established to ensure that nail design loads can be safely carried throughout the structure service life. Figure 8.5 presents an example of data reduction of soil nail load testing to calculate the creep movement between the 1- and 10-minute readings.

SOIL NAIL TEST DATA SHEET

Project: SR14–WALL #10 Station: 20+00 Nail No.: R1–5

Project No.: SR14 W10 Length: 21' (TENDON)

Date: 2–23–93 Bonded Length: 15' Hole Dia.: 6 in.

Field Inspector: J. SMITH Unbonded Length: 6'

Type Test: Verification _____ Ultimate: _____ Proof: X

Time	Load			Movement (in.)			Tendon Dia.	Tendon Grade
	Load Increments (%)	Load Increments (Kips)	Pressure Guage (psi)	Dial Gage 1	Dial Gage 2	Avg. Dial Gage	1.0 in.	#8 (GR 60)
							Comments	
10:00 AM	–	–	200	0.000	0.000	0.000	ALIGNMENT LOAD	
10:00	25	7.9	500	0.036	0.038	0.037		
10:02	25	7.9	500	0.038	0.040	0.039		
10:03	50	15.8	1000	0.995	0.996	0.996		
10:05	50	15.8	1000	0.998	0.999	0.999		
10:06	75	23.6	1500	0.165	0.168	0.167		
10:08	75	23.6	1500	0.170	0.172	0.171		
10:09	100	31.5	2000	0.234	0.237	0.236		
10:14	100	31.5	2000	0.240	0.242	0.241		
10:15	130	41.0	2600	0.360	0.361	0.361	Creep test	
10:15:30 (.5)	130	41.0	2600	0.364	0.365	0.365	1 to 10–minute hold	
10:16 (1)	130	41.0	2600	0.367	0.368	0.368	(load cell = 400)	
10:17 (2)	130	41.0	2600	0.369	0.370	0.370	1 to 10–minute	
10:18 (3)	130	41.0	2600	0.370	0.371	0.371	creep movement	
10:20 (5)	130	41.0	2600	0.371	0.372	0.372	= 0.372 – 0.368	
10:21 (6)	130	41.0	2600	0.371	0.372	0.372	= 0.004 in. < 0.04 in.	
10:25 (10)	130	41.0	2600	0.371	0.372	0.372	therefore creep	
							movement passes	
							acceptance criteria	

Source Porterfield et al. (1994).

Figure 8.3: Typical Data Sheet for Soil Nail Load Testing.

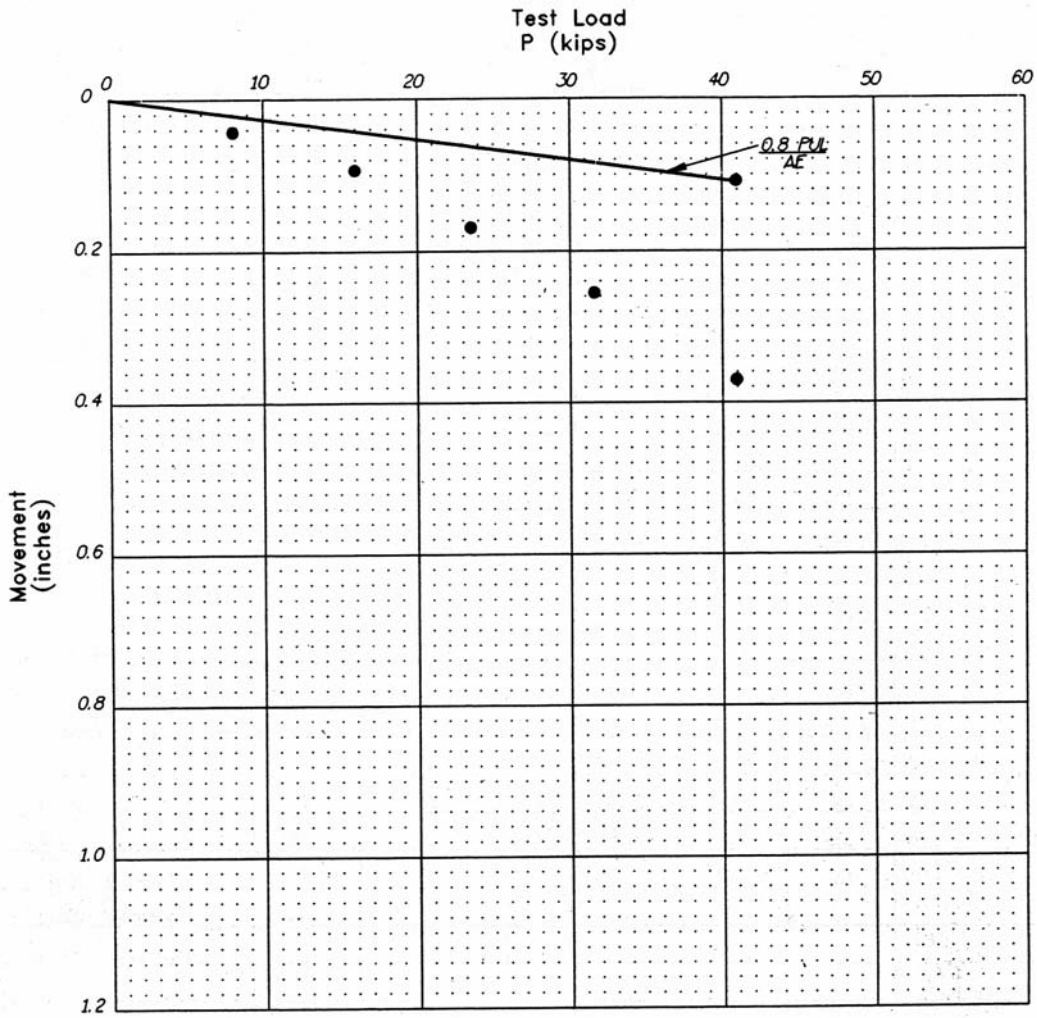
ELASTIC MOVEMENT

Project: SR14–WALL #10 Soil Nail No.: R1–5

Project No.: SR14 W10 Bonded Length: 15'

Date: 2–23–93 Unbonded Length: 6'

Bar Size: #8 (GR 60) Bar Dia.: 1.0 (in.) Cross–sectional Area: 0.79 (sq.in.)



Test Acceptance Criteria:

$$\text{Meas. Movement} > \frac{0.8 \times P \times UL \times 12}{A \times E} \; ?$$

$$\frac{(0.8)(41)(6)(12)}{(0.79)(29{,}000)} = 0.103$$

Meas. movement at max. P of 41 Kips = 0.372 in. > 0.103 in., Therefore Test Nail Passes Elastic Criteria

Where:

A = area of tendon (sq.in.)

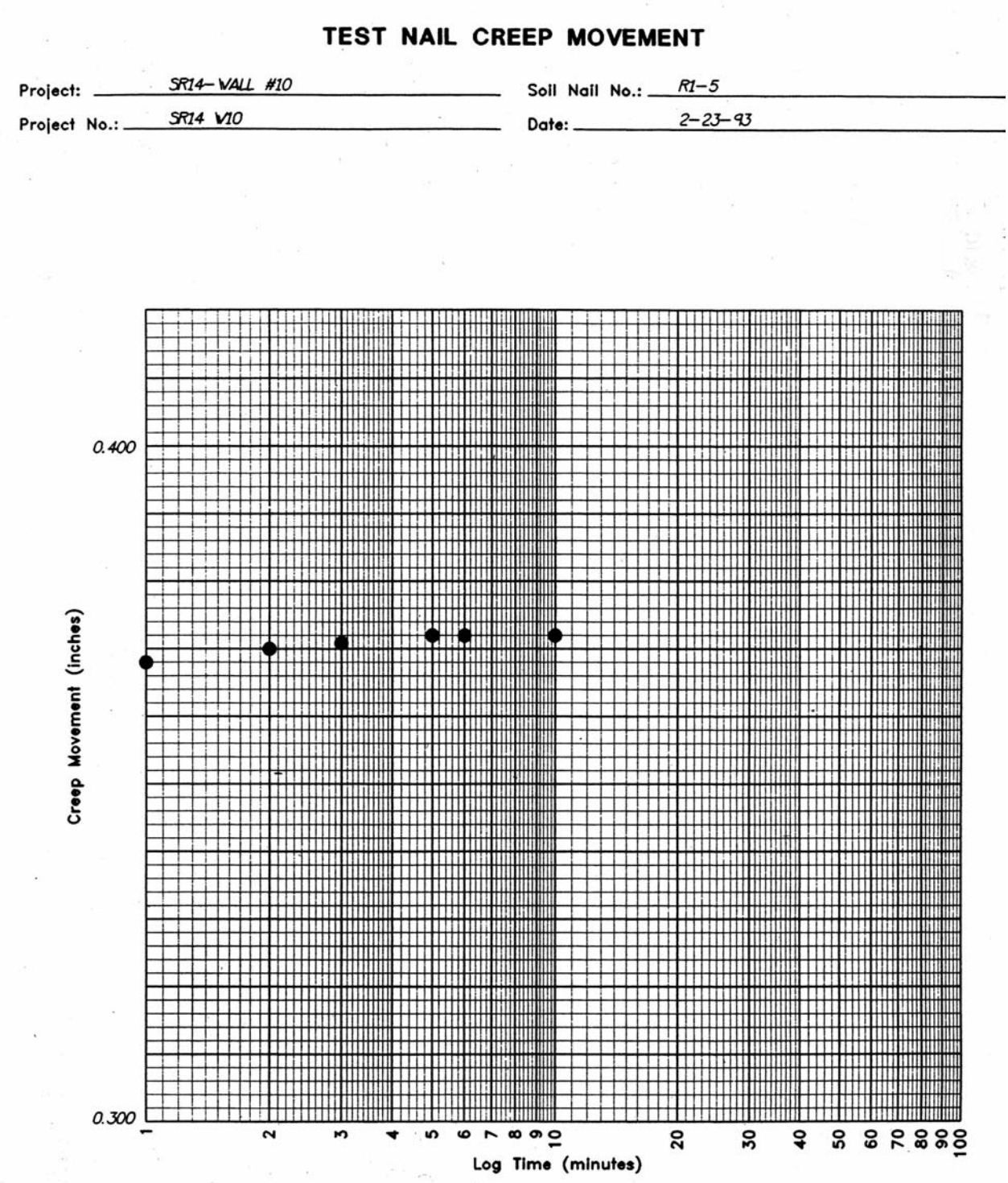
UL = unbonded length (ft.)

P = test load (Kips)

E = 29,000 Ksi

Source Porterfield et al. (1994).

Figure 8.4: Example of Data Reduction from Soil Nail Load Testing.



Source Porterfield et al. (1994).

Figure 8.5: Example of Data Reduction from Soil Nail Creep Testing.

8.6 LONG-TERM MONITORING

8.6.1 Introduction

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance. This section includes details necessary to plan and implement both limited and comprehensive monitoring programs for soil nail structures.

Performance monitoring should be considered too be included for any critical or unusual soil nail wall installation. Examples of critical or unusual installation are walls higher than 10-m (30-ft), walls constructed for road widening projects under existing bridges, and walls with high external surcharge loading. In addition, it is recommended that performance monitoring should be considered when a permanent soil nail wall is constructed for the first time in any State.

Performance monitoring instrumentation for such walls should include inclinometers, top-of- wall survey points, load cells, and strain gauges. Inclinometers and survey points are used to measure wall movements during and after construction. Load cells are installed on selected production nails at the wall face to measure the magnitude of nail head forces. By installing strain gauges in individual nails, the development and distribution of the nail forces may be measured to provide information to improve future designs. Monitoring for a period of at least 2 years after construction is recommended to examine service deformation and stress development in the nails and wall facing as a function of load, time, and environmental changes such as winter freeze-thaw cycles.

8.6.2 Parameters to be Monitored

The most significant measurement of overall performance of the soil nail wall system is the amount of deformation of the wall or slope during and after construction. Inclinometers along the face and at various distances away from the face provide the most comprehensive data on ground deformations.

The following list provides important elements to be considered during the development of a systematic approach to planning soil nail wall performance monitoring programs using geotechnical instrumentation:

- vertical and horizontal movements of the wall face;
- local movements or deterioration of the facing elements;
- drainage of the ground;
- performance of any structure supported by the reinforced ground, such as roadways, bridge abutments or footings, and slopes above the wall;
- loads in the nails, with special attention to the magnitude and location of the maximum load;
- load distribution in the nails due to surcharge loads;
- load change in the nails as a function of time;
- nail loads at the wall face;

- temperature (may cause real changes in other parameters and also affect instrument readings); and
- rainfall (often a cause of real changes in other parameters).

8.6.3 Soil Nail Wall Performance Monitoring Plan

Monitoring during wall construction should be performed to obtain data on the overall wall performance. As a minimum, a performance-monitoring plan should typically include requirements for the following features:

- face horizontal movements using surface markers on the facing and surveying methods, and inclinometer casings installed a short distance [typically 1 m (3 ft)] behind the facing;
- vertical and horizontal movements of the top of wall facing and the ground surface behind the shotcrete facing, using optical surveying methods;
- ground cracks and other signs of disturbance in the ground surface behind the top of wall, through daily visual inspection during construction and, if necessary, installation of crack gauges astride the cracks;
- local movements and or deterioration of the facing using visual inspections and instruments such as crack gauges; and
- drainage behavior of the structure, especially if groundwater is observed during construction; drainage can be monitored visually by observing outflow points or through standpipe piezometers installed behind the facing.

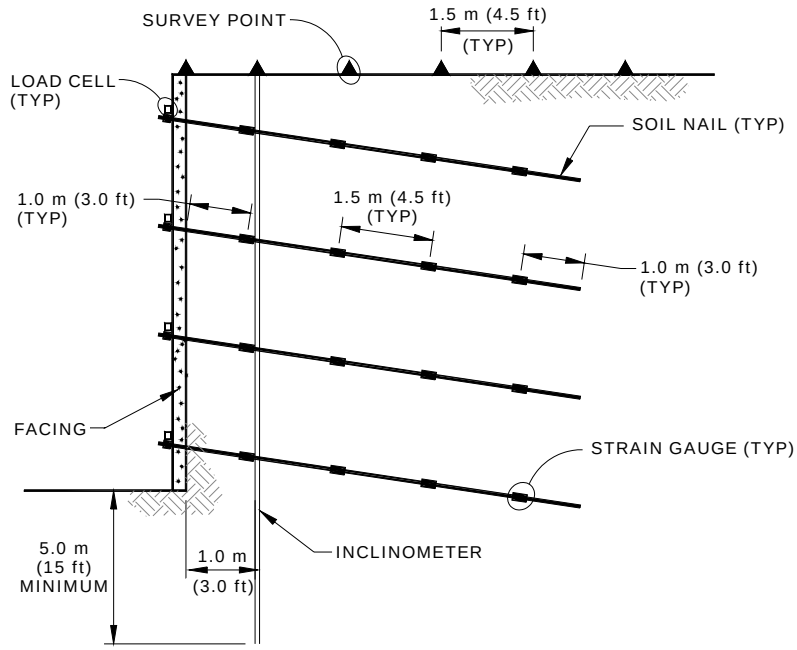
Alternatively, a soil nail wall performance-monitoring plan can be more comprehensive and continued over a longer time period for one or more of the following purposes:

- confirming design stress levels and monitoring safety during construction;
- allowing construction procedures to be modified for safety or economy;
- controlling construction rates; and
- enhancing knowledge of the behavior of soil nail structures to provide a base reference for future designs and possibly improving design procedures and/or reducing costs.

A more comprehensive monitoring plan might include the following:

- strain gauge monitoring along the length of the nail to determine the magnitude and location of the maximum nail load. Ideally, strain gauges are attached to the nail bar in pairs, and are mounted top to bottom at a 1.5-m (4.5-ft) spacing, diametrically opposed to address bending effects. The end of the bar should be inscribed so that the final orientation of the strain gauge can be verified;
- load cells to measure loads at the head of the nail; high quality nail load data near the head of the nail can generally be obtained by load cells rather than by strain gauges attached to the nail; and
- inclinometers to measure horizontal movements of the structure. Inclinometers behind the wall facing should be installed from the ground surface at various horizontal distances up to

one time the wall height. A typical instrumentation layout for a comprehensive monitoring plan is shown in Figure 8.6.



Source Byrne et al. (1998).

Figure 8.6: Typical Instrumentation.

A well-defined, systematic plan should be developed for all monitoring programs, whether limited or comprehensive. The first step is to define the purpose of the measurements. Every instrument on a project should be selected and installed to assist in answering a specific question. If there is no specific question, there should be no instrumentation.

More detailed discussions of appropriate instrumentation schemes and equipment requirements are contained in Chapter 8 of Christopher et al. (1990) and Byrne et al. (1988).

8.6.4 Soil Nail Wall Performance Monitoring Instruments

Soil nail wall performance monitoring instruments should be selected based on the parameters to be measured, the instrument's reliability and simplicity, and the instrument's compatibility with the readout devices specified for the project. A brief discussion of the various types of monitoring instruments typically employed for assessing soil nail wall performance is provided below.

Inclinometers

Inclinometers, preferably installed about 1 m (3 ft) behind the soil nail wall face, provide the most comprehensive data on wall deformations. Inclinometers are a well-established technology and are commercially available from several manufacturers.

Survey Points

Wall face deformation can be measured directly by optical surveying methods or indirectly with electronic distance measuring (EDM) equipment. Also, ground movements behind the soil nail wall

can be assessed by monitoring an array or pattern of ground surface points established behind the wall face and extending for a horizontal distance at least equal to the wall height (e.g. see Figure 8.6). In addition, reflector prisms attached to selected nails allow for electronic deformation measurements of discreet points on the soil nail wall face. The survey system is typically capable of measuring horizontal and vertical displacements to accuracy of 3 mm (0.12 in.) or better.

Strain Gauges

Soil nails instrumented with strain gauges allow assessment of the soil nail load distribution as the excavation progresses and after the completion of the soil nail wall installation. Conventional strain gauges will measure loads carried by both the grout and nail and will thereby depend to some extent on the in-place deformational characteristics of the grout and the interaction between the grout and drillhole wall, both of which are difficult to evaluate.

This approach could eliminate data interpretation problems associated with grout/nail interaction. It has been suggested that some type of mechanical assembly could be installed at each gauge location to break the grout column and ensure that all load is transferred to the nail bar at this point. Because the grout has some tensile strength, it will carry a portion of the total load. Thus, while strain measurements in the grout and nail are readily achievable, conversion of these measurements into nail loads is difficult to achieve with accuracy.

Load Cells at the Nail Head

Load cells installed at the soil nail head are used to provide reliable information on the actual loads that are developed at the facing.

CHAPTER 9 REFERENCES

AASHTO (1986). "Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part I, Specifications, 14th ed.," American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1988). "Manual on Subsurface Investigations," American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1992). "Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, Tests." 15th edition, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1994). "LRFD Bridge Design Specifications," 1st edition, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (1996). "Standard Specifications for Highway Bridges," 16th edition, American Association of State Highway and Transportation Officials, Washington, D.C.

ACI (1994). "Guide For The Evaluation of Shotcrete," Publication No. ACI 506.4R-94. American Concrete Institute, Farmington Hills, Michigan.

ACI (1995a). "Guide to Shotcrete," Publication No. ACI 506R-90, American Concrete Institute, Farmington Hills, Michigan.

ACI (1995b). "Specification for Materials, Proportioning, and Application of Shotcrete," Publication No. ACI 506.2-95, American Concrete Institute, Farmington Hills, Michigan.

ACI (1998). "Code requirements for nuclear safety-related concrete structures (ACI 349-97) and commentary," Publication No. ACI 349R-97, ACI Committee 349, American Concrete Institute, Farmington Hills, Michigan, p. 129.

Arman, A., Samtani, N., Castelli, R., and Munfakh, G. (1997). "Subsurface Investigations Participants Manual." Report FHWA-HI-97-021, Federal Highway Administration, Washington, D.C.

ASTM (2002). "Annual Book of ASTM Standards," American Society for Testing and Materials, Pennsylvania.

Banerjee, S., Finney, A., Wentworth, T., and Bahiradhan, M. (1988). "Evaluation of Design Methodologies for Soil-Nailed Walls," Vol. 1: "General," Vol. 2: "Distribution of Axial Forces in Soil Nails Based on Interpretation of Measured Strains; Evaluation of Design Methodologies for Soil "Nailed Walls," Vol. 3: "An Evaluation of Soil Nailing Analysis Packages." Department of Civil Engineering, University of Washington, Seattle, Washington.

Bang, S. (1991). “Contribution to Summary of Design Methods Comparison for Nailed Retaining Walls,” Demonstration Project 82, Federal Highway Administration, Washington, D.C.

Bieniawski, Z.T. (1989). “Engineering rock mass classifications,” Wiley, New York.

Blondeau, F., Christiansen, M., Guilloux, A., and Schlosser, F. (1984). “TALREN: Méthode de Calcul des Ouvrages en Terre Renforcée,” In Proceedings of the International Conference of In Situ Soil and Rock Reinforcement, Paris, France, pp. 219-224.

Briaud, J.-L. (1989). “The Pressuremeter Test for Highway Application,” Report FHWA-IP-89-008, Federal Highway Administration, Washington, D.C.

Briaud, J.-L. (1992). “The Cone Penetrometer Test,” Report FHWA-SA-91-043, Federal Highway Administration, Washington, D.C.

Briaud, J.-L. and Lim, Y. (1997). “Soil-Nailed Wall under Piled Bridge Abutment: Simulation and Guidelines,” Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 123, No. 11, pp. 1043-1050.

Briaud, J.-L., and J. Miran (1992). “The Flat Dilatometer Test,” Report FHWA-SA-91-044, Federal Highway Administration, Washington, D.C.

Brown, E.T. (Ed.), (1981). “Rock Characterization, Testing and Monitoring, ISRM Suggested Methods,” Pergamon, Oxford, UK.

Bruce, D.A. and R.A. Jewell (1987). “Soil Nailing: Application and Practice – 2 Parts,” Ground Engineering, Vol. 20, No. 1, Jan., pp. 21-28.

Byrne, R.J., Cotton, D., Porterfield, J., Wolschlag, C., and Ueblacker, G. (1998). “Manual for Design and Construction Monitoring of Soil Nail Walls,” Report FHWA-SA-96-69R, Federal Highway Administration, Washington, D.C.

CALTRANS (1991). “A User’s Manual for the SNAIL Program, Version 2.02 – Updated PC Version,” California Department of Transportation, Division of New Technology, Material and Research, Office of Geotechnical Engineering, Sacramento, California.

Caquot, A.I., and Kerisel, J. (1948). “Tables for the calculation of passive pressure, active pressure and bearing capacity of foundations.” Libraire du Bureau des Longitudes, de L’ecole Polytechnique, Gauthier-villars, Paris, France, 120p.

Casagrande, A. (1931). Discussion on “A New Theory of Frost Heaving,” by Benkelman and Olmstead, Proceedings Highway Research Board 11, Part 1, pp. 168-172.

Chandler, R.J. (1988). “The In-Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane.” Vane Shear Strength Testing in Soils: Field and Laboratory Studies, ASTM Publication No. STP 1014, pp. 13-44.

Chassie, R.G. (1994). “FHWA Ground Nailing Demonstration Project, Guideline Manual and Workshop,” Federal Highway Administration, Washington, D.C.

Cheney, R.S. (1988). “Permanent Ground Anchors,” Report FHWA-DP-68-1R, Federal Highway Administration, Washington, D.C.

Christopher, B.R., Gill, S.A., Juran, I., and Mitchell, J.K., (1990). “Reinforced Soil Structures, Volume 1, Design and Construction Guidelines,” Report No. FHWA-RD-89/043, Federal Highway Administration, Washington, D.C., November.

CLOUTERRE (1993). “Recommandations CLOUTERRE 1991 - Soil Nailing Recommendations 1991,” English Translation, Presses de l’Ecole Nationale des Ponts et Chaussées, Paris, France.

Deere, D.U. and Deere, D.W. (1988). “The rock quality designation (RQD) index in practice,” In Rock classification systems for engineering purposes, (ed. L. Kirkaldie), ASTM Special Publication 984, 91-101. American Society of Testing Materials, Philadelphia, Pennsylvania.

Deere, D.U., Hendron, A.J., Patton, F.D. and Cording, E.J. (1967). “Design of surface and near surface construction in rock,” In Failure and breakage of rock, Proc. 8th U.S. Symposium Rock Mechanics, (ed. C. Fairhurst), New York, Society of Mining Engineers, American Institute of Mining, Metallurgical and Petroleum Engineers, pp. 237-302.

Ebeling, R.M. and Morrison, E.E., Jr. (1992). “The Seismic Design of Waterfront Retaining Structures,” Technical Report ITL-92-11, U.S. Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

Elias, V. (1995). “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Walls and Reinforced Soil Slopes.” Publication No. FHWA-SA-96-072, Federal Highway Administration, Washington, D.C.

Elias, V. (1995). “Durability/Corrosion of Soil Reinforced Structures,” Report FHWA RD 89-186, Federal Highway Administration, Washington, D.C.

Elias, V. (2000). “Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,” Federal Highway Administration, Publication FHWA-NHI-00-044, Washington, D.C.

Elias, V. and Christopher, B.R. (1997). “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines,” Federal Highway Administration, Publication FHWA-SA- 96-071, Washington, D.C., 371 pp.

Elias, V. and Juran, I. (1991). "Soil Nailing for Stabilization of Highway Slopes and Excavations," Publication FHWA-RD-89-198, Federal Highway Administration, Washington D.C.

Elias, V., Christopher, B.R., and Berg, R. (2001). "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines," Federal Highway Administration, Publication FHWA-NHI-00-043, Washington, D.C., 394 pp.

Elias, V., Welsh, J., Warren, J., Lukas, R. (2001). "Ground Improvement Technical Summaries," Federal Highway Administration, Demonstration Project 116, Publication FHWA-SA-98-086R, Washington, D.C.

Felio, G.Y., Vucetic, M., Hudson, M., Barar, O., and Chapman, R. (1990). "Performance of Soil Nailed Walls during the October 17, 1989 Loma Prieta Earthquake," *In* Proceedings, 43rd Canadian Geotechnical Conference, Quebec, pp. 165-173.

Felio, G.Y.; et al. (1990). "Performance of soil nailed walls during the October 17, 1989 Loma Prieta earthquake," Canadian Geotechnical Conference, Oct. 10-12, 1990, Quebec, Canada, pp. 165-173.

FHWA (1993a). "FHWA International Scanning Tour for Geotechnology, September–October 1992, Soil Nailing Summary Report," Publication FHWA-PL-93-020, Federal Highway Administration, Washington, D.C.

FHWA (1993b). "French National Research Project Clouterre, 1991-Recomandations Clouterre 1991," (English Translation) Soil Nailing Recommendations, Publication FHWA-SA-93-026, Federal Highway Administration, Washington, D.C.

Gässler, G. and Gudehus, G., (1981). "Soil Nailing-Some Aspects of a New Technique," *In* Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3., Session 12, Stockholm, Sweden, pp. 665–670.

Golder (1993). "GOLDNAIL Soil Nailing Design Program," Golder Associates, Seattle, Washington.

Hoek E. and Brown E.T. (1980). "Underground Excavations in Rock," Institution of Mining and Metallurgy, London, England.

Hoek, E. (1983). "Strength of jointed rock masses," 23rd. Rankine Lecture, Géotechnique, Vol. 33, No. 3, pp. 187-223.

Hoek, E. (1994). "Strength of rock and rock masses," ISRM News Journal, Vol. 2, No. 2, pp. 4-16.

Hynes, M.E. and Franklin, A.G. (1984). “Rationalizing the Seismic Coefficient Method.” Miscellaneous Paper GL-84-13, U.S. Army Engineering Waterways Experiment Station, Vicksburg, Mississippi, 34 p.

IBC (2000). “International Building Code,” International Code Council, Inc., Building Officials and Code Administrators International, Inc., Country Club Hills, IL; International Conference of Building Officials, Whittier, California; and Southern Building Code Congress International, Inc., Birmingham, Alabama.

Jamiolkowski, M.; Ladd, C.C.; Germaine, J.T, and Lancellotta, R. (1985). “New Developments in Field and Laboratory Testing of Soils,” Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, August 12-16, Vol. 1, pp. 57-153.

Juran, I. and Elias, V. (1991). “Ground Anchors and Soil Nails in Retaining Structures,” in Foundation Engineering Handbook, 2nd Edition, (Ed.: Fang), pp. 868-905.

Juran, I., Baudrand, G., Farrag, K., and Elias, V. (1990). “Kinematical limit analysis for design of nailed structures,” Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 116, No. 1, pp. 54-72.

Kavazanjian, E. Jr., Matasović, N., Hadj-Hamou, T., and Sabatini, P.J. (1997). “Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I, Design Principles,” Geotechnical Engineering Circular No. 3, Report FHWA-SA-97-076, Federal Highway Administration, Washington, D.C.

Kingsbury, D.W., Sandford, T.C., and Humphrey, D.N. (2002). “Soil Nail Forces Caused by Frost,” In Proceedings of the Transportation Research Board, Annual Meeting, Washington, D.C.

Kramer, S.L. (1996). “Geotechnical Earthquake Engineering,” Prentice Hall, Upper Saddle River, New Jersey.

Kulhaway, F.H., and Mayne, F. (1990). “Manual on Estimating Soil Properties for Foundation Design,” Electric Power Research Institute, Palo Alto, California.

Lam, I.P. and Martin, G.R. (1986). “Seismic Design of Highway Bridge Foundations - Vol. II, Design Procedures and Guidelines.” Report FHWA-RD-86-102, Federal Highway Administration, McLean, Virginia.

Lang, T.A., (1972). “Rock Reinforcement,” Bulletin of the Association of Engineering Geologists, Vol. IX, No. 3, pp 215-239.

Long, J.H., Chow, E., Cording, E.T., and Sieczkowski, W.J. (1990). “Stability Analysis for Soil Nailed Walls,” Geotechnical Special Publication No. 25, American Society of Civil Engineers, pp. 676-691.

Mesri, G. (1988). “A Reevaluation of $S_{u(mob)}=0.22\sigma_p$’ using Laboratory Shear Tests.” Canadian Geotechnical Journal, Vol. 26, pp. 162-164.

Meyerhof, G.G. (1956). “Penetration Tests and Bearing Capacity of Cohesionless Soils,” Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 82, No. SM1, pp 1-19.

Mitchell, J.K. (1993). “Fundamentals of Soil Behavior,” Second Edition, Wiley, New York.

Mononobe, N. (1929). “Earthquake-Proof Construction of Masonry Dams.” In Proceedings of the World Engineering Conference, Vol. 9.

NEHRP (National Earthquake Hazard Reduction Program) (1997). “Guidelines for the seismic rehabilitation of buildings,” Publication FEMA-273, National Earthquake Hazard Reduction Program, Federal Emergency Management Agency, Washington, D.C.

Newmark, N. (1965). “Effects of Earthquakes on Dams and Embankments,” Geotéchnique, Vol. 15, No. 2, pp. 139-160.

Nicholson, P.J. (1986). “In Situ Earth Reinforcement at Cumberland Gap, U.S. 25E,” In American Society of Civil Engineers and Pennsylvania Department of Transportation Joint Conference, April, Harrisburg, Pennsylvania, 21 pp.

Okabe, S. (1926). “General Theory of Earth Pressure.” Journal of Japan Society of Civil Engineering, Vol. 12, No. 1.

Oral, T. and Sheahan, T.C. (1998). “The Use of Soil Nails in Soft Clays,” Geotechnical Special Publication No. 83, Design and Construction of Earth Retaining Systems (Finno et al., eds.), American Society of Civil Engineers, pp. 26–40.

Peck, R., Hanson, W.E., and Thornburn, T.H. (1974). “Foundation Engineering,” Wiley.

Peck, R.B. (1969). “Deep Excavations and Tunneling in Soft Ground, State of Art Report.” *In* Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Mexico, pp. 225-290.

Plumelle, C. Schlosser, F., Oclage, P., and Knochenmus, G. (1990). “French National Research Project on Soil Nailing: CLOUTERRE,” Geotechnical Special Publication No. 25, American Society of Civil Engineers, pp. 660-675.

Pockoski, M. and Duncan, J.M. (2000). “Comparison of Computer Programs for Analysis of Reinforced Slopes,” Center for Geotechnical Practice and Research, Via Department of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

Porterfield, J. A., Cotton, D.M., and Byrne, R.J. (1994). "Soil Nailing Field Inspectors Manual, Project Demonstration 103," Publication No. FHWA-SA-93-068, Federal Highway Administration, Washington, D.C.

PTI (1996). "Recommendations for Prestressed Rock and Soil Anchors," 3rd ed. Post-Tensioning Institute, Phoenix, Arizona.

Rabcewicz, L.V. (1964a). "The New Austrian Tunneling Method," Part 1, Water Power, London, Vol. 16, November, pp 453-457.

Rabcewicz, L.V. (1964b). "The New Austrian Tunneling Method," Part 2, Water Power, London, Vol. 16, December, pp 511-515.

Rabcewicz, L.V. (1965). "The New Austrian Tunneling Method," Part 3, Water Power, London, Vol. 17, January, pp 19-24.

Rabejac, S. and Toudic, P. (1974). "Construction d'un mur de soutènement entre Versailles-Chantiers et Versailles-Matelots," Trans.: Construction of a retaining wall between Versailles-Chantiers and Versailles-Matelots, Revue générale des chemins de fer, Vol. 93, pp. 232-237.

Sabatini, P.J. et al. (1999). "Ground Anchors and Anchored Systems," Geotechnical Engineering Circular No. 4, Publication FHWA-IF-99-015, Federal Highway Administration, Washington, D.C.

Sabatini, P.J. et al. (2002). "Soil and Rock Properties," Geotechnical Engineering Circular No. 5, Publication FHWA No.: TBD, Federal Highway Administration, Washington D.C.

Sabatini, P.J., Elias, V., Schmertmann, G.R., Bonaparte, R. (1997). "Geotechnical Engineering Circular No. 2, Earth Retaining Systems." Publication FHWA-SA-96-038, Federal Highway Administration, Washington, D.C.

Schlosser, F. (1982). "Behavior and Design of Soil Nailing," in Proceedings of the Symposium of Recent Developments in Ground Improvement Techniques, Asian Institute of Technology, Bangkok, Thailand, pp. 319-413.

Schlosser, F. (1983). "Analogies et différences dans le comportement et le calcul des ouvrages de soutènement en Terre Armée et par clouage des sols," (Trad.: Similarities and differences in the behavior and design of retaining structures of reinforced earth and soil nailing). Annales de L'Institut Technique de Batiment et des Travaux Publics, No. 418, Series: Sols et Fondations, Paris, France.

Schlosser, F., and Unterreiner, P. (1991). "Soil Nailing in France: Research and Practice," In Proceedings of the Transportation Research Board, Annual Meeting, Washington, D.C.

Schmertmann, J.H. (1975). “Measurement of In-Situ Strength,” In Proceedings of the Conference on In Situ Measurement of Soil Properties, American Society of Civil Engineers, pp 55-138.

Seed, H.B. and Whitman, R.V. (1970). “Design of earth retaining structures for dynamic loads,” State of the Art Papers presented at 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures, American Society of Civil Engineers.

Seible, F. (1996). “Structural Response Assessment of Soil Nail Wall Facings,” Report No. SSRP-96/01, Division of Structural Engineering, University of California, San Diego.

Sheahan, T.C., T. Oral, and C.L. Ho (2002). “A Simplified Trial Wedge Method for Soil Nailed Wall Analysis,” Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 128, No.:TBD.

Shen, C.K., Bang, S., Romstad, J.M., Kulchin, L., and Denatale, J.S. (1981b). “Field measurements of an earth support system,” Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 107, No. GT-12.

Shen, C.K., Herrmann, L.R., Romstand, K.M., Bang, S., Kim, Y.S., and Denatale, J.S. (1981a). “In situ Earth Reinforcement Lateral Support System,” Report No. 81-03, Department of Civil Engineering, University of California, Davis, California.

Shields, D.R., Schnabel, H., Weatherby, A.M. (1978). “Load transfer in pressure injected anchors,” Journal of the Geotechnical Engineering Division, Vol. 104, GT 9, September, pp. 1183-1196.

Skempton, A.W. (1957). Discussion of “Planning and Design of New Hong Kong Airport,” Institution of Civil Engineers, Vol. 7, June, pp. 305-307.

Stocker, M.F., Korber, G.W., Gässler, G., and Gudehus, G. (1979). “Soil Nailing,” in International Conference on Soil Reinforcement I, Paris, France, Vol. 2, pp. 469–474.

Tatsuoka,F., Koseki, J., and Tateyama, M. (1997). “Performance of reinforced soil structures during the 1995 Hyogo-ken Nanbu Earthquake,” Special Lecture, in Proceedings of the International Symposium on Earth Reinforcement (IS Kyushu ‘96), Balkema, Vol. 2, pp. 973-1008.

Terzaghi, K., Peck, R., and Mesri, G. (1996). “Soil Mechanics in Engineering Practice,” 3rd Ed., J. Wiley & Sons, Inc.

Thompson, S.R. and I.R. Miller (1990). “Design, Construction and Performance of a Soil Nailed Wall in Seattle, Washington,” Design and Performance of Earth Retaining Structures (P.C. Lambe and L.A. Hansen, Eds.), Geotechnical Special Publication No. 25, American Society of Civil Engineers, New York, pp. 629-643.

Tufenkjian, M.R. (2002). “Performance of Soil Nailed Retaining Structures During the 2001 Nisqually, Earthquake,” Proceedings of the 7th U.S. National Conference on Earthquake Engineering, July 21-25, Boston.

Tufenkjian, M.R. and Vucetic, M. (2000). “Dynamic failure mechanism of soil-nailed excavation models in centrifuge,” Journal of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 126, No. 3, pp. 227-235.

U.S. Department of the Navy (1982). “Foundations and Earth Structures,” NAVFAC DM-7.2, Naval Facilities Engineering Command, Alexandria, Virginia.

Vucetic, M., Iskandar, V.E., Doroudian, M., and Luccioni, L. (1996). “Dynamic Failure of Soil-Nailed Excavation in Centrifuge,” Earth Reinforcement, Balkema, pp. 829-834.

Vucetic, M., Tufenkjian, M., and Doroudian, M. (1993). “Dynamic Centrifuge Testing of Soil-Nailed Excavations,” ASTM Geotechnical Testing Journal, Vol. 16, No. 2 pp. 172-187.

Weatherby, D.E. (1982). “Tiebacks,” Report No. FHWA-RD-82-047, Federal Highway Administration, Washington D.C.

Williamson, D.A. (1984). “Unified Rock Classification System,” Bulletin of the Association of Engineering Geologists, Vol. 21, No. 3, p. 345-354.

Wire Reinforcement Institute (2001). “Manual of Standard Practice-Structural Welded Wire Reinforcement,” 6th Edition, Hartford, Connecticut.

Wright, S.G., and Duncan, J.M. (1992). “Limit Equilibrium Stability for Soil-Reinforced Slopes,” Transportation Research Record 1330, National Research Council, Washington, D.C., pp. 40–46.

Wroth, C.P. and Houlsby, G.T. (1985). “Soil Mechanics – Property Characterization and Analysis Procedures,” Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, August 12-16, Vol. 1, pp. 1-55.

Zarrabi-Kashani, K. (1979). “Sliding of gravity retaining wall during earthquakes considering vertical accelerations and changing inclination of failure surface,” S.M. Thesis, Dept. Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts.

APPENDIX A

REINFORCEMENT CHARACTERISTICS

TABLE A.1
THREADED BAR PROPERTIES [ASTM A615, GRADES 420 and 525 MPa (60 and 75 ksi)]

Nominal Bar Designation		Cross-Sectional Area		Nominal Unit Weight		Max. Diameter w/Threads		ASTM Grade	Yield Strength		Max. Axial Load	
English	mm	in.^{2}	mm^2	lbs/ft	kg/m	in.	mm	English	ksi	MPa	kips	kN
#6	19	0.44	284	1.50	2.24	0.86	21.8	60	60	414	26.4	118
								75	75	517	33.0	147
#7	22	0.60	387	2.04	3.04	0.99	25.1	60	60	414	36.0	160
								75	75	517	45.0	200
#8	25	0.79	510	2.67	3.98	1.12	28.4	60	60	414	47.4	211
								75	75	517	59.3	264
#9	29	1.00	645	3.40	5.06	1.26	32.0	60	60	414	60.0	267
								75	75	517	75.0	334
#10	32	1.27	819	4.30	6.41	1.43	36.3	60	60	414	76.2	339
								75	75	517	95.3	424
#11	36	1.56	1,006	5.31	7.91	1.61	40.9	60	60	414	93.6	417
								75	75	517	117.0	520
#14	43	2.25	1,452	7.65	11.39	1.86	47.2	60	60	414	135.0	601
								75	75	517	168.8	751

Source: Byrne et al. (1998).

TABLE A.2
WELDED WIRE MESH DIMENSIONS
METRIC AND ENGLISH UNITS

Mesh Designation [(1), (2)]		Wire Cross-Sectional Area per Unit Length[(3)]		Weight per Unit Area	
Metric	**English**	**Metric**	**English**	**Metric**	**English**
(mm x mm –mm^2 x mm^2)	**(in. x in.-in^2/100 x in.^{2} /100)**	**(mm^2/m)**	**(in.^{2}/ft)**	**(kg/m^2)**	**(lbs/ft^2)**
102x102 - MW9xMW9	4x4 - W1.4xW1.4	88.9	0.042	1.51	3.1
102x102 - MW13xMW13	4x4 - W2.0xW2.0	127.0	0.060	2.15	4.4
102x102 - MW19xMW19	4x4 - W2.9xW2.9	184.2	0.087	3.03	6.2
102x102 - MW26xMW26	4x4 - W4.0xW4.0	254.0	0.120	4.30	8.8
152x152 - MW9xMW9	6x6 - W1.4xW1.4	59.3	0.028	1.03	2.1
152x152 - MW13xMW13	6x6 - W2.0xW2.0	84.7	0.040	1.46	3.0
152x152 - MW19xMW19	6x6 - W2.9xW2.9	122.8	0.058	2.05	4.2
152x152 - MW26xMW26	6x6 - W4.0xW4.0	169.4	0.080	2.83	5.8

Source: WRI (2001).

Notes:
(1) The first two numbers indicate the mesh opening size, whereas the second pair of numbers following the prefixes indicates the wire cross-sectional area.
(2) Prefix M indicates metric units. Prefix W indicates plain wire. If wires are pre-deformed, the prefix D is used instead of W.
(3) This value is obtained by dividing the wire cross-sectional area by the mesh opening size.

TABLE A.3
REINFORCING BAR DIMENSIONS (ENGLISH AND METRIC)

Bar Designation		Nominal Diameter		Nominal Area	
English	Metric	in.	mm	in.^{2}	mm^2
3	10	0.375	9.6	0.11	71
4	13	0.500	12.7	0.20	129
5	16	0.625	15.9	0.31	199
6	19	0.750	19.1	0.44	284
7	22	0.875	22.2	0.60	387
8	25	1.000	25.4	0.79	510
9	29	1.128	28.7	1.00	645
10	32	1.270	32.3	1.27	819
11	36	1.410	35.8	1.56	1,006
14	43	1.693	43.0	2.25	1,452
18	57	2.257	57.3	4.00	2,581

Source: Byrne et al. (1998).

TABLE A.4: SELF-DRILLING HOLLOW CORE BAR PROPERTIES (MAI BAR TYPE)

Bar Description	Nominal Outer Diameter		Cross-Sectional Area		Yield Load		Ultimate Load		Weight	
	mm	in.	mm^2	$in.^2$	kN	kips	kN	kips	kg/m	lbs/ft
R25N	25	1	303	0.47	151	34	200	45	2.61	1.75
R32N	32	1 ¼	432	0.67	231	52	280	63	3.61	2.42
R32S	32	1 ¼	503	0.78	280	63	360	81	4.21	2.82
R38N	38	1 ½	748	1.16	401	90	498	112	5.97	4
R51L	51	2	903	1.4	449	101	498	112	7.47	5
R51N	51	2	1052	1.63	632	142	801	180	8.06	5.4

Source: http://www.dywidag-systems.com

TABLE A.5: PROPERTIES OF SELF-DRILLING HOLLOW CORE BAR PROPERTIES (CTS/TITAN BAR TYPE)

Bar Description	Effective Outer Diameter		Cross-Sectional Area		Yield Load		Ultimate Load		Weight	
	mm	in.	mm^2	$in.^2$	kN	kips	kN	kips	kg/m	lbs/ft
30/16	26	1.00	382	0.59	180	40.5	220	49.5	3.01	2.02
32/20	28	1 ⅛	445	0.69	210	47.2	260	58.0	3.42	2.30
30/11	26.2	1 1/16	446	0.69	280	63.0	338	76.0	3.50	2.35
40/20	36	1.42	644	1.00	430	96.7	510	114.7	5.35	3.60
40/16	36	1.42	879	1.36	528	118.7	660	148.4	6.90	4.64
52/26	48.8	1.94	1337	2.08	730	160.8	929	209.0	10.62	7.14

Source: http:/www.contech.com

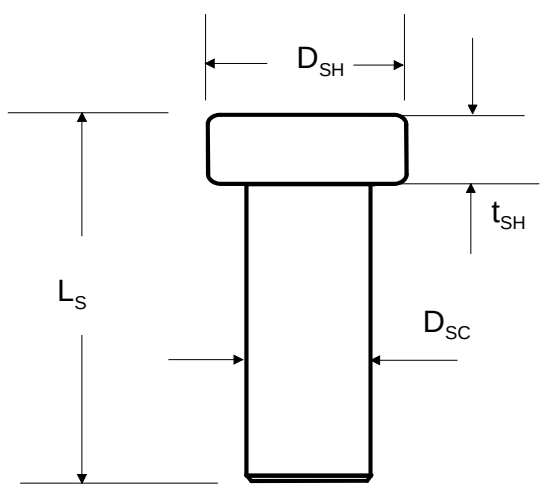
TABLE A.6
HEADED-STUD DIMENSIONS

Headed-Stud Size	Nominal Length L_s		Head Diameter D_H		Shaft Diameter D_S		Head Thickness t_H		Head Area/ Shaft Area	Head Thickness/ (Head Diameter-Shaft Diameter)
	mm	in.	mm	in.	mm	in.	in.	mm	in.	
$^1/_4$ x $4^1/_8$	105	4.125	12.7	0.5	6.4	0.25	4.7	0.19	4.0	0.75
$^3/_8$ x $4^1/_8$	105	4.125	19.1	0.75	9.7	0.38	7.1	0.28	4.0	0.75
$^3/_8$ x $6^1/_8$	156	6.125	19.1	0.75	9.7	0.38	7.1	0.28	4.0	0.75
$^1/_2$ x $4^1/_8$	105	4.125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
$^1/_2$ x $5^5/_{16}$	135	5.3125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
$^1/_2$ x $6^1/_8$	156	6.125	25.4	1	12.7	0.5	7.9	0.31	4.0	0.62
$^5/_8$ x $6^9/_{16}$	162	7.875	31.8	1.3	15.9	0.625	7.9	0.31	4.0	0.50
$^3/_4$ x $3^{11}/_{16}$	89	15.5	31.8	1.3	19.1	0.750	9.5	0.38	2.8	0.75
$^3/_4$ x $4^3/_{16}$	106	4.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
$^3/_4$ x $5^3/_{16}$	132	5.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
$^3/_4$ x $6^3/_{16}$	157	6.1875	31.8	1.25	19.1	0.75	9.5	0.38	2.8	0.75
$^7/_8$ x $4^3/_{16}$	102	4	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75
$^7/_8$ x $5^3/_{16}$	127	5	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75
$^7/_8$ x $6^3/_{16}$	152	6	34.9	1.4	22.2	0.875	9.5	0.38	2.5	0.75

Source: Byrne et al. (1998).

Nominal length indicated is before welding.

- For $\mathbf{D_S} \leq 1/2$”, L_S is approximately 1/8” shorter after welding.
- For $\mathbf{D_S} > 5/8$”, L_S is approximately 3/16” shorter after welding.



APPENDIX B

CHARTS FOR PRELIMINARY DESIGN

A series of design charts were developed as a design aid to provide preliminary nail length and maximum design tensile forces. The charts were developed using the computer program SNAIL (see Section 5 and Appendix F). These charts are only strictly applicable for the conditions they were developed for. The charts should only be used to obtain preliminary design values and should not be used in lieu of comprehensive analyses.

The charts were developed for a range of face batter, α; backslope, β; effective friction angle, ϕ'; and normalized bond strength, μ. The normalized bond strength is defined as:

$$\mu = \frac{q_u \, D_{DH}}{FS_b \gamma \, S_H \, S_V}$$

where: q_U is the ultimate bond strength (presented in Chapters 3 and 5); D_{DH} is the drillhole effective diameter; FS_P is the factor of safety against pullout (see Table 5.3), γ is the total unit weight of the soil behind the wall; and S_H and S_V are the nail horizontal and vertical spacing, respectively. Table B.1 presents the geometric and material conditions used for the development of the design charts.

Two types of charts were created. The first type of chart (Figures B.1a through B.6a) provides the necessary normalized nail length, L/H, required to achieve a global safety factor of FS_G=1.35, where L is the necessary nail length and H is the wall height. L/H varies as a function of the normalized pullout resistance. The values L/H were obtained for the most critical failure surface for the selected geometry and material properties. A total of six charts were created, one for each combination of α and β values shown in Table B.1. For intermediate values of α and β values, it is acceptable to interpolate between charts.

TABLE B.1
VARIABLE PARAMETERS

Parameter	Units	Values
Face Batter	degrees	0, 10
Backslope	degrees	0, 10, 20, 30
Effective Friction Angle	degrees	27, 31, 35, 39
Ultimate Bond Strength	KPa	52, 104, 172, 276, 689
	psi	8,15,25,40,100

The second type of charts (Figures B.1b through B.6b) provides the normalized maximum design nail force, $t_{max\text{-}s}$, as a function of μ. The value $t_{max\text{-}s}$ is defined as:

$$t_{max-s} = \frac{T_{max-s}}{S_H \; S_V}$$

where T_{max-s} is the maximum design nail force. Values of t_{max-s} were obtained from analyses with SNAIL by setting the factor of safety for global stability $FS_G = 1.0$. As discussed in Chapter 5, FS_G is set at 1.0 when determining the nail maximum design force to allow a rational utilization of the steel strength that is consistent with the soil strength mobilization. After T_{max-s} is determined and a steel tensile strength is selected, the necessary cross sectional area of the nail can be calculated.

Other design parameters listed in Table B.2 remained constant throughout the development of the design charts. Subsequently, these parameters were varied (see ranges in Table B.2) to investigate the influence of the specific parameters on the nail length and maximum design nail force calculated using the design charts. Nail inclination was kept constant at 15 degrees. The results indicate that the normalized length and maximum forces in the nail also depend on the global factor of safety, drillhole diameter, and the soil cohesion. The effect of the wall height was not significant, and thereby is not longer considered.

TABLE B.2
ADDITIONAL PARAMETERS

Included in μ?	Parameter	Units	Fixed Value used for Design Chart	Studied Range of Values
Included	Pullout Factor of Safety, FS_P	-	1.0	0.5 - 1.0
	Nail Horizontal Spacing, S_H	m (ft)	1.5 (5)	1.2 - 1.8 (4 - 6)
	Nail Vertical Spacing, S_V	m (ft)	1.5 (5)	1.2 - 1.8 (4 - 6)
	Total Unit Weight, γ	kN/m^3 (pcf)	18.9 (120)	17.3 - 20.5 (110 - 130)
Not Included	Global Factor of Safety, FS_G	-	1.35	1.35 - 2.0
	Drillhole Diameter, D_{DH}	mm (in.)	100 (4)	100 - 300 (4 – 12)
	Cohesion, c	kPa (psf)	5 (100)	5 – 24 (100 – 500)
	Wall Height, H	m (ft)	12 (40)	6.0 – 24 (20 – 80)

If the conditions of the problem being analyzed differ from those for which the charts were developed, the values L/H and t_{max-s} obtained from the charts must be corrected. Two sets of correction factors have been developed. One set of correction factors, C_{1L}, C_{2L}, and C_{3L}, were developed to correct the value L/H for drillhole diameter, soil cohesion, and global factor of safety, respectively. Values of C_{1L} are shown graphically as a function of the drillhole diameter, D_{DH} in

Figure B.7. Values of C_{2L} and C_{3L} are expressed as mathematical formulas also included in Figure B.7.

The second set of correction factors, C_{1F} and C_{2F}, were developed to correct $t_{max\text{-}s}$ for drillhole diameter and soil cohesion. No correction for global safety factor is necessary for $t_{max\text{-}s}$. Values of C_{1F} are shown graphically as a function of the drillhole diameter, D_{DH}, in Figure B.7. Values of C_{2F} are expressed as a mathematical formula included in Figure B.7.

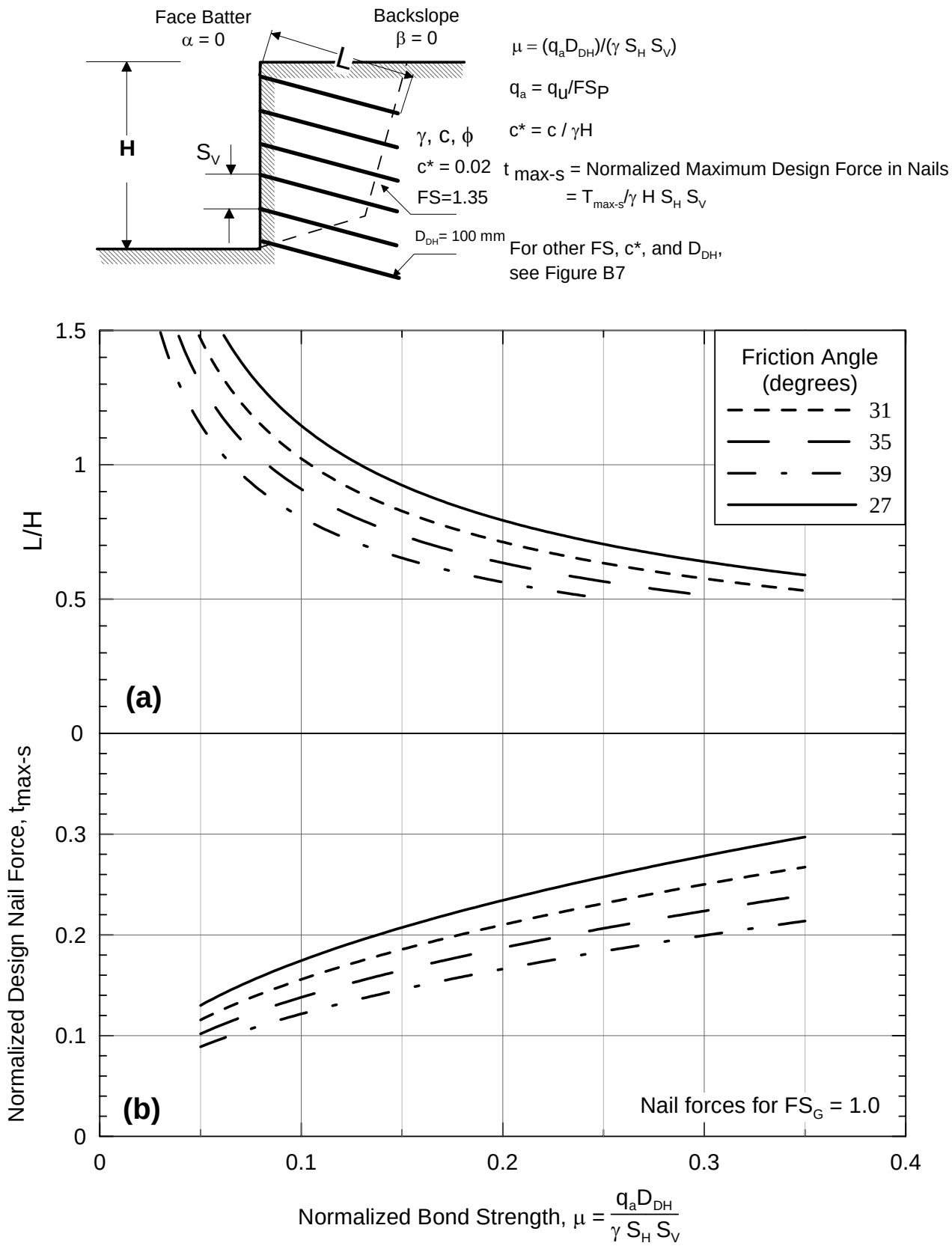


Figure B.1: Batter 0° - Backslope 0°

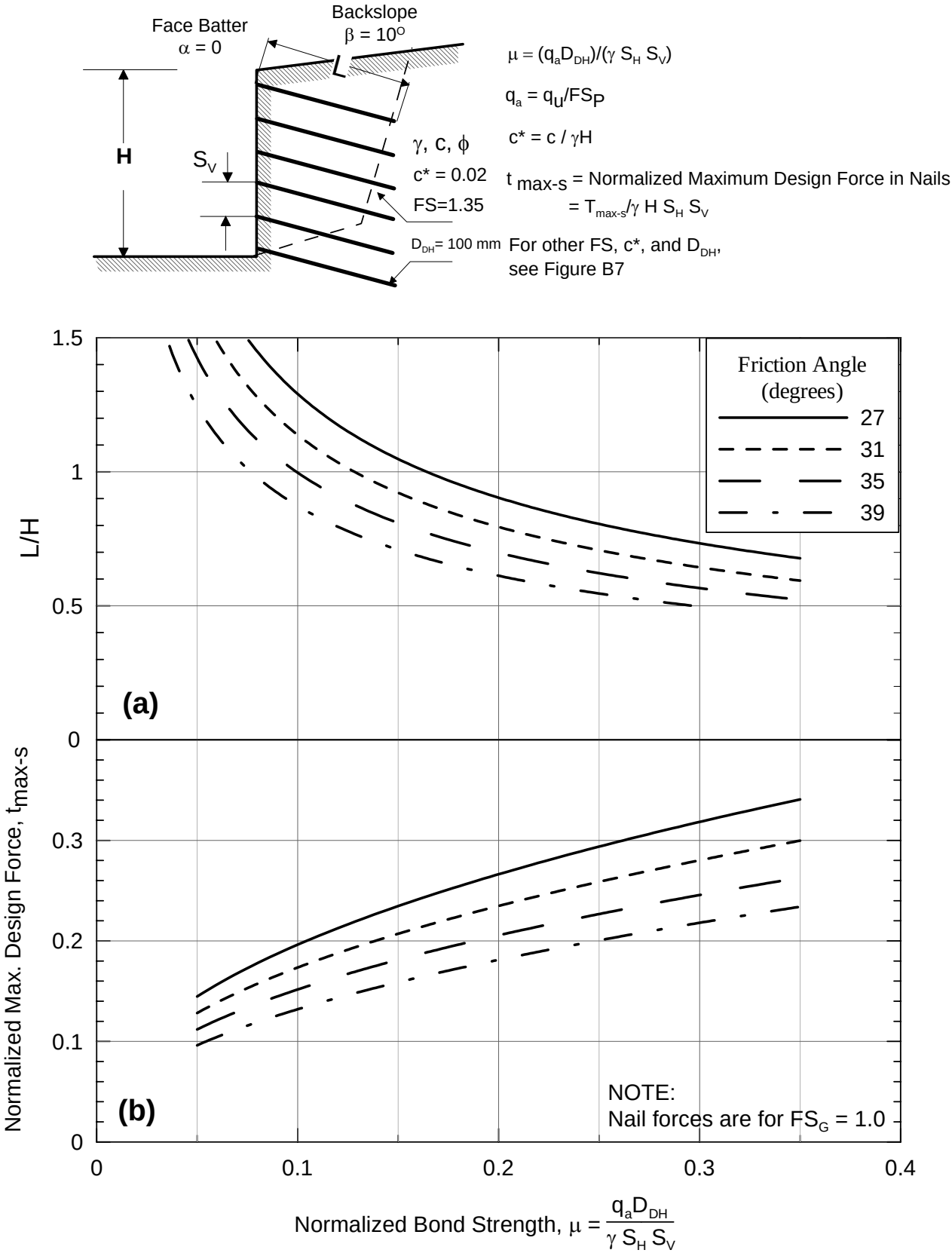


Figure B.2: Batter 0° - Backslope 10°

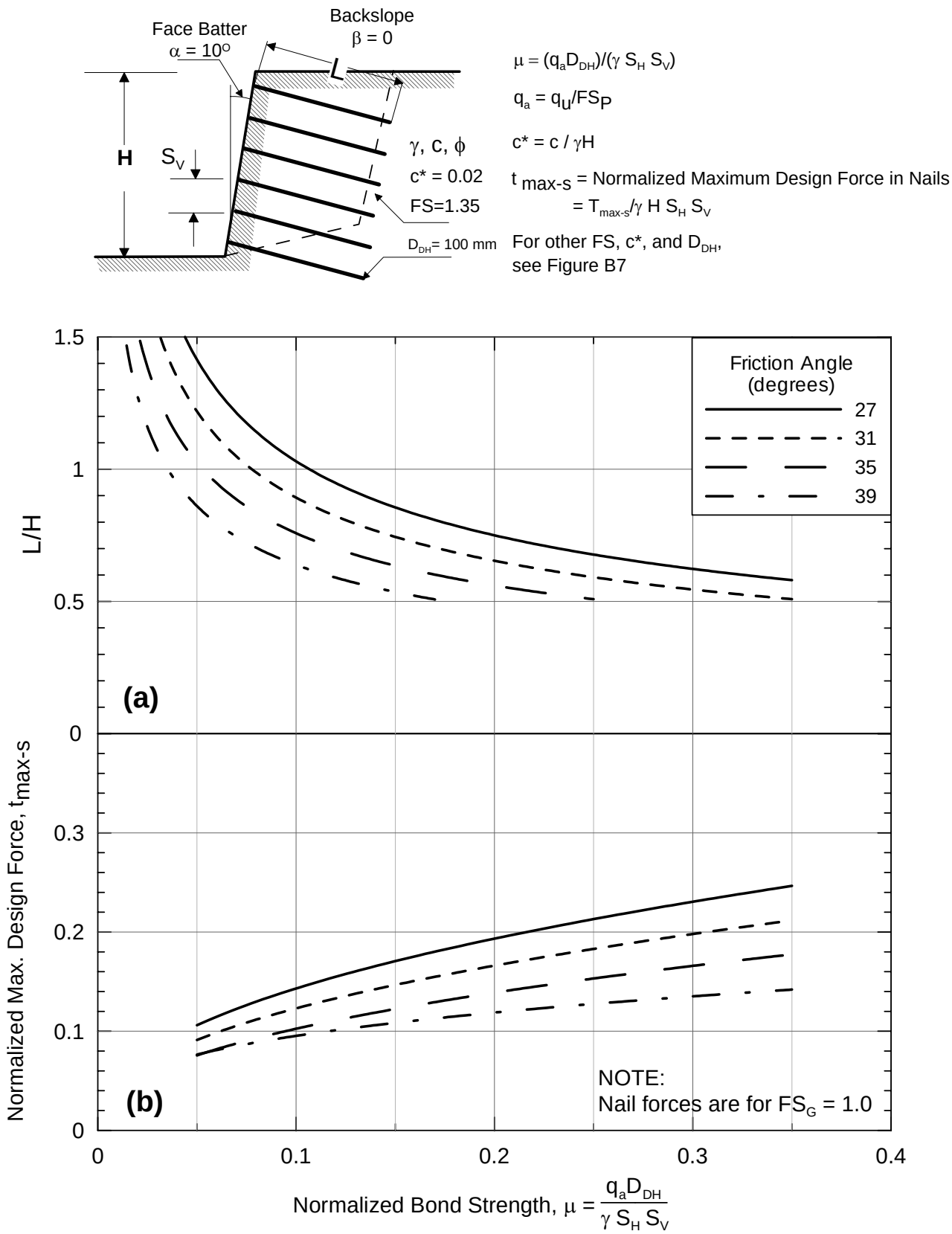


Figure B.3: Batter 10° - Backslope 0°

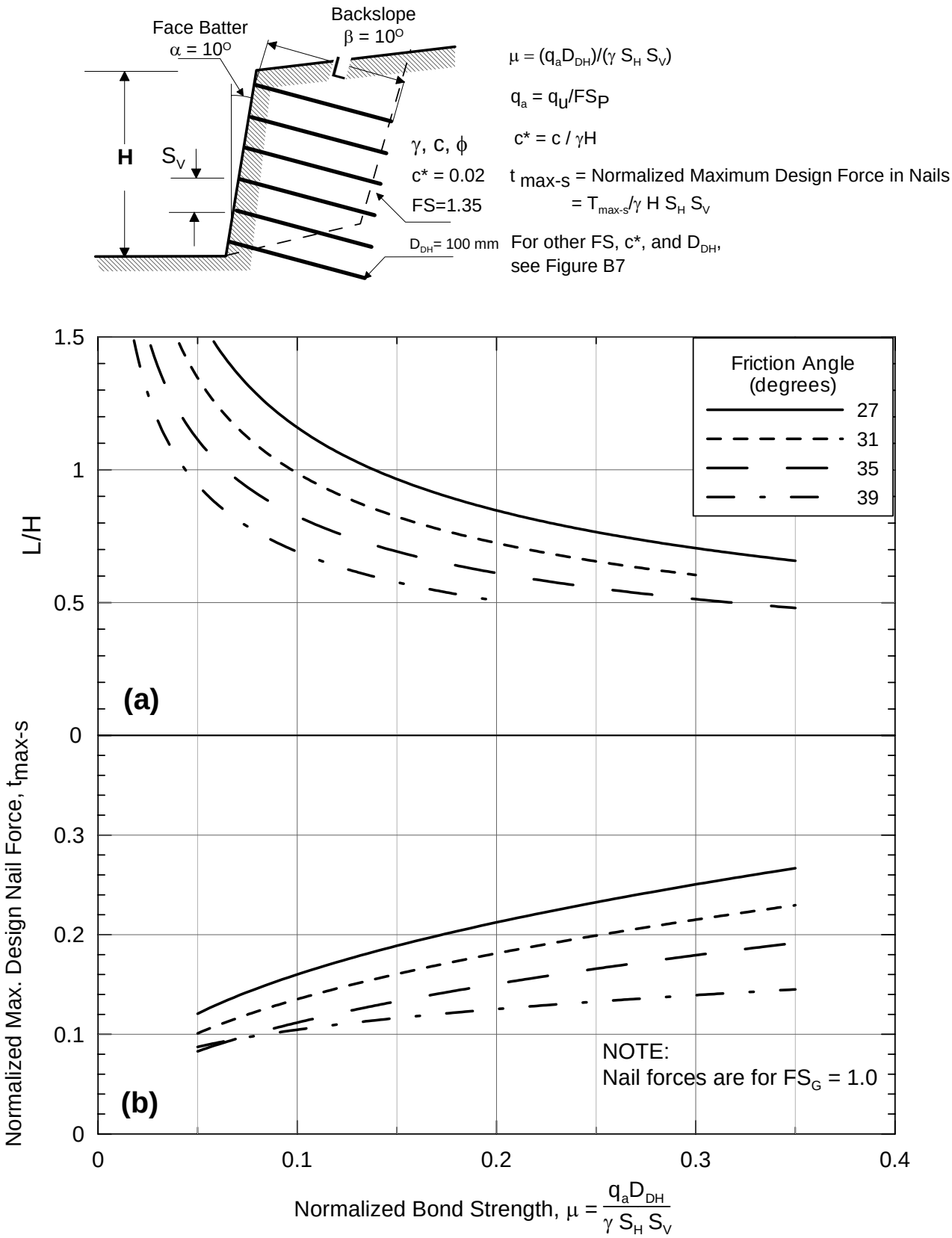


Figure B.4: Batter 10° - Backslope 10°

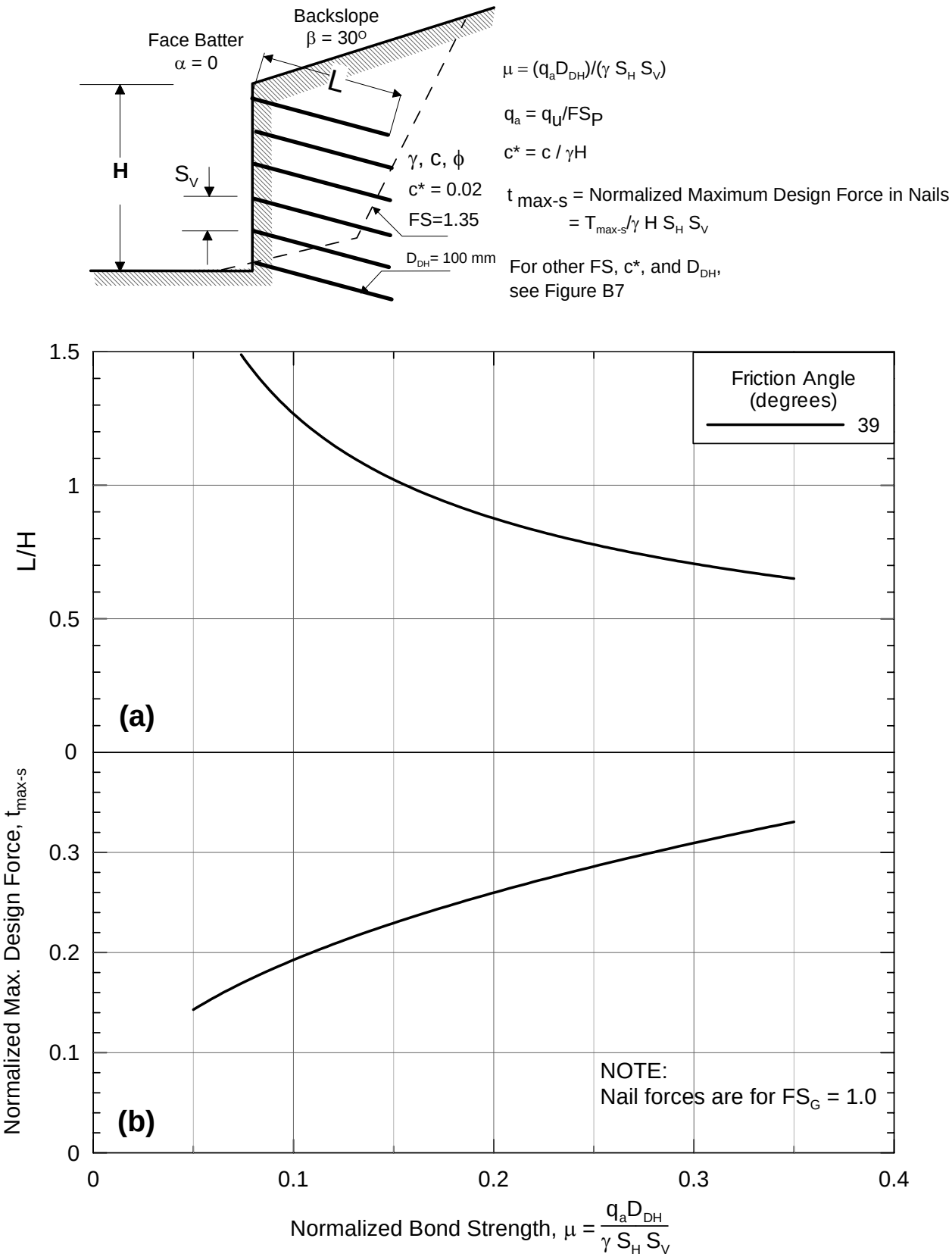


Figure B.5: Batter 0° - Backslope 30°

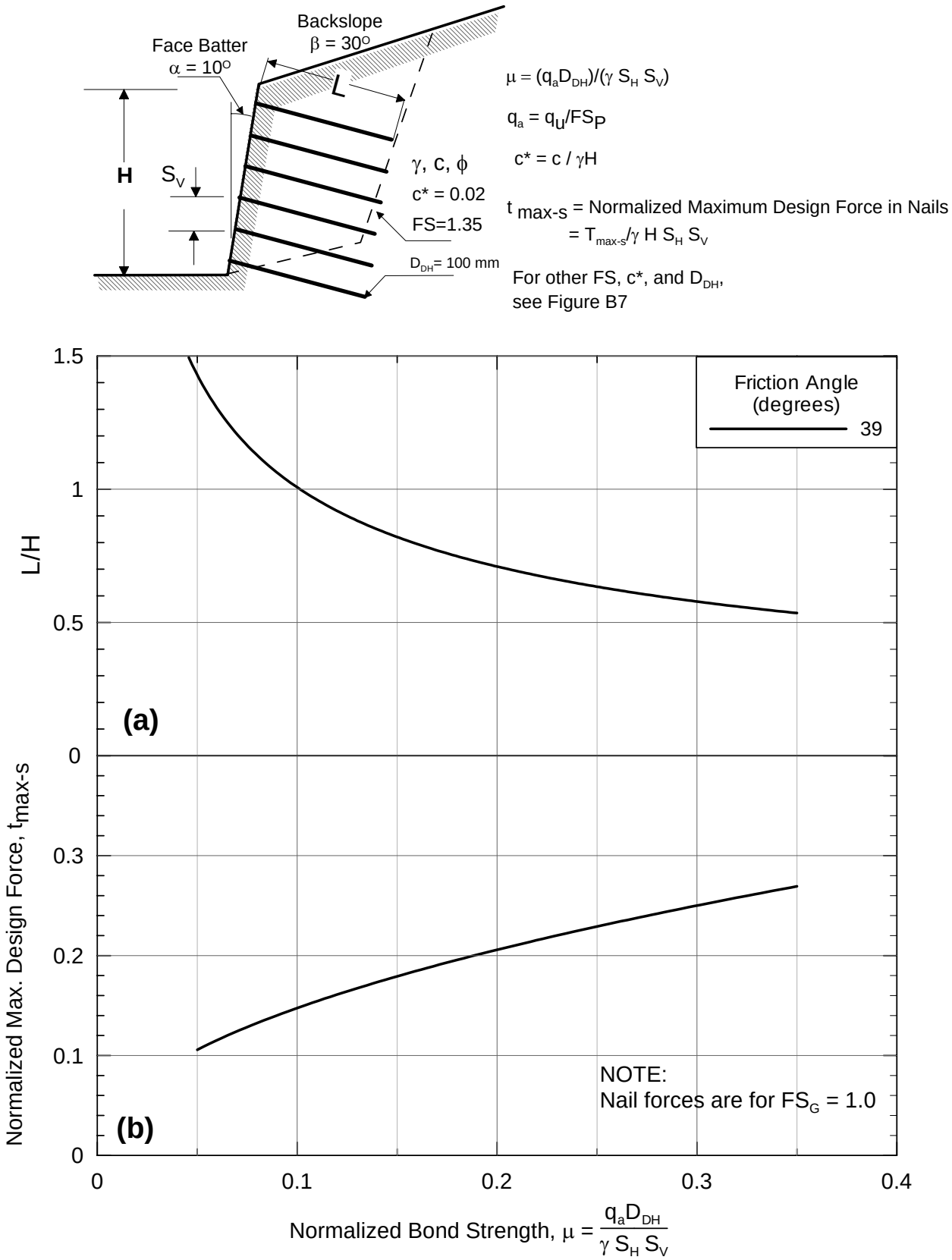


Figure B.6: Batter 10° - Backslope 30°

Corrections of Soil Length

$$\frac{L}{H}\ \text{(corrected)} = C_{1L} \times C_{2L} \times C_{3L} \times \frac{L}{H}\ \text{(from charts for } D_{DH} = 100\text{ mm, } c^* = 0.02\text{ , } FS_G = 1.35\text{)}$$

where:

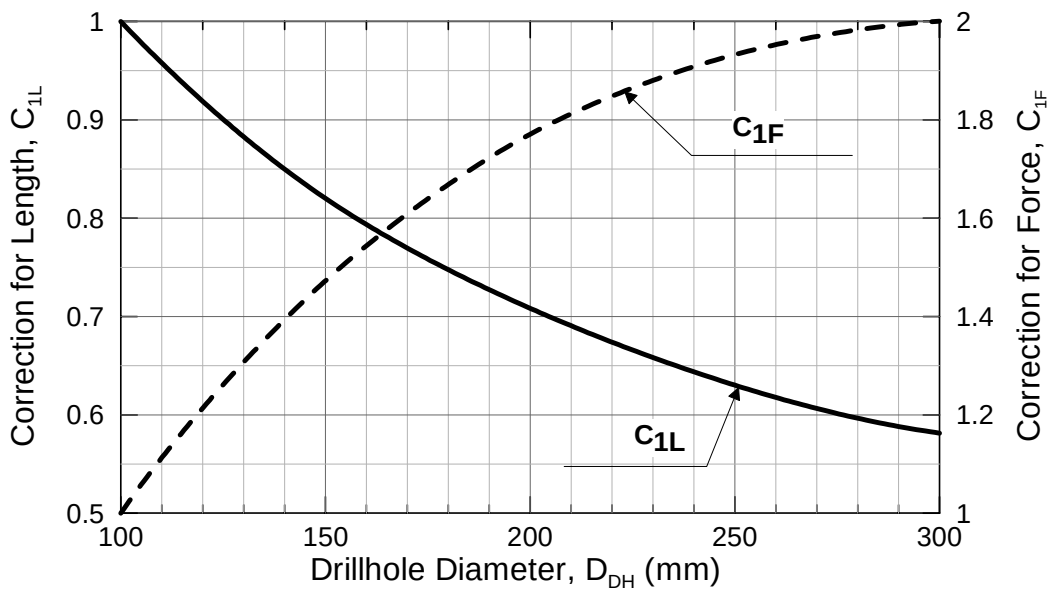
C_{1L} = Correction for Drillhole Diameter	see chart (a) below
C_{2L} = Correction for Soil Cohesion	see formula (b) below
C_{3L} = Correction for Global Factor of Safety	see formula (c) below

Corrections of Normalized Maximum In-Service Nail Force

$$t_{max\text{-}s}\ \text{(corrected)} = C_{1F} \times C_{2F} \times t_{max\text{-}s}\ \text{(from charts for } D_{DH} = 100\text{ mm, } c^* = 0.02\text{)}$$

C_{1F}= Correction for Drillhole Diameter	see chart (a)
C_{2F}= Correction for Soil Cohesion	see formula (b)

(a) Correction for Drillhole Diameter



(b) Correction for different values of normalized cohesion ($c^*=c/\gamma H$)

$C_{2L} = -4.0\ c^* + 1.09 \geq 0.85$

$C_{2F} = -4.0\ c^* + 1.09 \geq 0.85$

(c) Correction for different global factors of safety

$C_{3L} = 0.52\ FS + 0.30 \geq 1.0$

Figure B.7: Correction Factors

APPENDIX C CORROSION PROTECTION

C.1 INTRODUCTION

Protecting the nail bar and other metallic components of a soil nail against the detrimental effects of corrosion is necessary to assure adequate long-term durability of the soil nail wall. The level of corrosion protection that soil nails require depends largely on the nature of the project; specifically, the expected life, the perceived importance of the structure, and consequences of failure. In general, corrosion protection of soil nails is required for all permanent soil nail walls. Corrosion protection of soil nails can be achieved by physical and chemical protection measures or a combination of both. The selection of the type of corrosion protection depends on the design life of the structure (i.e., temporary or permanent), agressivity of the ground environment, the consequences of failure of the soil nail wall system, and the additional cost of providing a higher level of protection. In this appendix, basic information on corrosion is presented along with a description of corrosion protection systems used in soiling applications and the criteria used to evaluate the necessary level of corrosion protection for a soil nail system.

C.2 BASIC CONCEPTS OF CORROSION

C.2.1 General

Corrosion is the process in which a metal deteriorates, changes its physical properties, and ultimately dissolves due to chemical or electrochemical reactions with its surrounding environment. Most refined metals revert naturally and irreversibly from a less stable state to their native, more stable state, if conditions promoting corrosion are present and prevail over factors inhibiting corrosion.

The basic mechanism of metallic corrosion consists of the movement of ions in an electric circuit: from a metal surface (anode), through a nonmetallic conductor in solution (electrolyte), and onto another surface (cathode) due to a voltage difference, differences in oxygen concentrations, or differences in other environmental conditions. With time, the anode is consumed by the loss of metal into the electrolyte. In grouted soil nails, flow of ions can occur between bare reinforcing steel and a nearby metal object or between points on the metal surface not covered by grout. Numerous factors affect the rate of corrosion, including the characteristics of the metallic surface (e.g., steel, zinc), the type of electrolyte (e.g., water, industrial fluids), the presence and concentration of substances in the electrolyte (e.g., salts as carbonates, chlorides), the nature of the anode/electrolyte interface and other environmental factors.

C.2.2 Corrosion Mechanisms

The main corrosion mechanisms that may affect ungrouted soil nails are general corrosion. For grouted nails, the main corrosion mechanism is pitting or localized corrosion. Other mechanisms such as stress corrosion, hydrogen embrittlement and fatigue corrosion are usually not significant for soil nail applications. This section provides a general overview of general and localized

corrosion. Detailed information on the effects of various corrosion mechanisms on steel is provided in Weatherby (1982).

General Corrosion:

General corrosion occurs as a thin layer of rust uniformly distributed on the surface of steel bars. This type of corrosion can develop when unprotected steel is exposed to the environment during shipping and improper on-site storage. Under certain conditions, the thin rust layer becomes a protective film against further corrosion, a mechanism referred to as passivation. Metal loss is typically not significant with general corrosion, as long as the exposure time is limited and detrimental conditions promoting corrosion (see below) are absent. It is good practice to inspect nail bars for any surface corrosion and to remove thin layers of rust by wiping before the bars are installed in the drillholes. Lightly rusted bars may be inserted into the drillhole without rust removal in temporary applications.

Pitting or Localized Corrosion:

Localized corrosion develops as pitting or crevices and is confined to specific locations along the steel bar. In very corrosive or aggressive ground conditions, grouted bars will pit after only a few weeks of exposure. In general, localized corrosion is triggered by the removal of the protective passivation coating. Pitting and crevices promote non-homogenous concentrations of ions, an increase in electrochemical potential and further localized corrosion. The localized corrosion can be severe when aggressive ions, such as chlorides and/or sulfates are present. As the pit propagates deeper, the bar cross-section is reduced and this may eventually lead to a brittle, sudden failure. Pits or cracks on the surface of the steel bar are adequate reason for rejection of the bar.

C.2.3 Conditions Promoting Corrosion

The conditions promoting corrosion include:

- low resistivity of ground;
- high concentration of chlorides or sulfides in ground or groundwater;
- too low or too high hydrogen potential (pH) of ground or groundwater;
- high saturation conditions; and
- stray currents.

The factors above collectively define ground corrosion potential (or aggressivity of the ground). Examples of aggressive soils and factors that may increase corrosion potential include:

- *Acidic Soils*: These soils include soils with a high level of soluble iron and are characterized with low hydrogen potential (i.e., pH < 5).
- *Sodic Soils*: These are alkaline soils (i.e., pH < 9) with components favoring corrosion and are common in arid environments. Low precipitation and intense evaporation cause soluble salts (e.g., sodium, chloride, and sulfates) to be transported from the bedrock to shallow layers.

- *Calcareous Soils*: These are alkaline soils (7 < pH < 9) with large concentrations of sodium, calcium, calcium-magnesium carbonates and sulfates. Examples of these soils include those derived from calcite, dolomite, and gypsum.
- *Organic Soils*: These soils have unusually high water content (e.g., peats, mucks, and cinders) and may contain humic acid.
- *Materials of Industrial Origin*: These industrial waste "soils" can have pH values that vary significantly and extend along the whole pH scale. Examples of industrial waste soils are slag, fly ash, fills with construction debris, mine tailings, and acid mine waste.
- *Electrical Currents:* Corrosion may be induced in nail bars (or other metallic parts) when stray electrical currents are applied repeatedly. Stray currents can derive from power sources, such as electric rail systems, electrical transmission systems, and welding operations, and is particularly damaging in a marine environment. However, when the sources are located more than about 30 to 60 m (100 to 200 ft) from the nail bars, the potential of stray current corrosion is minimal (Elias, 2000).
- *Other Environmental Factors*: The corrosion potential of granular soils tends to increase slightly when they are in the 60 to 80 percent of the degree of saturation range. In regions where deicing salts are used, the top 2.5 m (7.5 ft) of soil behind a soil nail wall should be assumed to contain a higher concentration of chlorides.

C.2.4 Evaluation of Soil Corrosion Potential

Tests listed in Table C.1 (equivalent to Table 3.9) are used to classify the corrosion potential of the ground.

Table C.1: Criteria for Assessing Ground Corrosion Potential.

Test	Units	Strong Corrosion Potential/Aggressive	Mild to no Corrosion Potential/Non-Aggressive	ASTM Standard	AASHTO Test Method
pH	–	< 4.5, >10	5.5 < pH < 10	G51	T 289-91
Resistivity	ohm-cm	<2,000	Greater than 5,000	G57	T 288-91
Sulfates	ppm(1)	>200	Less than 200	D516	T 290-91
Chlorides	ppm	>100	Less than 100	D512	T 291-91
Stray current	–	Present	–	–	–

Note: (1) ppm = parts per million.

In general, the ground is classified with a strong corrosion potential or aggressive if any one of the conditions listed in the first column of Table C.1 exceeds the limits listed in the third column of the table during the service life of the soil nail wall. In addition, buried structures immediately adjacent to the project having suffered from corrosion or direct chemical attack might be an indication of strong corrosion potential. If all the conditions listed in the first column of Table C.1 satisfy the conditions listed in the fourth column of Table C.1, the ground is classified with no (or occasionally mild) corrosion potential. Tests from a nearby site can be used to evaluate the corrosion potential of the site if the designer can establish that the ground conditions are similar. Otherwise, if tests are

not performed, then the ground should be assumed to be aggressive. Classification of ground aggressivity should consider the possibility of changes during the service life of the soil nail wall, which may cause the ground to become aggressive (e.g., near mining operations, chemical plants, or chemical storage areas). Only experienced personnel must perform tests on soil resistivity, soil chemistry, and presence of stray currents.

C.3 CORROSION PROTECTION SYSTEMS

C.3.1 Introduction

Corrosion protection can be provided by physical and chemical protection, or a combination thereof. Physical protection involves placing a continuous barrier between the nail bar, other metallic parts, and the corrosion sources. Chemical protection consists of the use of a sacrificial material or a dialectric material, which will preclude the flow of electric current. Some of the corrosion protection systems currently in use utilize a combination of these mechanisms. In addition, when stray currents are of concern, electrical isolation of the nail assemblies should be used. The most common systems used to provide corrosion protection for soil nails are described below.

C.3.2 Methods for Corrosion Protection for Soil Nails

C.3.2.1 Grout Protection

This method of corrosion protection involves fully covering the bar with neat cement grout. After the bar is centered in the drillhole, neat grout is injected and fills up the annular space around the steel bar. Grout encapsulation provides both physical and chemical corrosion protection. When a minimum grout cover is in place, components such as carbonates and chlorides in the soil, and oxygen and humidity in the air are prevented or delayed in reaching the bar due to passivation. Additionally, the grout must have low permeability to ensure the effectiveness of the encapsulation. The grout provides an alkaline environment that reduces the corrosion potential. A minimum grout cover of 25 mm (1 in.) between the bar and the soil should be specified.

C.3.2.2 Epoxy Coating and Grout

Corrosion protection with epoxy (Figure C.1) consists of coating the nail bar with a fusion-bonded epoxy that is applied by the manufacturer prior to shipment to the construction site. Cement grout is placed around all epoxy-coated nail bars. The minimum required thickness of epoxy coatings is 0.4 mm [16 thousandths of an inch (mils)]. The epoxy coating provides physical and chemical protection, as epoxy is a dielectric material. In transporting and handling bars, the epoxy coating may be damaged before nail installation. Therefore, it is not uncommon to spray epoxy coating in the field on chipped or nicked surfaces. Applicable standards for epoxy coating are found in ASTM A-775.

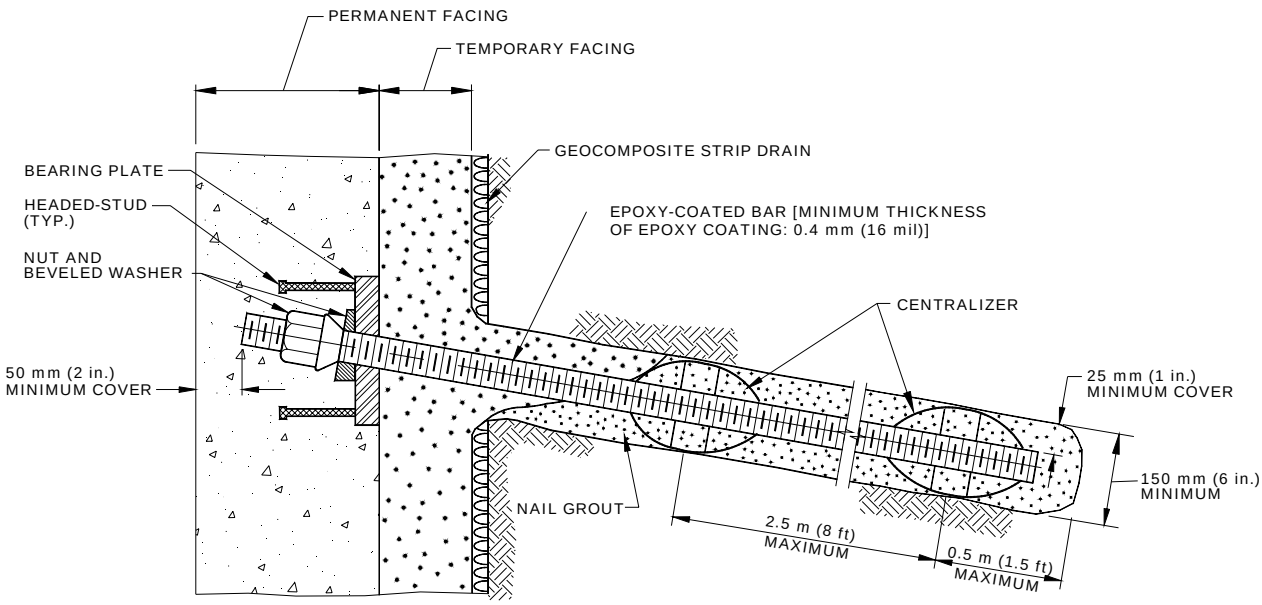
C.3.2.3 Galvanized Coating and Grout

A common method of providing corrosion protection is galvanization, which consists of applying a zinc coating on the steel surface. The process is performed by hot-dipping bars and other metallic

pieces with zinc. The protection provided by galvanized coating is both physical and chemical, as this process forms a protective layer of zinc oxide. Cement grout is placed around all galvanized nail bars.

C.3.2.4 Encapsulation (Double Corrosion Protection)

For maximum corrosion protection, a polyvinyl chloride (PVC) or high-density polyethylene (HDPE), minimum 1-mm (0.04-in.) thick corrugated sheathing may be installed around the grouted nail bar (Figure C.2), similar to what is done for ground anchors in permanent applications (whether the ground is aggressive or not). This practice is common in soil nail applications used for permanent soil nail walls built in an aggressive or unknown environment. The sheathing is corrugated to transfer the effect of anchorage to the surrounding grout. Grout must completely fill the annular spaces inside and outside the sheathing. The minimum grout cover between the sheathing and the nail bar is 10 mm (0.4 in.). This distance allows the injected grout to flow without difficulty and provides sufficient physical protection. Outside the sheathing, the minimum grout cover between the sheathing and the drillhole wall must be 20 mm (0.8 in.).



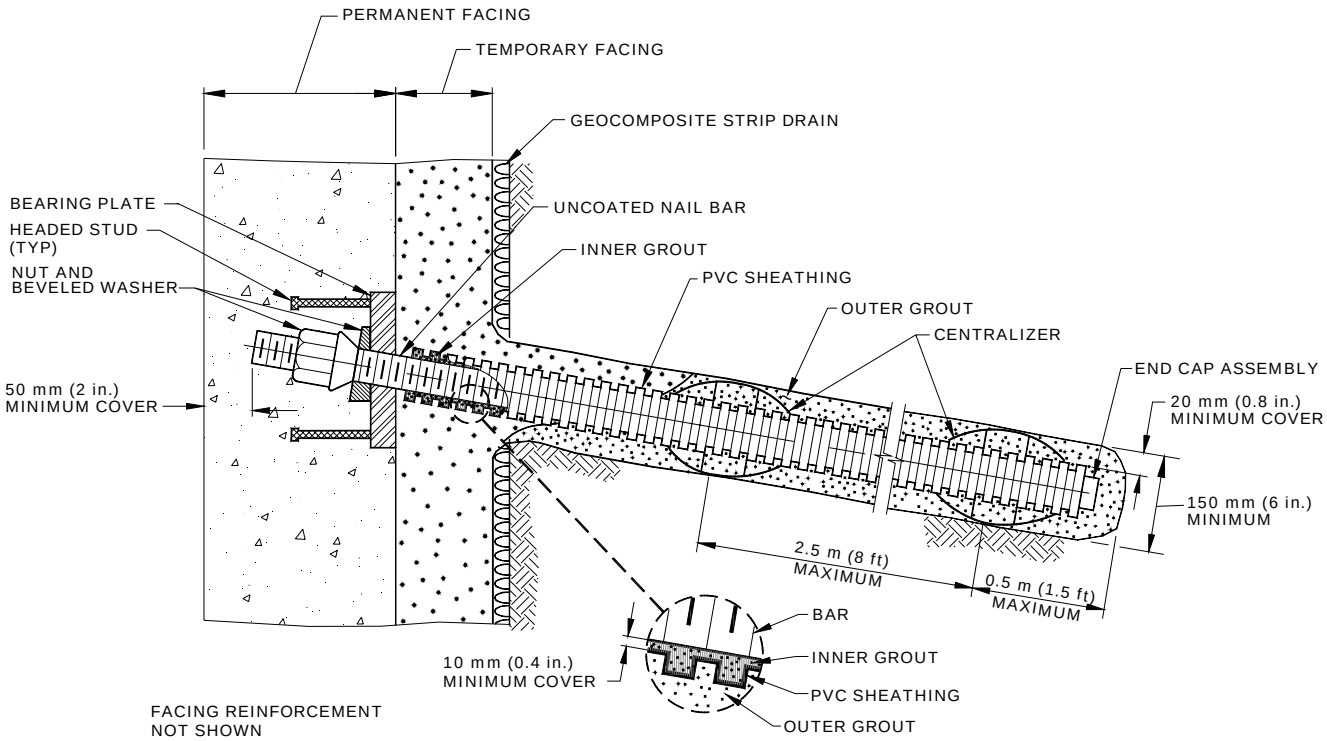
Modified from Byrne et al. (1998).

Figure C.1: Grouted Epoxy-Coated Nail (Class I Protection).

In some systems, the inner annular space is grouted in the shop and the whole assembly transported to the project site. The sheathing must be sufficiently strong to resist transportation, handling, and installation. Additionally, sheathing must be non-reactive with concrete, chemically stable, ultra-violet-light resistant, and impermeable. The minimum sheathing wall thickness is typically 0.875 to 1 mm (35 to 40 mils). Certain sheathing techniques may be proprietary.

C.3.3 Nail-Head Corrosion Protection

The use of materials made of galvanized steel and a minimum cover of 50 mm (2 in.) of concrete or permanent shotcrete provide corrosion protection of bearing plates, washers, and nuts. Epoxy coating can be applied on bearing plates and nuts.



Modified from Byrne et al. (1998).

Figure C.2: Sheathing-Encapsulation on Grouted Nail (Class I Protection).

C.3.4 Protection Against Stray Currents

When stray currents are present, protection is provided by electrical isolation. The method consists of interrupting the current passage between the electric source and the nail bar. The isolation can occur along the path or at the nail. PVC sheathings can be used for corrosion protection to provide isolation. When the sheathing is not present, the bearing plate and the nail head should be isolated from steel in the facing and all other nails. Effectiveness of electrical isolation must be field tested and verified by personnel qualified for this task after nail installation and before grouting.

C.4 CORROSION PROTECTION LEVELS

Two levels, or classes, of corrosion protection are commonly specified in U.S. practice:

- Class I Protection (two mechanisms for maximum protection), and

- Class II Protection (one mechanism for intermediate protection).

Table C.2 presents the protection levels and the protection systems commonly used in soil nail applications.

Table C.2: Corrosion Protection Requirements.

Class Protection	Level of Corrosion Protection	Protection Elements	Structure Service Life
I	Maximum	Grout and epoxy-coated bar, or Grout and PVC sheathing encapsulation	• Permanent
II	Intermediate	Grout and bare bar	• Temporary • Permanent with: o no aggressive soil; o no serious failure consequences; and o with high costs for increased corrosion protection level

Figures C.1 and C.2 present examples of Class I protection. In the U.S. practice of drilling and grouting nail bars, a Class II protection level is automatically provided, even if the ground has no corrosion potential. The use of grout in conjunction with PVC sheathing is known as Double Corrosion Protection and is used in aggressive or unknown conditions.

When using corrosion protection levels Class I or Class II as defined here, it is not necessary to incorporate a sacrificial thickness into the design. Sacrificial thickness is never used as the sole protection method in permanent applications. In temporary applications, unprotected, bare bars can be driven, as long as the soil corrosion potential is mild or insignificant. A preliminary and safe (for most conditions) estimate of the required sacrificial total thickness in unprotected bars is 2 mm (0.08 in).

C.5 CRITERIA FOR SELECTION OF CORROSION PROTECTION LEVELS

Existing guidelines developed for soil and rock ground anchors (see PTI, 1996) are used herein as a means to select the correct level of corrosion protection for soil nails. Although the detrimental effects of corrosion may be less in passive nail bars as compared to post-tensioned ground anchors, the use of the PTI guidelines in soil nail walls is helpful because the protection methods in both applications are similar. However, some elements of corrosion protection present in ground anchor applications (i.e., in trumpets and unbonded lengths) are not used in soil nail walls.

The PTI guidelines for ground anchors indicate that the selection of the level of corrosion protection is dictated by the following factors.

Service Life: Service life is based on the permanency of the structure (i.e., temporary or permanent). A service life of 18 months or less qualifies the structure as temporary. If the service life of the temporary structure is likely to be extended due to construction delays, an evaluation

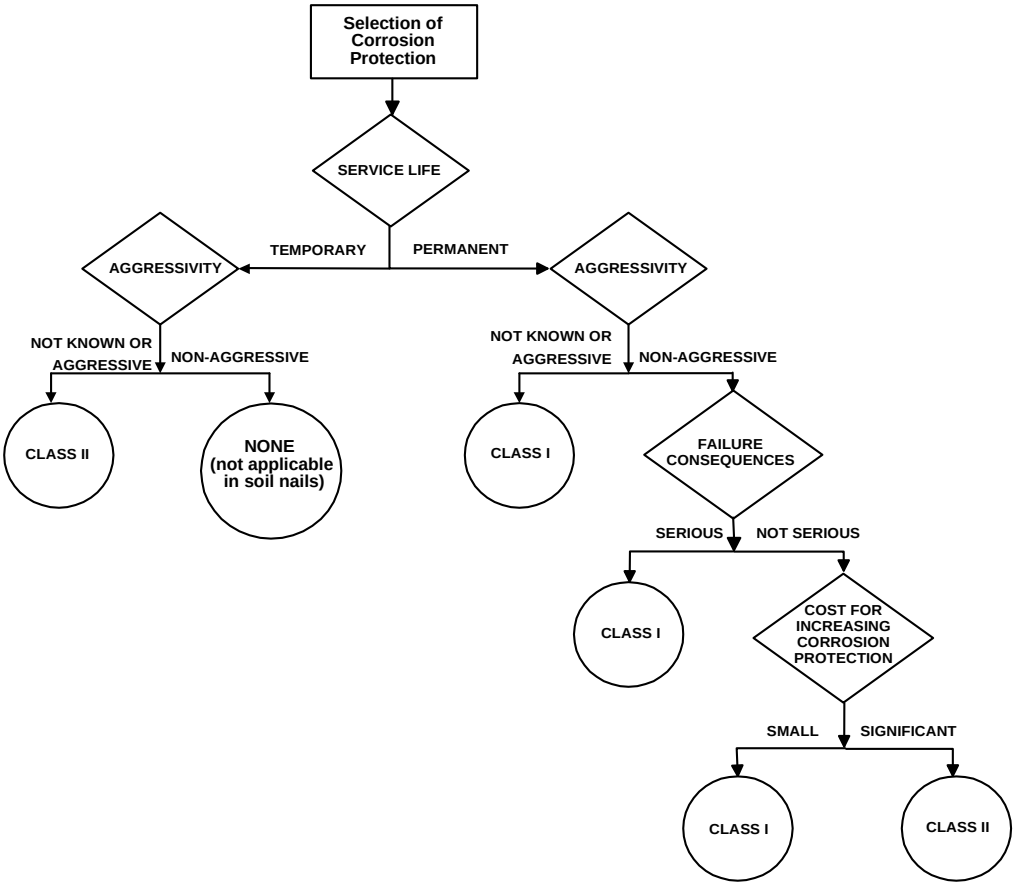
should be made to determine if additional corrosion protection, particularly in corrosive ground conditions, is necessary.

Ground Corrosion Potential: Nails in environments with high corrosion potential require the highest class of corrosion protection listed for each service life. Class II corrosion protection for temporary soil nail walls and Class I corrosion protection for permanent soil nail walls.

Failure Consequences: Serious consequences include loss of life, collapse of the wall, damage to nearby utilities and structures, structural repairs, and impact to traffic. These risks are expected in urban areas, walls alongside heavily traveled highways, and areas with problematic soil conditions where slope movements have been experienced. The PTI guidelines specify a Class I protection when the potential failure consequences are serious, regardless of soil corrosion potential.

Cost for Higher Corrosion Protection: The cost of providing a Class I protection level is higher than for a Class II because a larger drillhole diameter is necessary to install the sheathing and the cost of materials associated with the sheathing and/or epoxy-coated bars required for providing Class I protection is greater. The owner of the project should consider whether the increased costs of providing the higher level of corrosion protection are justifiable.

Figure C.3 presents a decision tree that can be used to select a level of corrosion protection consistent with project-specific constraints.



Modified from PTI (1996).

Figure C.3: Criteria for Selection of Corrosion Protection.

APPENDIX D

DESIGN EXAMPLE

SOIL NAIL WALL SUPPORTED EXCAVATION

INTRODUCTION

The soil profile behind the wall and the project requirements are similar to those of Design Example 1 presented in GEC No. 4 *Ground Anchors* (Sabatini et al., 1999). The similitude of project conditions will be helpful in comparing soil nail and ground anchor technologies.

INITIAL CONSIDERATIONS

A 10-m (33-ft) high soil nail wall is to be constructed as part of a roadway project. The road where the wall is to be constructed has a low to medium volume of traffic and thereby can be considered to be non-critical. Because the wall will be a permanent structure, aesthetic requirements call for a CIP concrete permanent facing. A 7.3-m (24-ft) road will be constructed 3 m (9.8 ft) behind the wall. The wall is to be constructed in medium dense silty sand with clay seams, as shown in Figure D.1. Underground utilities will be installed in the future. Although the final location of the utilities is not known, the utilities are expected to be installed in the area of influence of the nails, as indicated in Figure D.1. Additionally, some light structures will be built in the future approximately 10 m (33 ft) behind the wall. There is no source of corrosion potential at the site. The site is in a seismic zone and the horizontal seismic coefficient to consider in the analyses was estimated to be 0.17 from a seismic study.

SUBSURFACE CHARACTERIZATION

General

Geotechnical borings drilled in front of, alongside, and behind the proposed wall alignment indicate that the subsurface stratigraphy is relatively uniform. The profile shown in Figure D.1 is considered to be representative of the soil stratigraphy along the alignment of the wall.

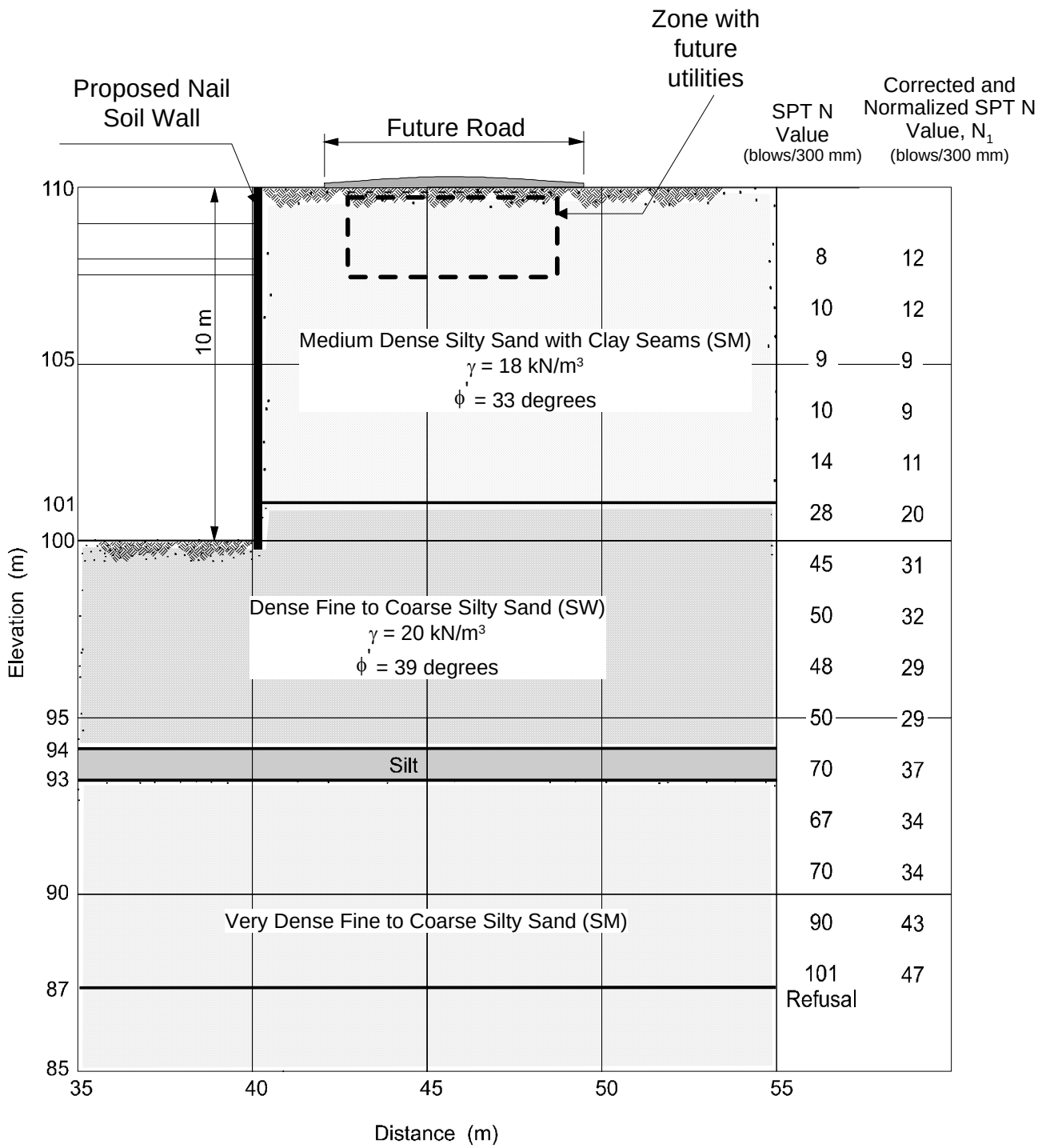


Figure D.1: Subsurface Stratigraphy and Design Cross-Section.

The design of the proposed wall shown in Figure D.1 will proceed according to the step-by-step methodology described in Chapter 6.

Predesign considerations

Some of the Predesign considerations the design Engineer must consider (i.e., topography, site access, ROW, utilities, adjacent structures, aesthetic requirements, and deformation, stability, and durability performance criteria) have been provided above.

STEP 1: Initial Soil Nail Wall Considerations (Section 6.2)

A. Wall Layout

1. Wall height (H), wall length (L_e), and face batter (α) are as follows:

   a. H = 10 m (33 ft);
   b. Wall Length, L_e >> H; and
   c. Face batter, $\alpha = 0$.

B. Soil Nail Vertical and Horizontal Spacing, S_H and S_V

1. Select S_V = 1.5 m (5 ft), which is typical;
2. Select $S_H = S_v$ = 1.5 m (5 ft). $S_H \times S_v$ = 2.25 m^2 (25 ft^2)≤ 4 m^2 (40 ft^2); and
3. Values of vertical spacing near the top and toe of the wall, $S_{V0} = S_{VN}$ = 0.5 m (1.6 ft), as shown in Figure D.2.

C. Soil Nail Pattern on Wall Face

1. Select a rectangular pattern, as the ground conditions are not so poor so as to justify a staggered pattern, in which a more uniform distribution of soil nail forces behind the wall is created.

D. Soil Nail Inclination, i

1. Select i = 15 degrees for all nails except the top row; and
2. Select i = 20 degrees for top row of nails to avoid planned utilities.

E. Soil Nail Length Distribution

1. Two nail length patterns are considered: one with uniform nail length and one with non-uniform nail length (Figure D.2);

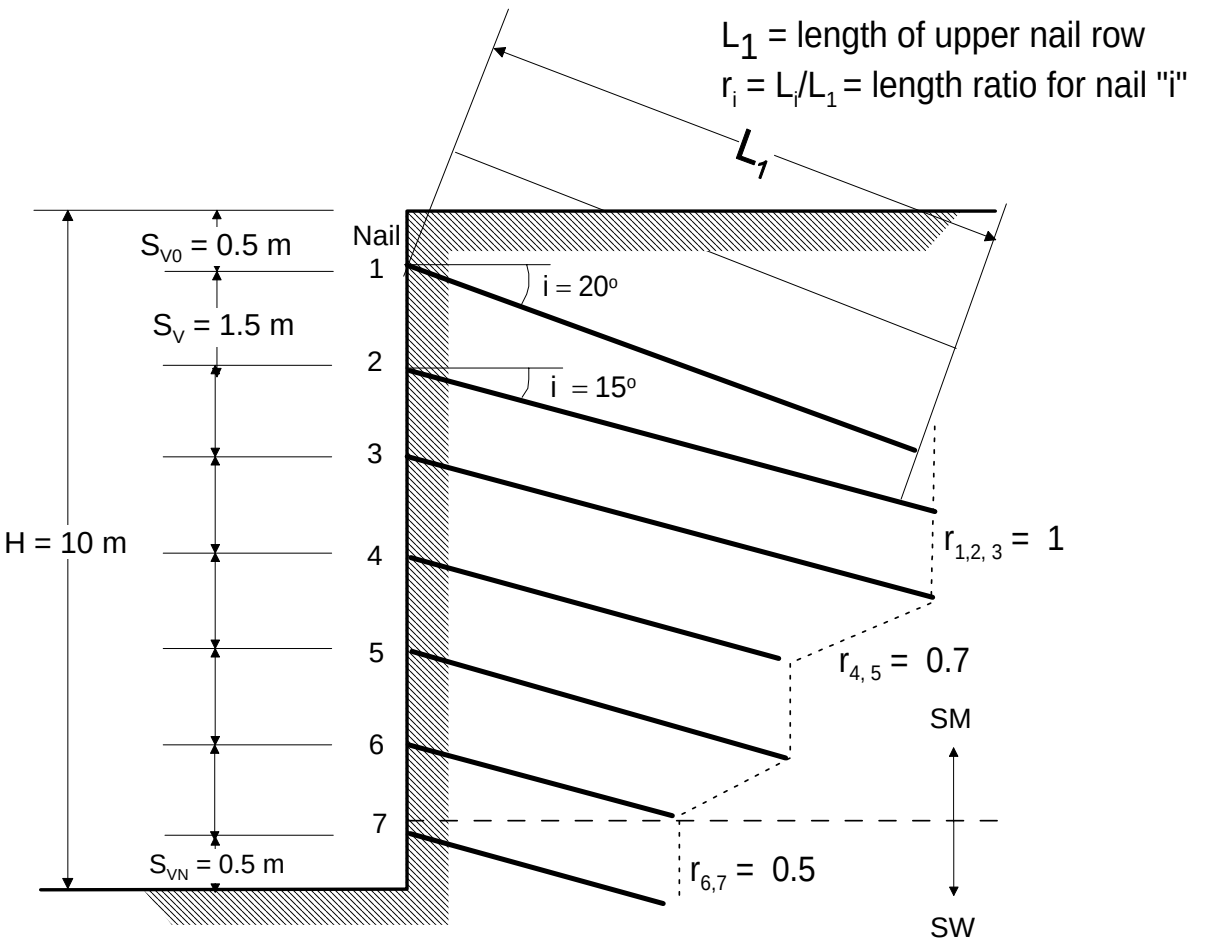


Figure D.2: Non-Uniform Nail Length Pattern.

2. The uniform pattern was selected to evaluate a widely used soil nail configuration that can be directly designed using design charts, is less prone to avoid cause poor performance in relation to sliding stability, and is easier to construct; and

3. The non-uniform pattern was selected with the objective of installing shorter nails in the last rows so that they would not penetrate in the lower, dense stratum, and thereby avoiding potential difficult drilling in this layer. To compensate for the lower nails in the lower portion of the wall, longer nails were selected for the upper

portions. This configuration would ideally be more desirable from the viewpoint of construction and post-construction deformation, as smaller wall lateral deflections are expected. **The major limitation of this configuration is that it is more prone to sliding stability, which must be thoroughly verified**.

4. It can be noticed that the candidate nail distributions selected in this example meet the criteria established in Section 6.2 E, as they:

   - A uniform length pattern is possible and is the first option;
   - In the non-uniform distribution:
     - longer nails are used in upper nail rows than in the lower ones to reduce wall deformations while maintaining the same global factor of safety;
     - no short nails are used in the top rows of the wall; and
     - excessively short nails in lower portion of wall are avoided. Shorter nails at the bottom should be not smaller than 0.5 H.
     - Non-uniform nail length patterns may be used if soil layers with very dissimilar conditions are encountered.

F. Soil Nail Materials

1. Select threaded solid bars;
2. Select a yield tensile strength, f_y = 520 MPa = 0.52 kN/mm^2 (Grade 75); and

G. Soil Properties

1. Soil Stratigraphy

   a. Figure D.1 provides the soil stratigraphy. In addition, a profile of the soil penetration resistance, which is represented by corrected and normalized SPT N_1 values, is also shown in Figure D.1;

2. Soil Unit Weight

   a. Figure D.1 provides soil unit weights. If this information is not available, the unit weight can be estimated from the SPT N-values provided in Figure D.1 following recommendations included in Section 3.6.3.

3. Soil Shear Strength Parameters

   a. Figure D.1 provides selected values of selected soil shear strength parameters. If this information were not available, estimate the soil ultimate shear strength parameters following procedures described in Section 3.6.4.

b. The shear strength of the silty sand behind the wall is mainly frictional, and the internal friction angle can be derived from correlations with the soil penetration resistance (e.g., Table 3.5);

c. For the temporary unsupported open face only, a nominal apparent cohesion of 2.5 kPa (50 psf) is assumed.

4. Bond Strength

The selection of the ultimate bond strength is deferred until the drilling technique is discussed in item H, part 4.

5. Groundwater Conditions

Groundwater was not encountered in any of the borings. These observations and supplementary review of groundwater data in the general project area indicate that groundwater levels at the site occur below elevations 93 m (305 ft).

6. Summary

For the conditions shown in Figure D.1, the following values are adopted:

Upper Silty Sand deposit

ϕ' = 33 degrees
c' = assumed conservatively as 0 for long-term conditions
c' = 2.5 kPa (50 psf) (temporary apparent cohesion)
γ = 18 kN/m^3 (115 pcf)

Lower Silty Sand deposit

ϕ' = 39 degrees
c' = 0
γ = 20 kN/m^3 (125 pcf)

H. Other Initial Considerations

1. Corrosion Potential

a. Soil testing indicates that the ground at the site has a resistivity greater than 5,000 ohm-cm and a pH between 6.2 and 6.8. Analyses also indicate that sulfides, sulfites, chlorides, and other substances known to promote corrosion are either absent or occur in insignificantly low concentrations. Additionally, stray currents are not present near the project site. As none of the conditions that promote

corrosion (listed in Table 3.9 or Table C.1) is present, the ground is considered non-aggressive.

b. Although the assessment of corrosion protection requirements can be deferred to after stability analysis, it is advantageous to perform this assessment first to minimize design iterations. For example, had the ground resulted aggressive, PVC sheathing [i.e., Class Corrosion Protection I (see Table C.2)] would have been necessary in this permanent soil nail wall. To accommodate the sheathing, a relatively large drillhole may be required. As a larger drillhole increases the nail pullout capacity, it is important to identify early in the design process the need to deploy sheathing.

c. Now the necessary level of corrosion protection is determined based following the flow chart of Figure C.3 (reproduced in Figure D.3).

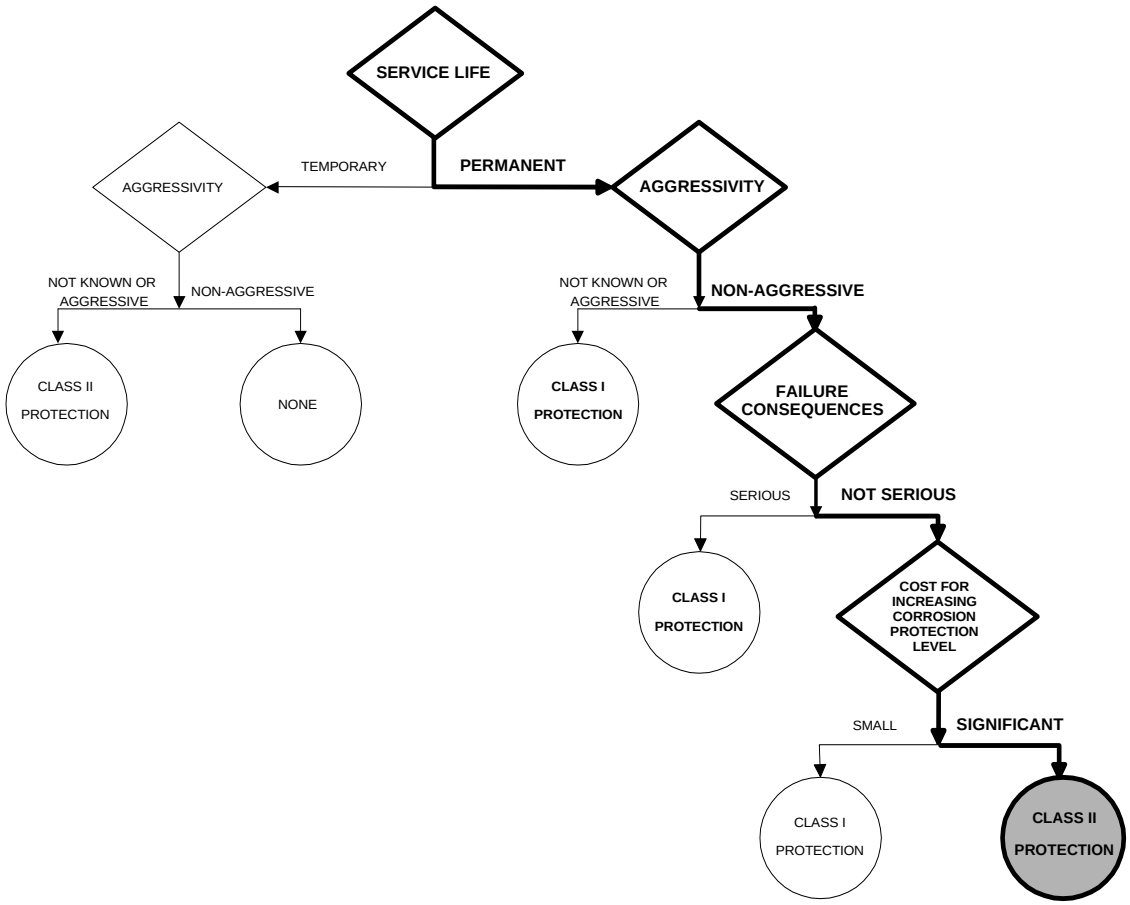


Figure D.3: Flow Chart for Level of Corrosion Protection.

- *Service Life*: Permanent structure;
- *Soil Aggressivity:* As indicated earlier, the ground conditions are considered to be non-aggressive;
- *Failure Consequences*: Because the proposed roadway adjacent to the soil nail wall has low to medium traffic volume, the consequences of a potential failure of the soil nail wall are considered to be moderate to small; and
- *Cost for Increasing Corrosion Protection Level*: It is assumed that the cost of providing an increased level of corrosion protection is significant.

d. The flow chart in Figure D.3 indicates that the required level of corrosion protection is Class II; and

e. Table C.2 indicates that this level of corrosion protection can be achieved with grout. The minimum grout cover is 25 mm (1 in.).

2. Drilling Methods

It is anticipated that either auger (first option) or driven casing methods will be used for the ground conditions at the site.

3. Drillhole Diameter

a. The drillhole diameter is estimated as the minimum compatible with the predominant soil conditions, drilling and nail installation method, and corrosion protection requirements. A preliminary drillhole diameter of D_{DH} = 150 mm (6 in.) is selected.

b. Verify the available cover for the selected drillhole diameter. It is assumed conservatively that a threaded nail bar size No. 32 (#10) will be used. This bar size has a maximum diameter of the threads of 36 mm (1.43 in.), as indicated in Table A.1. Provided that adequately spaced centralizers are installed, the available cover is (150 - 36)/2 = 57 mm (2¼ in.), which is greater than the minimum recommended cover of 25 mm (1 in.).

4. Bond Strength

Ultimate bond strengths were estimated from Table 3.10 based on soil conditions and construction methods. For cohesionless soils, and either auger or driven casing, it is assumed that the ultimate bond strengths are:

Upper Silty Sand: q_U = 100 kPa (14.5 psi)

Lower Silty Sand: q_U = 150 kPa (21.8 psi)

5. Safety Factors

The safety factors adopted for the project conditions are adopted from Table 5.3 and are presented in Table D.1 below.

Table D.1: Minimum Safety Factors.

Resisting Component	Symbol	Minimum Factor of Safety	
		Static Loads	Seismic Loads
Global Stability (long-term condition)	FS_G	1.5	1.1
Global Stability (1st Excavation Lift)	FS_G	1.2	NA
Bearing Capacity(1)	FS_H	3.0	2.3
Sliding Stability (1)	FS_{SL}	1.5	1.1
Pullout Resistance	FS_P	2.0	1.5
Nail Bar Tensile Strength	FS_T	1.8	1.35
Facing Flexure	FS_{FF}	1.5	1.1
Facing Punching Shear Failure	FS_{FP}	1.5	1.1
Headed-Stud Tensile Failure (A307 Bolt)	FS_{HT}	2.0	1.5

Note: (1) Not considered in example.

6. Loads

a. The combination of loads for the project conditions is adopted from AASHTO (1996) recommendations. Loads due to wind, ice, rib shortening, shrinkage, and temperature are not present. Only two load groups are considered: basic loads and seismic loads. However, some of the loads usually considered in these load groups, including buoyancy, centrifugal force, and live impact load, are absent. As lateral earth pressures (E) is implicitly considered in conventional soil nail walls, the resulting load scenarios are as follows:

Load Group I = [D + L]
Load Group VII = [D + EQ]

Where:

D is the dead load;
L is the live load; and

EQ is the seismic load.

b. In addition, a separate case of temporary load during excavation, with only permanent loads D, is considered.

c. The following loads considerations are made:

- D: The dead load includes the weight of the soil, the road behind the wall, and the CIP concrete facing.
- L: AASHTO (1996) recommends that an equivalent soil surcharge of 0.6 m (2 ft) acting within a distance equal to half the wall height behind the wall be added to account for (traffic) live load in the future road on top of the wall. This additional load is considered in the global stability analysis.
- EQ: An equivalent horizontal pseudo-static load with a horizontal seismic coefficient, k_h is considered for seismic loads in the global analysis.
- Note that the effect of the lateral earth pressures (E) must be considered explicitly in sliding stability analysis.

STEP 2: Preliminary Design Using Preliminary Charts (Section 6.3)

A. A preliminary design of the nail length and bar diameter can be performed using the series of simplified charts included in Appendix B. These charts are useful to obtain, in a simple way, initial estimates of the bar length and diameter without going through a full design. Although it is not necessary to use these charts in conjunction with a full design, this design example will present both approaches to illustrate the methodologies.

B. The nail length for the uniform nail length pattern can directly read from the charts in Appendix B. However, the nail length for the non-uniform pattern will be estimated considering that the total nail length for non-uniform patterns is approximately 10 to 15 percent larger than that for uniform nail length patterns under similar conditions;

C. Note that in using the simplified charts, a number of simplifications must be made as follows. (i) the soil conditions must be considered to be uniform for the entire depth of the wall and corresponding to the upper silty sand deposit; (ii) the nail inclination is considered to be uniform as i = 15 degrees (i.e., the steeper top nail row cannot be modeled); (iii) the live load cannot be considered explicitly, and must be considered as an equivalent additional soil overburden.

D. Uncorrected Uniform Nail Length

1. For the conditions defined previously, and with pullout factor of safety, FS_{PP}, from Table D.1, the allowable bond strength, q_a, is calculated as follows:

   $q_a = q_U / FS_P = 100/2 = 50$ kPa (7.3 psi)

2. The normalized pullout resistance, μ, to be entered in the charts is:

   $\mu = (q_a D_{DH})/(\gamma S_H S_V) = 50 \times 0.15 / (18 \times 1.5 \times 1.5) = 0.185$

3. The normalized cohesion, c*, is:

   $c^* = c/\gamma H = 2.5/ (18 \times 10) = 0.014$

4. For $\alpha = \beta = 0$, and $\phi' = 33°$, the reference normalized soil length, L/H, is obtained from Figure B.1 (reproduced in this appendix as Figure D.4) as L/H = 0.71.

5. The charts were obtained for the following geometric and material conditions:
   - drillhole diameter: $D_{DH} = 100$ mm (4 in.)
   - normalized cohesion: $c^* = 0.02$
   - factor of Safety: $FS_G = 1.35$ (for temporary soil nail walls)

6. The following corrections to L/H are necessary (Figure D.5):

   Drillhole diameter: $C_{1L} = 0.83$ (read from Figure B-7 for D_{DH}=150 mm);
   Soil Cohesion: $C_{2L} = -4.0 \times c^* + 1.09 = -4.0 \times 0.014 + 1.09 = 1.03 \geq 0.85$; and
   Safety Factor: $C_{3L} = 0.52 \times FS_G + 0.30 = 0.52 \times 1.5 + 0.30 = 1.08 \geq 0.85$.

7. The adjusted normalized nail length is adjusted as:

   $L/H = C_{1L} \times C_{2L} \times C_{3L} \times L/H = 0.83 \times 1.03 \times 1.08 \times 0.71 = 0.92 \times 0.71 = 0.66$

8. To account for the added surcharge from live load, increase wall height by 0.6 m (2 ft); thereby, H = 10.6 m (34.8 ft).

9. The soil nail results:

   $L = 0.66 \times 10.6 = 7.0$ m (23 ft)

10. Considering that seven rows of nails are needed, the total nail length for the uniform-length pattern, $L_{TOT\ U}$, is:

    $L_{TOT\ U} = 7\ L = 7 \times 7 = 49.0$ m (161 ft)

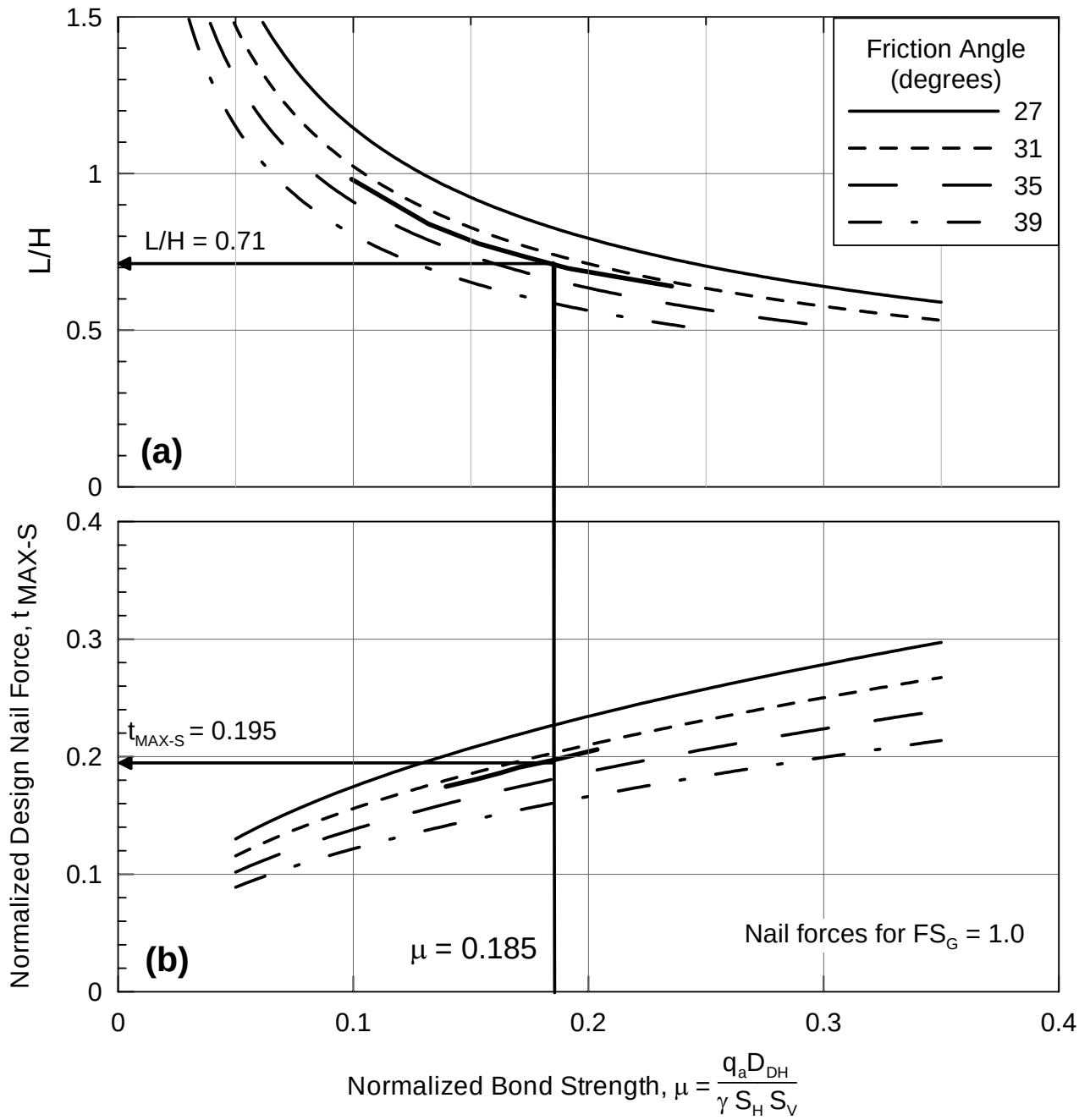


Figure D.4: Normalized L/H and Tensile Force for α = β = 0, ϕ' = 33°.

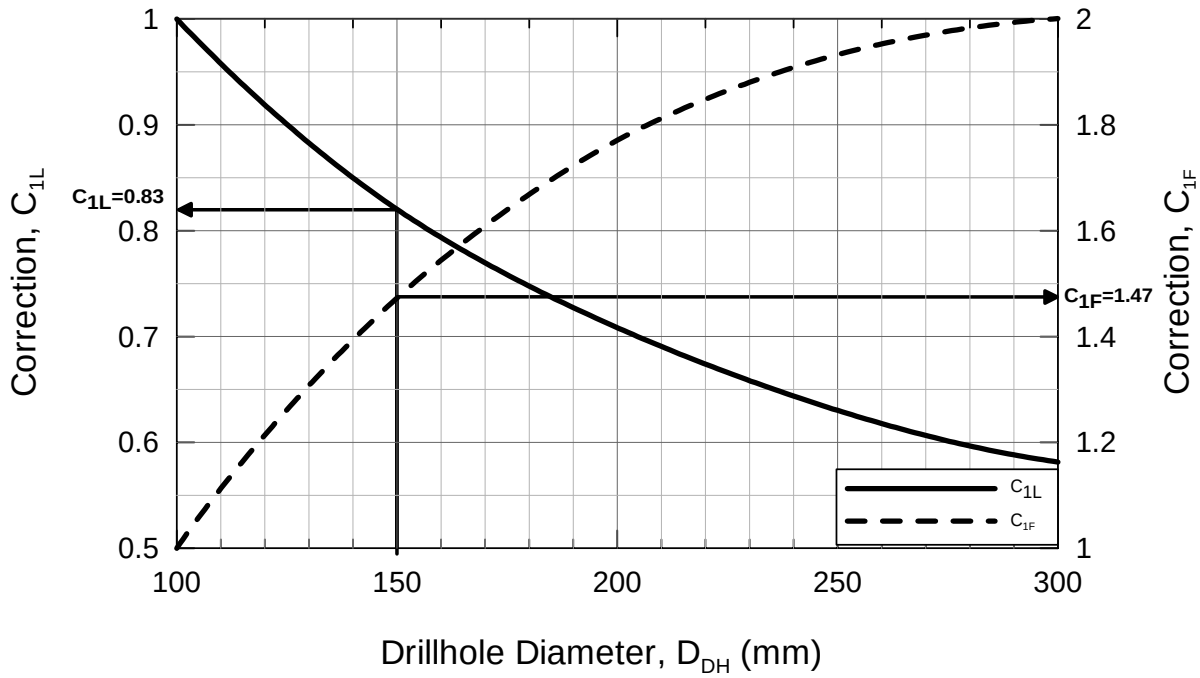


Figure D.5: Drillhole Diameter Correction.

B. Non-Uniform Nail Length Pattern

1. The nail length for the non uniform pattern, $L_{TOT\ NU}$, is estimated to be 10 percent larger than the total length of the uniform pattern (Section 6.2), $L_{TOT\ NU} = 1.1\ L_{TOT\ U}$

2. Using the normalized lengths $r_i = L_i/L_1$ from Figure D.3, the total nail length of the non-uniform pattern is calculated as:

   $L_{TOT\ NU} = \sum r_i\ L_1 = L_1\ (3 \times 1 + 2 \times 0.7 + 2 \times 0.5) = 5.4\ L_1$

   Where: L_i = length of nail i, L_1 = length of the upper nail 1, and

   $L_1 = 1.1 \times 49 / 5.4 = 10.0$ m (32.7 ft),

   $L_2 = L_3 = 10.0$ m (32.7 ft),

   $L_4 = L_5 = 0.7 \times 10.0$ m = 7.0 m (23 ft), and

   $L_6 = L_7 = 0.5 \times 10.0$ m = 5.0 m (16.4 ft).

C. Nail Maximum Tensile Force

1. Read normalized maximum design nail force, $t_{max\text{-}s}$, from charts in Appendix B (reproduced as Figure D.4).

$t_{max\text{-}so} = 0.195$

2. Apply corrections:

   Drillhole diameter: $C_{1F} = 1.47$ (from Figure D.5)
   Soil Cohesion: $C_{2F} = -4.0 \times c^* + 1.09 = -4.0 \times 0.014 + 1.09 = 1.03 \geq 0.85$.

3. The corrected normalized maximum nail force is:

   $t_{max\text{-}s} = C_{1F} \times C_{2F} \times t_{max\text{-}s} = 1.47 \times 1.03 \times 0.195 = 0.30$

4. The maximum design nail force is:

   $T_{max\text{-}s} = \gamma \times H \times S_H \times S_V \times t_{max\text{-}s} = 18 \times 10.6 \times 1.5^2 \times 0.30 = 128$ kN (28 kips)

D. The nail tensile capacity, R_T, is calculated as:

$R_T = FS_T \times T_{max\text{-}s} = 1.8 \times 128 = 230$ kN (51 kips)

E. The necessary nail bar cross-sectional area, A_T can be calculated as:

$$A_t = \frac{T_{max\text{-}s} \times FS_T}{f_Y} = \frac{R_T}{f_Y}$$

$$A_t = \frac{128\ [\text{kN}] \times 1.8}{0.52\ [\text{kN/mm}^2]} = 443\ \text{mm}^2\ (0.69\ \text{in.}^2)$$

F. From Table A.1, a threaded bar of 25 mm diameter (#8) with a cross-sectional area of 510 mm^2 (0.79 $in.^2$) is selected.

G. A summary of parameters obtained from the simplified charts is presented below.

1. Uniform length pattern

   L = 7.0 m (23.0 ft)

2. Non-uniform length pattern

   $L_1 = L_2 = L_3 = 10.0$ m (32.7 ft)

   $L_2 = L_5 = 7.0$ m (23.0 ft)

   $L_6 = L_7 = 5.0$ m (16.4 ft)

3. Maximum design nail force

   $T_{max\text{-}s} = 128$ kN (28 kips). This value is considered tentatively the same for the uniform and non-uniform patterns.

4. Necessary steel area:

   A_t = 443 mm^2 (0.69 in.^{2})

5. Actual steel area:

   A_t = 510 mm^2 (0.79 in.^{2})

STEP 3: FINAL DESIGN (Section 6.4)

A. External Failure Modes

1. Evaluation of Global Stability.

   a. The global stability is evaluated with the computer program SNAIL. The following geometric and load conditions considered in this example are included in Table D.2.

Table D.2: Load Cases.

Load Case	Nail-Length Pattern	Loads	Number of Nails	Excavation Depth H, in m (ft)	Failure Surface	Figure
1	No nails	D	0	First Lift 0.7 (2.3)	Through toe	D.6
2	Uniform	DL + L	7	Full: 10 (33)	Through toe	D.7
3	Non-uniform	D	4	Midheight 5.5 (18)	Through toe	D.8
4	Non-uniform	DL + L	7	Full: 10 (33)	Through toe	D.9
5	Non-uniform	D + EQ	7	Full: 10 (33)	Through toe	D.10

   b. The soil shear strength parameters and bond strengths were entered. The same soil-strength parameters were considered for both static and seismic loads. The potential for liquefaction is considered negligible.

   c. The nail lengths and maximum design nail forces are the iteratively calculated with SNAIL for the FS_G included in Table D.1.

   d. To ensure that pullout failure controls over tensile or punching shear failure, artificially large values of nail diameter and facing capacity are entered in SNAIL.

2. Results

a. Factors of Safety: The results of the SNAIL analyses are presented in Figures D.6 through D.10, and summarized in Table D.3. The figures show the critical failure

surfaces that correspond to the critical factor of safety. SNAIL also reports nine other failure surfaces in output files.

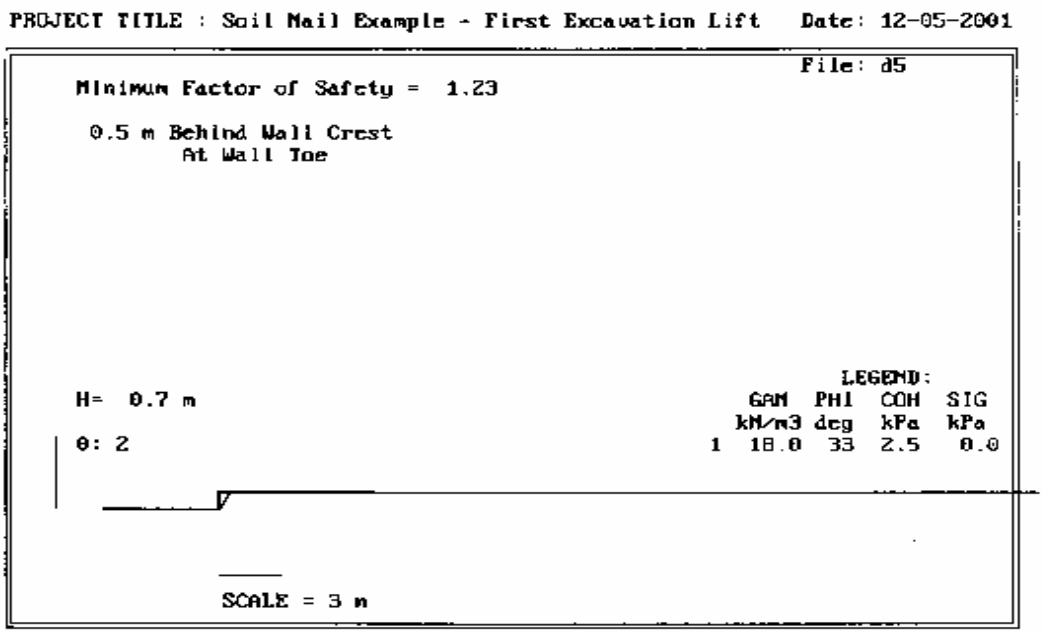


Figure D.6: Load Case 1: First Excavation Lift – No Nails – No Live Loads.

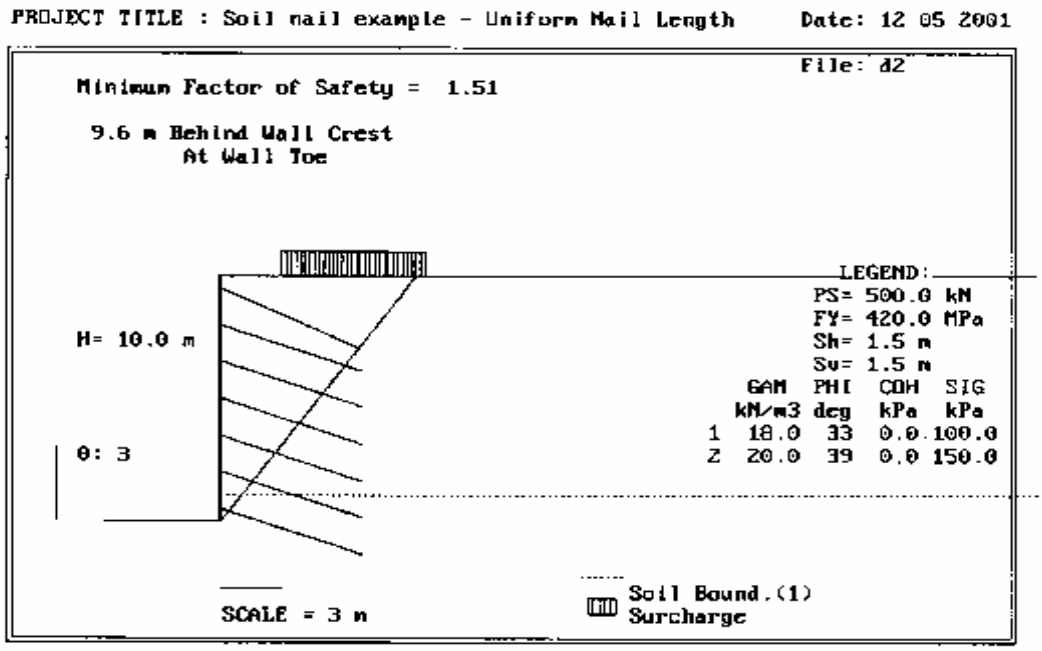


Figure D.7: Load Case 2: Uniform Length Pattern and Static Loads.

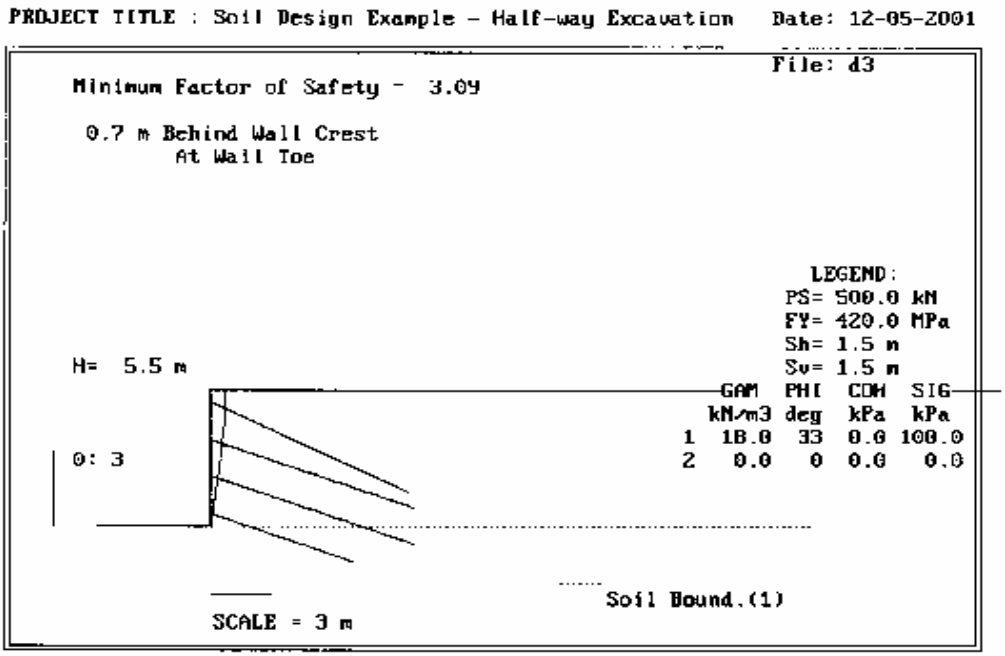


Figure D.8: Load Case 3: Excavation Depth of H = 5.5 m.

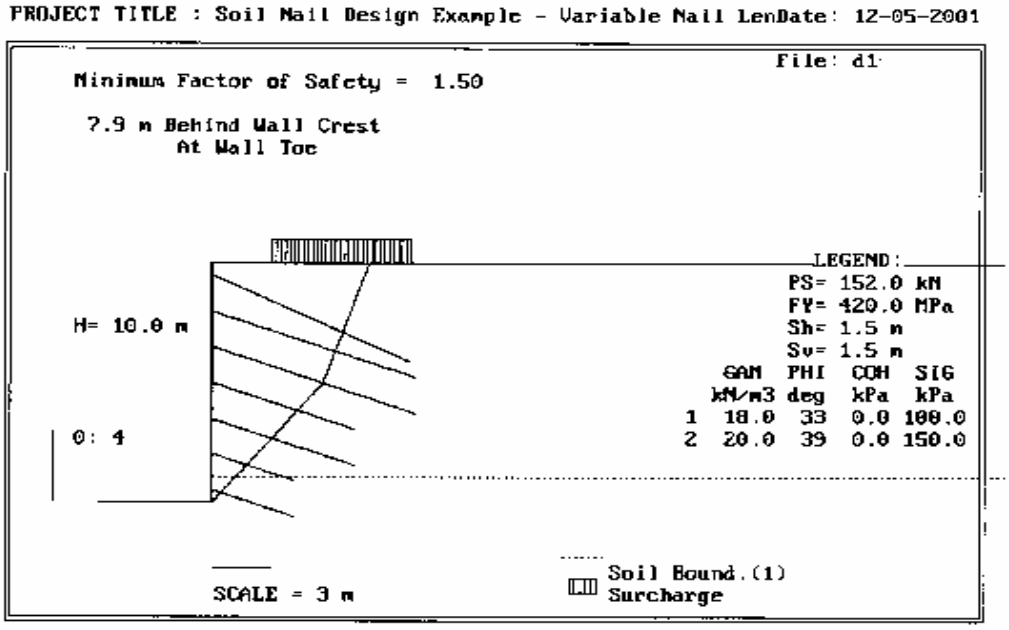


Figure D.9: Load Case 4: Non-Uniform Length Pattern and Static Loads.

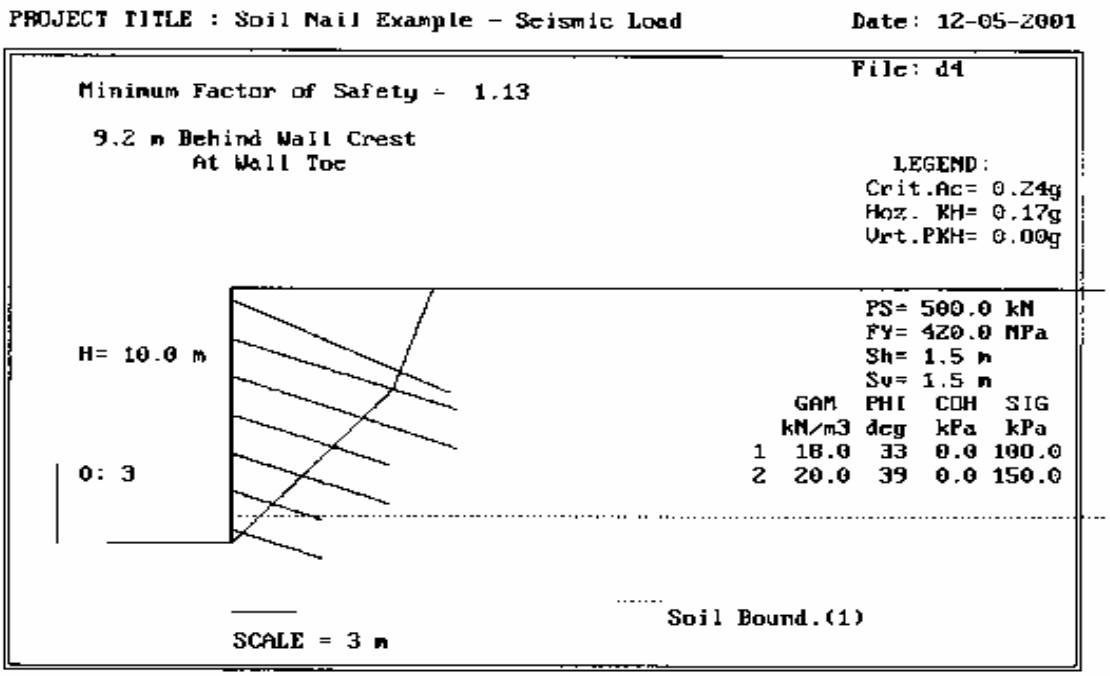


Figure D.10: Load Case 5: Non-Uniform Length Pattern and Seismic Loads.

The computed factors of safety for each of these cases are summarized below.

Table D.3: Summary of Results.

Case	Description	Calculated FS_G	Minimum Acceptable FS_G
1	No nails	1.23	1.2
2	Uniform Pattern – Static	1.51	1.5
3	Half Excavation – Static	3.09	1.35
4	Non-uniform Pattern – Static	1.50	1.5
5	Non-uniform Pattern – Seismic	1.13	1.1

Global Stability

All calculated FS_G met the minimum criteria for global stability.

Case 1 illustrates the beneficial effect of the cohesion on the stability of the first lift. Case 2 shows adequate stability margin. This case proved to be the most critical for the maximum design nail forces, $T_{max\text{-}s}$, for the temporary facing.

Case 3 illustrates that the FS_G may be significantly larger than at shallow and intermediate excavation depths after nails are in place. The nail length and nail forces were determined based on Case 4 because this case is the one that can be least represented with the simplified charts. The nail length determined for Case 4 was also verified for seismic loads (Case 4). Results from this case show that the calculated FS_G is greater than the minimum recommended safety factor for k_h = 0.17. SNAIL also provides the yield acceleration (critical horizontal seismic coefficient) k_h = 0.24, for which FS_G = 1.

Bearing Capacity

Taking advantage of SNAIL's capabilities, the case of a failure surface passing below the wall toe (not shown) was also analyzed to consider the potential of a deep-seated failure. These analyses gave a much higher FS_G than the minimum recommended factors of safety.

Sliding Stability

Due to space limitation a sliding stability is not performed in this example. A discussion on the ground conditions is warranted. Based on similarities in ground conditions and project characteristics, it can considered that the dense silty sand at the bottom of the excavation is sufficiently competent so that a sliding stability failure is not likely for the uniform nail distribution. However, these favorable conditions may not hold true for the non-uniform nail distribution. Therefore, it is recommended that sliding stability must be given full consideration in any design of soil nail wall and design engineers must consider very cautiously non uniform nail distributions.

b. Nail Length

For a FS_G = 1.5, the necessary nail lengths are:

- *Uniform Nail Length (Cases 1 - 3)*

  L = 7.0 m (23.0 ft)
  $L_{TOTAL\ U}$ = 50.4 m (165.4 ft)

- *Non Uniform Nail Length (Cases 4-5)*

  $L_1 = L_2 = L_3$ = 10.0 m (32.7 ft)
  $L_4 = L_5$ = 7.0 m (23.0 ft)
  $L_6 = L_7$ = 5.0 m (16.4 ft)
  $L_{TOTAL\ NU}$ = 55.4 m (177.3 ft)

The total lengths for the non-uniform and uniform length patterns vary approximately 7 percent from each other (i.e., confirming previous understanding). The difference between the SNAIL-base nail lengths and those calculated with the design charts is within 10 percent. This comparison indicates that the design charts can provide reasonably close agreements with nail lengths computed with software.

c. Maximum Design Nail Forces

The maximum design nail forces are calculated for the most critical failure surface following the procedure described in Section 6.4. The procedure consists of correcting the maximum nail forces T_{max} calculated for a global $FS_G > 1$ by multiplying this term by the ratio of <u>average</u> nail forces $T_{avg\text{-}s}/T_{avg}$. $T_{avg\text{-}s}$ is the average nail force calculated for $Fs_G = 1.0$, and T_{avg} and is the average nail force calculated for $FS_G > 1$. As such, this maximum design nail force takes into account the soil strength full mobilization.

The calculation of the maximum design nail forces corresponding to the most critical failure surfaces was carried out for Cases 2 and 4, as indicated in Table D.4.

Although the average force per nail is quite similar in the two nail patterns, the difference in maximum nail forces between Cases 2 and 4 is significant. This departure can be explained by the dissimilar nail-force distribution with depth in both cases. In Case 2 with uniform lengths, the nail length beyond the critical failure surface increases significantly near the bottom of excavation (Figure D.7) and the nail forces near the bottom are large. As a result of these large nail lengths behind the failure surfaces, the manner in which the maximum design nail forces are mobilized are significantly different from nail to nail. On the other hand, in Case 4 with non-uniform length, the nail lengths beyond the critical failure surface (Figure D.9) are comparable in all nails; therefore, the mobilization of nail forces with depth is more consistent from nail to nail than for Case 2.

The maximum design nail force was calculated with SNAIL as $T_{max\text{-}s}$ = 138 kN (30 kip) for Case 2. This value is approximately 8 percent larger than the value estimated using the design charts for a uniform length pattern. This favorable comparison again indicates that the design charts provide a valuable tool to obtain reasonably good estimates of $T_{max\text{-}s}$ to size the nail bar.

Table D.4: Maximum Design Nail Forces.

Nail #	Symbol	Nail Force in kN (kip)	
		Case 2: Uniform Pattern	Case 4: Non-Uniform Pattern (Static)
1	T_1	0.4 (0.1)	87.1 (19.6)
2	T_2	20.9 (4.7)	100.5 (22.6)
3	T_3	48.8 (10.7)	115.9 (26.1)
4	T_4	76.7 (17.2)	71.7 (16.1)
5	T_5	104.6 (23.5)	102.9 (23.1)
6	T_6	171.8 (38.6)	63.4 (14.3)
7	T_7	240.5 (54.1)	135.0 (30.3)
Maximum Nail Force (FS_G = 1.5)	T_{max}	240.5 (54.1)	135.0 (30.3)
Average Nail Force (FS_G = 1.5)	T_{ave}	94.8 (21.3)	96.6 (21.7)
Average Nail Force (FS_G = 1)(1)	$T_{ave}^{(1)}$	54.2 (12.2)	65.2 (14.7)
Average (FS_G = 1) ÷ Average (FS_G = 1.5)	$\frac{T_{ave-s}}{T_{ave}}$	0.57	0.67
Maximum Design Nail Force (FS_G = 1)	$T_{max\text{-}s} = T_{max} \times \frac{T_{ave-s}}{T_{ave}}$	**138 (31)**	**91 (21)**

Note: (1) Computed by SNAIL.

B. Seismic Considerations (Section 6.4 - Item B)

1. Define seismic loads
   Already given.

2. Horizontal seismic coefficient, k_h
   Given.

3. Seismic global stability analysis
   Results of global safety factors are included in Table D.3.

4. Sliding Stability
   Sliding stability analysis is not performed in this example.

C. Internal Failure Modes (Section 6.4 - Item C)

1. Nail Pullout Resistance
   Intrinsically accounted for in SNAIL.

2. Nail Tensile Resistance

With: $T_{max\text{-}s}$ = 138 kN (31 kip) (uniform pattern)
$T_{max\text{-}s}$ = 91 kN (21 kip) (non uniform pattern)
f_Y = 550 Mpa (75 ksi),

calculate the necessary nail bar sectional area, A_T, as:

$$A_t = \frac{T_{max\text{-}s} \times FS_T}{f_y}$$

$$A_t = \frac{138[\text{kN}] \times 1.8}{0.52\,[\text{kN/mm}^2]} = 478\ \text{mm}^2\ (0.74\ \text{in.}^2)\ \text{for Case 2}$$

$$A_t = \frac{91[\text{kN}] \times 1.8}{0.52\,[\text{kN/mm}^2]} = 315\ \text{mm}^2\ (0.48\,\text{in.}^2)\ \text{for Case 4}$$

The necessary nail bar area computed from the SNAIL analysis for Case 2 is close to the value estimated using the design charts.

3. Select Bar Size

Threaded bars with a cross-sectional area of 510 mm^2 (0.79 $in.^2$) and 25 mm (# 8) in diameter are selected.

D. Facing Design

1. Design nail head tensile force T_o, is estimated as:

$$T_o = T_{max-s}\left[0.6 + 0.2\,(S_v - 1)\right] = \left[0.6 + 0.2\,(1.5 - 1)\right] T_{max-s} = 0.7\ T_{max-s}$$

T_o = 0.7 × 138 = 97 kN (21 kip) for Case 2

T_o = 0.7 × 91 = 64 kN (14 kip) for Case 4

where S_V is the nail vertical spacing.

The safety factors correspond to the potential failure modes of the nail-facing connection including the flexural and punching shear failures. Because a two-phase facing construction is used in this project, flexural and shear-punching failure modes must be evaluated separately for the temporary and the permanent facing. Additionally, for the final facing, a tensile failure of the headed studs is considered.

Using the largest calculated force at the wall facing (i.e., T_o = 0.7 × 138 = 97 kN, for uniform nail pattern) along with the minimum factors of safety listed in Table D.1, the minimum capacity requirements are obtained:

Facing Capacity	Symbol	Static Loads		Seismic Loads	
		FS	Capacity (kN)	FS	Capacity (kN)
Flexure	R_{FF}	1.5	146	1.1	107
Punching Shear	R_{FP}	1.5	146	1.1	107
Headed Stud Tensile	R_{FS}	2.0	194	1.5	146

These necessary facing capacities will be compared against the existing facing capacities. The existing facing capacities are established once the facing design, which is presented in the following section, is adopted.

2. Facing Thickness

The facing system features are shown in Figure D.11 and summarized in Table D.5.

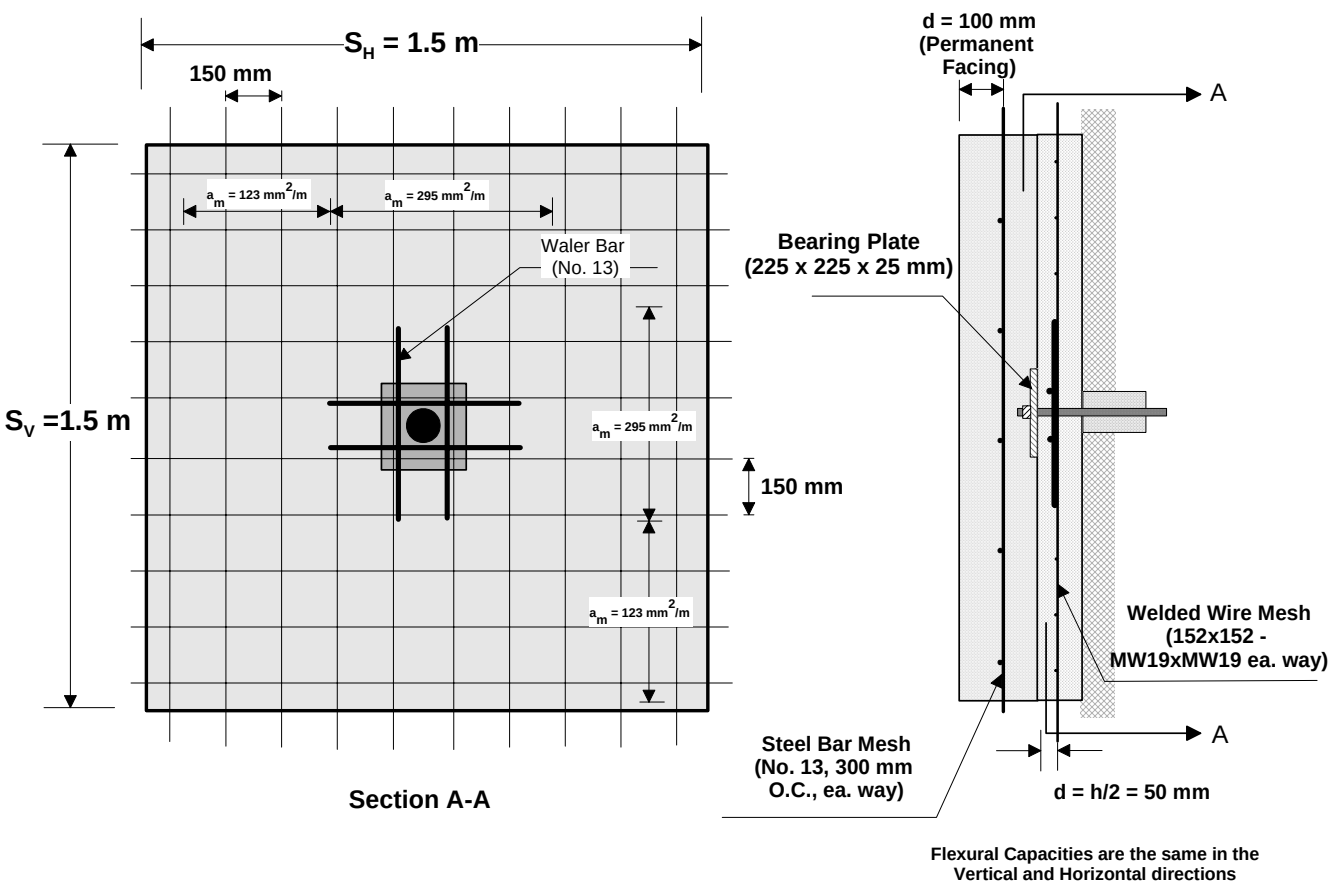


Figure D.11: Reinforcement in Facing.

3. Facing Materials

See Table D.5.

Table D.5: Facing Main Features.

Element	Description	Temporary Facing	Permanent Facing
General	Thickness (h)	100 mm (4 in.)	200 mm (8 in.)
	Facing Type	Shotcrete	CIP Concrete
	Comp. Strength, f'_c	21 MPa (3,000 psi)	28 MPa (4,000 psi)
Reinforcement	Type	WWM	Steel Bars Mesh
	Grade	420 (Grade 60)	420 (Grade 60)
	Denomination	152 × 152 MW 19 × MW 19 (6 × 6 - W2.9 × W2.9)	No. 13 @ 300 mm (each way) [No. 4 @ 12 in. (each way)]
Other Reinf.	Type	Waler Bars 2 × 13 mm (2 × #8)	–
Bearing Plate	Type	4 Headed-Studs $^1/_2 \times 4^1/_8$	–
	Steel	250 MPa (Grade 420)	–
	Dimensions	Length; L_P = 225 mm (9 in.)	–
		Thickness: t_P = 25 mm (1 in.)	–
Headed Studs	Dimensions	–	Nominal Length: L_s = 105 mm (4 in.)
		–	Head Diameter: D_H = 25.4 mm (1 in.)
		–	Shaft Diameter: D_S = 12.7 mm ($^1/_2$ in.)
		–	Head Thickness: t_H = 7.9 mm (0.3 in.)
		–	Spacing: S_{SH} = 150 mm (6 in.)

4. Flexural Resistance, R_{FF}, Verification (Section 6.4 - Item D.4)

Temporary Facing

a. Limiting Reinforcement Ratios

The reinforcement ratio is defined as:

$$\rho = \left(\frac{a_s}{d}\right)$$

where: a_S is the total reinforcement cross-sectional area, d is the distance from the farthest concrete fiber in compression to the centroid of the reinforcement (assumed to be concentrated in the middle of the shotcrete section), Assume d = h/2. For the temporary facing d = 100/2 = 50 mm (2 in.) and for permanent facing d = 200/2 = 100 mm (4 in.).

The minimum and maximum recommended reinforcement ratios are:

$$\rho_{min}[\%] = 0.24\ \frac{\sqrt{f'_c[psi]}}{f_y[ksi]} = 0.24\ \frac{\sqrt{3{,}000}}{75} = 0.18\,\%$$

$$\rho_{max}[\%] = 0.05\ \frac{f'_c[psi]}{f_y[ksi]}\left(\frac{90}{90+f_y[ksi]}\right) = 0.05\ \frac{3{,}000}{75}\left(\frac{90}{90+75}\right) = 1.1\,\%$$

Considering the width of the analysis section, b = 1 m (3.3 ft), the ratio of reinforcement is area per unit length is

$a_S = \rho\ d \times$ width

$a_{S\ MIN} = 0.0018 \times 50$ [mm] x 1.0 [m] = 90 mm^2/m (0.043 in.^{2}/ft)

$a_{S\ MAX} = 0.0110 \times 50$ [mm] x 1.0 [m] = 550 mm^2/m (0.26 in.^{2}/ft)

b. Select Reinforcement

Use a mesh 152 × 152 – MW19 × MW19 (6 × 6 - W2.9 × W2.9 mesh in English units). With Table A.2 the total reinforcement area per unit length at midspan is:

a_{sm} = 123 mm^2/m = 1.23 × 10^{-4} m^2/m (0.058 in.^{2}/ft)

At the nail, also place two No. 13 (# 4) vertical and horizontal waler bars. In both directions, the total nominal area is (using Table A.3):

A_S = 2 × 129 = 258 mm^2 (0.4 in.^{2}).

The additional reinforcement over the nail head could have also accomplished by overlapping a rectangular piece of WWM [with enough development length, (see AASHTO, 1996 or ACI, 2000)] in this location.

c. The total reinforcement area per unit length around the nails is:

$$a_{sn} = a_{sm} + \frac{As}{S_M} =$$

$$a_{sn} = \frac{123 \times 1.5 + 258}{1.5} = 295\ \text{mm}^2/\text{m} = 2.95 \times 10^{-4}\ \text{m}^2/\text{m}\ (0.14\ \text{in.}^2/\text{ft}),$$

In addition, the ratio of the existing reinforcement areas around the nail and midspan is 258/123 = 2.1, which guarantees that the ratio of flexural capacities around the nail and midspan is less than the recommended limiting ratio of 2.5. Because the placed reinforcement is the same in both vertical and horizontal directions and the vertical and horizontal spacing are the same, the total

reinforcement area per unit length calculated above is the same for each direction.

d. Verify Minimum Reinforcement Ratios

a_{sm} = 123 mm^2/m > 90 mm^2/m

a_{sn} = 295 mm^2/m > 90 mm^2/m

which satisfies this limit.

e. Verify Maximum Reinforcement Ratios

a_{sm} = 123 mm^2/m < 550 mm^2/m

a_{sn} = 295 mm^2/m < 550 mm^2/m

which satisfies these limits.

Reinforcement Details

Reinforcement details should follow AASHTO (1996) specifications or ACI (2000) and should include:

- Minimum cover = 50 mm (2 in.)
- Appropriate development lengths
- Appropriate splice locations and specific lengths

f. Select Factor C_F

Use C_F = 2 for temporary facing from Table 5.1.

g. Flexural Capacity

The facing resistance for flexure is estimated using the simplified formulas from Section 5.4.1 as follows:

$$R_{FF} = \frac{C_F}{265} \times (a_{sn} + a_{sm})[\text{mm}^2/\text{m}] \times \left(\frac{S_h}{S_v}\right) \times h[\text{m}] \times f_y[\text{MPa}] =$$

$$R_{FF} = \frac{2}{265} \times (295 + 123) \times 0.10 \times 420 = 132 \text{ kN (29 kip)}$$

This value could have also been computed from Table 6.4. With the total reinforcement ratio:

$$\rho_{TOT} = \frac{(295 + 123)}{1{,}000 \times 50} \times 100 = 0.84\ \%$$

the facing resistance for flexure results in R_{FF} = 129 kN, which is practically the same result.

h. Calculate ultimate loads, $FS_{FF} \times T_o$ and compare with flexural capacity, R_{FF}

For the temporary facing, FS_{FF} = 1.35

Design load = 1.35 × 97 = 131 kN (29 kips) (Case 2)

Design load = 1.35 × 64 = 86 kN (19 kips) (Case 4)

Remaining cases must also be verified.

As the design loads are always smaller than R_{FF}, there is no need to redesign.

Permanent Facing

a. Limits of Reinforcement Ratios

$$\rho_{min}[\%] = 0.24 \frac{\sqrt{f_c'[psi]}}{f_y[ksi]} = 0.24 \frac{\sqrt{4{,}000}}{75} = 0.21\%$$

$$\rho_{max}[\%] = 0.05 \frac{f_c'[psi]}{f_y[ksi]} \left(\frac{90}{90 + f_y[ksi]} \right) = 0.05 \frac{4{,}000}{75} \left(\frac{90}{90+75} \right) = 1.46\%$$

Considering the width of the analysis section, b = 1 m (3.3 ft), the ratio of reinforcement is area per unit length is

$a_S = \rho\ d \times$ width

$a_{S\ MIN}$ = 0.0021 × 100 [mm] x 1.0 [m] = 210 mm^2/m (0.043 $in.^2$/ft)

$a_{S\ MAX}$ = 0.0146 × 100 [mm] x 1.0 [m] = 1460 mm^2/m (0.26 $in.^2$/ft)

b. Select Reinforcement

Use a reinforcement mesh made of No. 16 metric bars (#5 in English units) at 300 mm (12 in.) center-to-center each way. No waler bars are used. The total reinforcement area per unit length at midspan and around the nails is:

$a_{sm} = a_{sn}$ = 199 × 1,000/300 = 663 mm^2/m (0.31 $in.^2$/ft)

c. Reinforcement Ratios

The reinforcement ratios along each direction and the total reinforcement ratio are:

$$\rho_{hm} = \rho_{hn} = \rho_{vm} = \rho_{vn} = \frac{663}{1{,}000 \times 100} \times 100 = 0.66\ \%$$

$$\rho_{TOT} = 1.33\ \%$$

d. Select Factor C_F

For permanent facing use $C_F = 1$ (from Table 5.1)

Because the same reinforcement is used in both directions and throughout the permanent facing, the flexural capacity per unit length at midspan and at nails is:

$$R_{FF} = \frac{1}{265} \times 1326 \times 0.2 \times 420 = 419 \text{ kN (93 kips)}$$

With the total reinforcement ratio ρ_{TOT} = 1.33 percent, the facing resistance for flexure can be also estimated with Table 6.4, which gives R_{FF} = 421 kN (92 kips), same result as that with formulas

e. Calculate ultimate loads and compare with flexural capacity

With FS_{FF} = 1.5 for the permanent facing,

Design load = 1.5 × 97 = 146 kN (32 kips) (Case 2)

Design load = 1.5 × 64 = 96 kN (21 kips) (Case 4)

The flexural capacity of the permanent facing R_{FF} = 421 kN (92 kips), the capacity is verified. There is no need to redesign.

5. Punching Shear Resistance (R_{FP})

a. Calculate R_{FP} for temporary facing

The punching shear failure consists of the failure of a truncated cone of mean diameter $D'_C = L_P + h$. The resisting shear force on this cone, V_F, is calculated as:

$$V_F = 330\ \sqrt{f'_c[\text{MPa}]}\ \pi\ D'_c\,[\text{m}]\,h\,[\text{m}]$$

For the problem conditions, V_F results:

$$V_F = 330 \times \sqrt{21} \times \pi \times (0.225 + 0.10) \times 0.10 = 154 \text{ kN (34 kip)}$$

The resistance against punching shear failure is:

$$R_{FP} = C_P V_F,$$

where C_P is assumed conservatively equal to unity, thus:

$R_{FP} = 154$ kN (34 kip).

This value could have also been computed from Table 6.4. For h = 0.1 m (4 in.), f'_c = 21 MPa (3,000 psi), and L_P = 225 mm (9 in.), Table 6.4 gives R_{FP} = 150 kN (35 kip), which is the same value calculated with formulas.

b. Calculate R_{FP} for permanent facing

D'_C is defined for permanent facings as D'_C = minimum of $S_{HS} + h_C$ or $2h_C$, where S_{HS} is the headed-stud separation and h_C the effective headed-stud length. For the problem conditions:

$h_C = L_S + t_P - t_{SH}$ = 105 + 19.1 –7.9 = 116 mm (4.6 in.), and S_{HS} = 150 mm (6 in.)

D'_C = min (150 + 116 or 2 × 116) = 232 mm (9.1 in.)

Therefore, the resisting shear force is:

$$V_F = 330 \times \sqrt{28} \times \pi \times 0.23 \times 0.116 = 146 \text{ kN (32 kip)}$$

The resistance against punching shear failure is:

$$R_{FP} = C_P V_F$$

where C_P is assumed conservatively equal to unity, thus:

$$R_{FP} = 146 \text{ kN}$$

Table 6.4 gives, for h_C = 116 mm (4.6 in.), f'_c = 28 Mpa (4,000 psi), and S_S = 150 mm (6 in.), a shear-punching resistance of R_{FP} = 148 kN (33 kips).

6. Headed-Stud Tensile Resistance (R_{FS})

Calculate the nail headed-stud tensile capacity

$R_{FS} = 4\ A_H\ f_Y = 4 \times (\pi\ 12.7^2/4) \times 0.420$ = 213 kN (47 kips). Table 6.4 provides essentially the same value.

A summary of the facing capacities and verification is presented in Table D.6. The lowest value of the facing failure mechanisms is the flexure capacity of the construction shotcrete facing. This is the typical case of most soil nail walls when a 100-mm (4-in.) thick initial facing is used. However, when the long-term distributed load acts behind the wall, the punching shear capacity of the permanent facing controls. In all cases, the existing facing capacities exceed the criteria established by the minimum recommended factors of safety.

Table D.6: Summary and Verification of Facing Capacities.

Failure Mode	Symbol	Facing	Design Loads ($T_o \times FS$) (1)			Remarks
			Static Loads	Seismic Loads	Facing Capacity (kN)	
Flexure	R_{FF}	Temp.	131	107	132	**OK**
		Perm.	146		419	**OK**
Punching Shear	R_{FP}	Temp.	131		154	**OK**
		Perm.	146		146	**OK**
Headed Stud Tensile	R_{FS}	Perm.	194	146	213	**OK**

Note: (1) Corresponds to Case 2.

STEP 4: DEFORMATION ESTIMATES (Section 6.5)

Construction-induced wall deflections can be estimated from existing correlations (Clouterre, 1991) presented in Figure 5.23. For a vertical soil nail wall with sandy soil behind, it is expected that the maximum vertical and horizontal permanent deflections at the top of the wall (δ_h, δ_v, respectively) will be approximately:

$$\delta_h = \delta_v = H / 500 = 10 \times 1{,}000 /500 = 20 \text{ mm (4 in.)}$$

It is estimated that these deflections are within tolerable limits for this type of noncritical structure. The wall deformation is expected to have some influence within a distance D_{DEF} behind the wall. This distance of influence is estimated to be (Figure 5.23):

$$D_{DEF} = C\ H\ (1- \tan \alpha) = 0.8\ H = 8 \text{ m (26 ft)}$$

Because future structures will be placed at least 10 m (33 ft) behind the wall, the potential detrimental effect of the wall lateral deflections is not considered significant in this area.

It is expected that the deflections will increase to their maximum values over a period time, probably weeks or up to a few months after nail installation.

The long-term performance regarding lateral deflections will be better for the non uniform-nail-length pattern.

If consistent with local practice, or if there is any indication of past poor performance of slopes and other retaining structures in the area, it may be advisable to specify a monitoring control system during and after construction.

STEP 5: OTHER DESIGN CONSIDERATIONS

A. Drainage

The low groundwater table at the site indicates that special considerations of internal drainage are not warranted. Internal drainage is limited to the installation of geocomposite drain strips, weepholes, and toe drains, as specified. Details of the surface control are left to the contractor and for review by the engineer.

B. Other Performance Verification

Other performance criteria include durability and aesthetics. No significant exposure to aggressivity is expected at the site. Additionally, the CIP facing will have conventional cement content and water/cement ratios, so this facing system is expected to provide sufficient durability.

Aesthetic requirements called for a CIP concrete permanent facing as a minimum. Therefore, this criterion is also met.

SUMMARY OF DESIGN

Nails

Element	Description	Values
Nail Pattern	Square	-
Nail Spacing	Vertical, S_V	1.5 m (5 ft)
	Horizontal, S_H	1.5 m (5 ft)
Nail Inclination	Not uniform	20° (first nail), 15° (remaining)
Nail Number	Per section	7
Nail Length	Uniform Pattern	L = 7.0 m (23 ft)
	Non-Uniform Pattern	L_1, L_2, L_3=10.0 m (32.7 ft), L_4, L_5=7.0m (23.0 ft), L_6 = L_7 = 5.0 m (16.4 ft)
Nail Bar	Type	Threaded No. 25 mm (No. 8)
	Material	Steel Grade 520 (75 ksi)
Drillhole	Minimum Diameter	150 mm (6 in.)
Corrosion Protection	Grout-protected nail bar	Class II Protection
	Minimum Cover	50 mm (2 in.)
	PVC Centralizers	as specified
Grout	Neat Cement	minimum f'_c = 21 MPa (3,000 psi) (at 28 days)
Ultimate Bond Strength	Minimum specified	$Q_u = \pi \times 0.15 \times 100$ = 47 kN/m (3.3 kips/ft)

Facing

Element	Description	Temporary Facing	Permanent Facing
General	Thickness (h)	100 mm (4 in.)	200 mm (8 in.)
	Facing Type	Shotcrete	CIP Concrete
	Comp. Strength, f'_c	21 MPa (3,000 psi)	28 MPa (4,000 psi)
Reinforcement	Type	WWM	Steel Bar Mesh
	Grade	420 MPa (Grade 60)	420 MPa (Grade 60)
	Denomination	152 × 152 MW 19 × MW 19 (6 × 6 - W2.9 × W2.9)	No. 13 @ 300 each way (No. 4 @ 12 in. ea. way)
Other Reinforcement	Type	Waler Bars 2 × 13 mm (2 × #8)	-
Bearing Plate	Type		4 Headed-Studs $^1/_2 \times 4^1/_8$
	Steel	250 MPa (Grade 420)	–
	Dimensions	Length; L_{BP} = 225 mm (9 in.)	–
		Thickness: t_P = 25 mm (1 in.)	–
Headed Studs	Dimensions	–	Nominal Length: L_s = 105 mm (4 in.)
		–	Head Diameter: D_H = 25.4 mm (1 in.)
		–	Shaft Diameter: D_S = 12.7 mm ($^1/_2$ in.)
		–	Head Thickness: t_H = 7.9 mm (0.3 in.)
		–	Spacing: S_{HS} = 150 mm (6 in.)

Internal Drainage

Geocomposite drain strips, weepholes, and toe drains, as specified.

CONCLUSIONS

The available soil information indicates that the design of the soil nail wall for the project requirements indicated is feasible. Materials commonly used in soil nail walls, as well as typical dimensions, were used in the design. These conditions met all the design criteria for stability and strength established for this type of noncritical structure. The level of corrosion protection necessary is not particularly demanding.

APPENDIX E

CONSTRUCTION SPECIFICATIONS FOR SOIL NAIL WALLS

This appendix presents the following specifications:

Procedural Specifications for Soil Nail Walls E-2
Performance Specifications for Soil Nail Walls E-17
Shotcrete Specifications for Soil Nail Walls E-19
Geotechnical Instrumentation for Soil Nail Walls E-28

The specifications contain sufficient information to construct soil nail walls according to the current U.S. practice. The specifications provide commentary to assist the design engineer in making appropriate modifications for project-specific conditions. For materials used in soil nail walls, the reader is referred to other sources including AASHTO (1996), and FHWA (2002) and ACI references on shotcrete [e.g., ACI (1994), ACI (1995a)]. For testing of materials, the reader is referred to AASHTO (1992) and ASTM (2002).

SOIL NAIL WALL

PROCEDURAL SPECIFICATIONS

PART 1 GENERAL

1.01 SECTION INCLUDES

A. Scope of Work
B. Material
C. References to other specifications

[Section 1.01 may not be necessary if Federal Highway Specifications 2002 (FHWA, 2002) are used.]
(Project-specific needs may require different type of facing including reinforced shotcrete, cast-in-place concrete, and precast concrete panels.)

1.02 SCOPE OF WORK

A. This work consists of constructing a permanent soil nailed wall as specified herein and as shown on the plans. The Contractor shall furnish all labor, materials, and equipment required to complete the work. The Contractor shall select the excavation, drilling, and grouting methods and **the diameter of the drillholes to meet the performance requirements specified herein or shown on the plans.**

B. The work shall include excavating in staged lifts in accordance with the approved Contractor's plan; detailing the drilling of the soil nail drillholes to the diameter and length required to develop the specified capacity; grouting the nails; providing and installing the specified drainage features; providing and installing bearing plates, washers, nuts, and other required miscellaneous materials; and constructing the required temporary shotcrete face and constructing the final structural facing.

(Project-specific needs may require different type of facing including reinforced shotcrete, cast-in-place concrete, and precast concrete panels.)

1.03 MATERIAL

A. Material shall conform to the following sections and subsection *(use as required)*.

1. CIP concrete [*see AASHTO (1996), Section 8 "Concrete Structures"]*.
2. Reinforcing Steel [*see AASHTO (1996), Section 9 "Reinforcing Steel"]*.
3. Permanent Shotcrete *(see specification in this appendix)*.

4. Forms and Falsework (*use agency-standard specifications*).
5. Architectural Finish (*use agency-standard specifications)*.

B. Soil Nails

1. *Nail Solid Bar.* AASHTO M31/ASTM A615, Grade 420 or 520, ASTM A 722 for Grade 1035. Deformed bar, continuous without splices or welds, new, straight, undamaged, bare, or epoxy-coated, or encapsulated as shown on the Plans. Threaded, a minimum of 150 mm (6 in.) on the wall anchorage end, to allow proper attachment of bearing plate and nut. Threading may be continuous spiral deformed ribbing provided by the bar deformations (continuous thread bars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, provide the next-larger bar number designation from that is shown on the Plans, at no additional cost.

2. *Bar Coupler.* Bar couplers shall develop the full ultimate tensile strength of the bar as certified by the manufacturer.

3. *Fusion Bonded Epoxy Coating.* ASTM A 775. Minimum 0.4 mm (0.016 in.) thickness electrostatically applied. Bend test requirements are waived. Coating at the wall anchorage end of epoxy-coated bars may be omitted over the length provided for threading the nut against the bearing plate.

4. *Encapsulation.* Minimum 1-mm (0.04-in.) thick, corrugated, HDPE tube conforming to AASHTO M252 or corrugated PVC tube conforming to ASTM D1784, Class 13464-B.

C. Soil Nail Appurtenances

1. *Centralizer.* Manufactured from Schedule 40 PVC pipe or tube, steel, or other material not detrimental to the nail steel (wood shall not be used); securely attached to the nail bar; sized to position the nail bar within 25 mm (1 in.) of the center of the drillhole; sized to allow tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole.

2. *Nail Grout.* Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 10.5 MPa (1,500 psi) and a minimum 28-day compressive strength of 21 MPa (3,000 psi), per AASHTO T106/ASTM C109.

3. *Fine Aggregate.* AASHTO M6/ASTM C33.

4. *Portland Cement.* AASHTO M85/ASTM C150, Type I, II, III, or V.

5. *Admixtures.* AASHTO M194/ASTM C494. Admixtures that control bleed, improve flowability, reduce water content, and retard set may be used in the grout subject to review and acceptance by the Engineer. Accelerators are not permitted. Expansive admixtures may only be used in grout used for filling sealed encapsulations. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations.

6. *Film Protection.* Polyethylene film per AASHTO M171.

D. Bearing Plates, Nuts, and Welded Stud Shear Connectors.

1. *Bearing Plates.* AASHTO M183/ASTM A36.

2. *Nuts*. AASHTO M291, grade B, hexagonal, fitted with beveled washer or spherical seat to provide uniform bearing.

3. *Shear Connectors*. AASHTO Construction Specifications, Section 11.3.3.1.

E. Welded Wire Fabric. AASHTO M55/ASTM A185 or A497.

F. Reinforcing Steel. AASHTO M31/ASTM A615, Grade 420, deformed.

G. Geocomposite Sheet Drain. Manufactured with a drainage core (e.g., geonet) and a drainage geotextile attached to or encapsulating the core. Drainage core to be manufactured from long chain synthetic polymers composed of at least 85 percent by mass of polypropylenes, polyester, polyamine, polyvinyl chloride, polyoleofin, or polystyrene and having a minimum compressive strength of 275 kPa (40 psi) when tested in accordance with ASTM D 1621 Procedure A. The drainage core with the geotextile fully encapsulating the core shall have a minimum flow rate of 1 liter per second per meter of width tested in accordance with ASTM D 4716. The test conditions shall be under an applied load of 69 kPa (10 psi) at a gradient of 1.0 after a 100-hour seating period.

H. Underdrain and Perforated Pipe

1. *Pipe*. ASTM 1785 Schedule 40 PVC solid and perforated wall; cell classification 12454-B or 12354-C, wall thickness SDR 35, with solvent weld or elastomeric joints.

2. *Fittings*. ASTM D3034, Cell classification 12454-B or C, wall thickness SDR 35, with solvent or elastomeric joints.

I. Temporary Shotcrete. Submit for approval, all materials, methods, and control procedures for this work.

1.04 CONTRACTOR QUALIFICATIONS

A. The soil nailing contractor shall have completed at least 3 permanent soil nail retaining wall projects during the past 3 years totaling at least 1,000 m^2 (10,000 ft^2) of wall face area and at least 500 permanent soil nails.

B. Provide a Registered Professional Engineer with experience in the construction of permanent soil nail retaining walls on at least 3 completed projects over the past 3 years. The Contractor may not use consultants or manufacturer's representatives to meet the requirements of this section. Provide on-site supervisors and drill operators with experience installing permanent soil nails on at least three projects over the past 3 years.

1.05 SUBMITTALS

A. The Contractor shall submit a brief description of at least 3 projects, including the owning agency's name, address, and current phone number; location of project; project contract value; and scheduled completion date and actual completion date for the project.

B. At least 60 calendar days before starting soil nail work, identify the Engineer, on-site supervisors, and drill operators assigned to the project, and submit a summary of each individual's experience. Only those individuals designated as meeting the qualifications requirements shall be used for the project. The Contractor cannot substitute for any of these individuals without written approval of the Owner or the Owner's Engineer. The Owner's Engineer shall approve or reject the Contractor qualifications and staff within 15 working days after receipt of the submission. Work shall not be started on any soil nail wall nor materials ordered until the Contractor's qualifications have been approved by the Owner's Engineer. The Engineer may suspend the work if the Contractor substitutes unqualified personnel for approved personnel during construction. If work is suspended due to the substitution of unqualified personnel, the Contractor shall be fully liable for additional costs resulting from the suspension of work and no adjustment in contract time resulting from the suspension of the work will be allowed.

C. The Contractor is responsible for providing the necessary survey and alignment control during the excavation for each lift, locating drillholes and verifying limits of wall installation. At least 30 days before starting soil nail work, submit a Construction Plan to the Engineer that includes the following.

1. The start date and proposed detailed wall construction sequence.

2. Drilling and grouting methods and equipment, including the ***drillhole diameter proposed to achieve the specified pullout resistance values shown on the plans*** and any variation of these along the wall alignment.

3. Nail grout mix design, including compressive strength test results (per AASHTO T106/ASTM C109) supplied by a qualified independent testing lab verifying the specified minimum 3-day and 28-day grout compressive strengths. Previous test results for the same grout mix completed within one year of the start of grouting may be submitted for verification of the required compressive strengths.

4. Nail grout placement procedures and equipment.

5. Temporary shotcrete materials and methods.

6. Soil nail testing methods and equipment setup.

7. Identification number and certified calibration records for each test jack and pressure gauge and load cell to be used. Jack and pressure gauge shall be calibrated as a unit. Calibration records shall include the date tested, the device identification number, and the calibration test results and shall be certified for an accuracy of at least 2 percent of the applied certification loads by a qualified independent testing laboratory within 90 days prior to submittal.

8. Manufacturer Certificates of Compliance for the soil nail ultimate strength, nail bar steel, Portland cement, centralizers, bearing plates, epoxy coating, and encapsulation.

D. The Engineer shall approve or reject the Contractor's Construction Plan within 30 working days after the submission. Approval of the Construction Plan does not relieve the Contractor of his responsibility for the successful completion of the work.

1.06 STORAGE AND HANDLING

A. Store and handle soil nail bars in a manner to avoid damage or corrosion. Replace bars exhibiting abrasions, cuts, welds, weld splatter, corrosion, or pitting. Repair or replace any bars exhibiting damage to encapsulation or epoxy coating. Repaired epoxy coating areas shall have a minimum 0.3-mm (0.012-in.) thick coating.

1.07 EXCAVATION

A. The height of exposed unsupported final excavation face cut shall not exceed the vertical nail spacing plus the required reinforcing lap or the short-term stand-up height of the ground, whichever is less. Complete excavation to the final wall excavation line and apply shotcrete in the same work shift, unless otherwise approved by the Engineer. Application of the shotcrete may be delayed up to 24 hours if the contractor can demonstrate that the delay will not adversely affect the excavation face stability.

B. Excavation of the next-lower lift shall not proceed until nail installation, reinforced shotcrete placement, attachment of bearing plates and nuts, and nail testing have been completed and accepted in the current lift. Nail grout and shotcrete shall have cured for at least 72 hours or attained at least their specified 3-day compressive strength before excavating the next underlying lift.

1.08 NAIL INSTALLATION

A. Provide nail length and drillhole diameter necessary to develop the load capacity to satisfy the acceptance criteria for the design load required, **but not less than the lengths or diameters shown in the plans.** Drill holes for the soil nails at the locations, elevations, orientations, and lengths shown on the Plans. Select drilling equipment and methods suitable for the ground conditions and in accordance with the accepted installation methods submitted by the Contractor. The use of drilling muds or other fluids to remove cuttings will not be allowed. If caving ground is encountered, use cased drilling methods to support the sides of the drillholes. **[The use of self-drilling nail bars (also known as hollow, self-grouting or pressure-grouted nail bars) will not be allowed.]** Provide nail bars as shown in the Plans. Provide centralizers sized to position the bar within 25 mm (1 in.) of the center of the drillhole. Position centralizers as shown on the Plans so that their maximum center-to-center spacing does not exceed 2.5 m (8.2 ft). Also locate centralizers within 0.5 m (1.5 ft) from the top and bottom of the drillhole.

1.09 GROUTING

A. Grout the drillhole after installation of the nail bar and within 2 hours of completion of drilling. Inject the grout at the lowest point of each drillhole through a grout tube, casing, hollow-stem auger, or drill rods. Keep the outlet end of the conduit delivering grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. Completely fill the drillhole in one continuous operation. Cold joints in the grout column are not allowed except at the top of the test bond length of proof tested production nails.

B. Test nail grout according to AASHTO T106/ASTM C109 at a frequency of one test per mix design and a minimum of one test for every 40 m^3 (52 cy) of grout placed. Provide grout cube test results to the Engineer within 24 hours of testing.

1.10 NAIL TESTING

A. Perform both verification and proof testing of designated test nails. Perform verification tests on sacrificial test nails at locations shown on the Plans. Perform proof tests on production nails at locations selected by the Engineer. Testing of any nail shall not be performed until the nail grout and shotcrete facing have cured for at least 72 hours or attained at least their specified 3-day compressive strength.

B. Testing equipment shall include 2 dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The pressure gauge shall be graduated in 500 kPa (75 psi) increments or less. Measure the nail head movement with a minimum of 2 dial gauges capable of measuring to 0.025 mm (0.001 in.).

1.11 VERIFICATION TESTING OF SACRIFICIAL NAILS

A. Perform verification testing prior to installation of production nails to confirm the appropriateness of the Contractor's drilling and installation methods, and verify the required nail pullout resistance.

B. Verification test nails shall have both bonded and unbonded lengths. Along the unbonded length, the nail bar is not grouted. The unbonded length of the test nails shall be at least 1 m (3 ft). The bonded length of the soil nail during verification tests, L_{BVT}, shall be at least 3 m (10 ft) but not longer than a maximum length, $L_{BVT\ max}$, such that the nail load does nor exceed 90 percent of the nail bar tensile allowable load during the verification test. Therefore, the following requirements shall be met:

$$L_{BVT} \le \begin{cases} 3\,\text{m}\ (10\,\text{ft}) \\ L_{BVT\,max} \end{cases}$$

The length $L_{BVT\ max}$ is defined as:

$$L_{BVT\,max} = \frac{C_{RT} \times A_t \times f_Y}{Q_{ALL} \times FS_{T\,ver}}$$

where,

C_{RT} = Reduction coefficient. Use C_{RT} = 0.9 for 420 and 520 MPa (Grade 60 and 75) bars. *If 1,035 MPa (Grade 150) bars are allowed in the job, use C_{RT} = 0.8*;

A_t = Nail bar cross-sectional area;
f_Y = Nail bar yield tensile strength;
Q_{ALL} = Allowable pullout resistance per unit length ($Q_{ALL} = Q_u/FS_P$), as specified herein or in plans; and
FS_{Tver} = Factor of safety against tensile failure during verification tests (use 2.5 or, preferably, 3).

The maximum bonded length shall be preferably based on production nail maximum bar grade. Provide larger bar sizes, if required, to meet the 3-m (10-ft) minimum test bonded length requirement at no additional cost.

The Design Test Load (DTL) shall be determined as follows:

$$DTL = L_{BVT} \times Q_{ALL}$$

DTL shall be calculated based on as-built bonded lengths.

C. Perform verification tests by incrementally loading the verification test nails to failure or a maximum test load of 300 percent of the DTL in accordance with the following loading schedule. Record the soil nail movements at each load increment.

Verification Test Loading Schedule.

Load	Hold Time
0.05 DTL max.(AL)	1 minute
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL (Creep Test)	60 minutes
1.75 DTL	10 minutes
2.00 DTL	10 minutes
2.50 DTL	10 minutes max.
3.0 DTL or Failure	10 minutes max.
0.05 DTL max. (AL)	1 minute (record permanent set)

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the DTL. Dial gauges should be set to "zero" after the alignment load has been applied. Following application of the maximum load (3.0 DTL) reduce the load to the alignment load (0.05 DTL maximum) and record the permanent set.

D. Hold each load increment for at least 10 minutes. Monitor the verification test nail for creep at the 1.50 DTL load increment. Measure and record nail movements during the creep portion of the test in increments of 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. Maintain the load during the creep test within 2 percent of the intended load by use of the load cell.

1.12 PROOF TESTING OF PRODUCTION NAILS

A. Perform successful proof testing on 5 percent of the production soil nails in each nail row or a minimum of 1 per row. The Engineer shall determine the locations and number of proof tests prior to nail installation in each row. Production proof test nails shall have both bonded and temporary unbonded lengths. The temporary unbonded length of the test nail shall be at least 1 m (3 ft). The bonded length of the soil nail during proof production tests, L_{BPT}, shall be the least of 3 m (10 ft) and a maximum length, $L_{BPT\ max}$, such that the nail load does nor exceed 90 percent of an allowable value of the nail bar tensile load during the proof production test. Therefore, the following requirements shall be met:

$$L_{BPT} \leq \begin{cases} 3\,\text{m}\ (10\,\text{ft}) \\ L_{BPT\,max} \end{cases}$$

The length $L_{BPT\,max}$ is defined as:

$$L_{BPT\,max} = \frac{C_R \times A_t \times f_Y}{Q_{ALL} \times FS_{T\,proof}}$$

where,

C_{RT} = Reduction coefficient. Use 0.9 for 420 and 520 MPa (Grade 60 and 75) bars. *If 1,035 MPa (Grade 150) bars are allowed in the job, use C_{RT} = 0.8*;

A_t = Nail bar cross-sectional area;

f_Y = Nail bar yield tensile strength;

Q_{ALL} = Allowable pullout resistance per unit length ($Q_{ALL} = Q_u/FS_P$), as specified herein or in plans; and

FS_{Tproof} = Factor of safety against tensile failure during proof production tests (use 1.5.

The maximum bonded length shall be based on production nail maximum bar grade. Production proof test nails shorter than 4 m (12 ft) in length may be constructed with less than the minimum 3-m (10-ft) bond length.

The Design Test Load (DTL) shall be determined as follows:

$$DTL = L_{BPT} \times Q_{ALL}$$

DTL shall be calculated based on as-built bonded lengths.

B. Perform proof tests by incrementally loading the proof test nail to 150 percent of the DTL in accordance with the following loading schedule. Record the soil nail movements at each load increment.

Proof Test Loading Schedule.

Load	Hold Time
0.05 DTL max. (AL)	Until Movement Stabilizes
0.25 DTL	Until Movement Stabilizes
0.50 DTL	Until Movement Stabilizes
0.75 DTL	Until Movement Stabilizes
1.00 DTL	Until Movement Stabilizes
1.25 DTL	Until Movement Stabilizes
1.50 DTL (Max. Test Load)	Creep Test (see below)

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the DTL. Dial gauges should be set to "zero" after the alignment load has been applied.

C. The creep period shall start as soon as the maximum test load (1.50 DTL) is applied and the nail movement shall be measured and recorded at 1 minute, 2, 3, 5, 6, and 10 minutes. Where the nail movement between 1 minute and 10 minutes exceeds 1 mm (0.04 in.), maintain the maximum test load for an additional 50 minutes and record movements at 20 minutes, 30, 50, and 60 minutes. Maintain all load increments within 5 percent of the intended load.

1.13 TEST NAIL ACCEPTANCE CRITERIA

A. A test nail shall be considered acceptable when all of the following criteria are met:

1. For verification tests, the total creep movement is less than 2 mm (0.08 in.) between the 6- and 60-minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

2. For proof tests, the total creep movement is less than 1 mm (0.04 in.) during the 10-minute readings or the total creep movement is less than 2 mm (0.08 in.)

during the 60-minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

3. For verification and proof tests, the total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length.

4. A pullout failure does not occur at 3.0 DTL under verification testing and 1.5 DTL test load under proof testing. Pullout failure is defined as the inability to further increase the test load while there is continued pullout movement of the test nail. Record the pullout failure load as part of the test data.

B. Maintaining stability of the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails cannot be satisfactorily grouted subsequent to testing; the proof test nail shall become sacrificial and shall be replaced with an additional production nail installed at no additional cost to the owner.

1.14 TEST NAIL REJECTION

A. If a test nail does not satisfy the acceptance criterion:

1. For verification test nails, the Engineer will evaluate the results of each verification test. Installation methods that do not satisfy the nail testing requirements shall be rejected. The Contractor shall propose alternative methods and install replacement verification test nails. Replacement test nails shall be installed and tested at no additional cost.

2. For proof test nails, the Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test nail and the adjacent passing proof test nail. Alternatively, the Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient load carrying capacity. Installation and testing of additional proof test nails or installation of additional or modified nails as a result of proof test nail failure(s) will be at no additional cost.

1.15 WALL DRAINAGE NETWORK

A. Install and secure all elements of the wall drainage network as shown on the Plans. The drainage network shall consist of installing geocomposite drain strips, PVC connection pipes, wall footing drains, and weepholes as shown on the Plans.

Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting.

1. *Geocomposite Drain Strips.* Install geocomposite drain strips centered between the columns of nails as shown on the Plans. The drain strips shall be at least 300 mm (12 in.) wide and placed with the geotextile side against the ground. Secure the strips to the excavation face and prevent shotcrete from contaminating the geotextile. Drain strips will be vertically continuous. Make splices with a 300 mm (12 in.) minimum overlap such that the flow of water is not impeded. Install drain plate and connector pipe at base of each strip. Repair damage to the geocomposite drain strip, which may interrupt the flow of water.

2. *Footing Drains.* Install footing drains at the bottom of each wall as shown on the Plans. The drainage geotextile shall envelope the footing drain aggregate and pipe and conform to the dimensions of the trench. Overlap the drainage geotextile on top of the drainage aggregate as shown on the Plans. Replace or repair damaged or defective drainage geotextile.

1.16 SHOTCRETE FACING

A. Provide construction shotcrete facing and permanent shotcrete facing (if required) in accordance with _____. *(Refer to the permanent shotcrete specification that is provided subsequently in this Appendix*). Where shotcrete is used to complete the top ungrouted zone of the nail drill hole near the face, position the nozzle into the mouth of the drill hole to completely fill the void.

1. *Final Face Finish.* Shotcrete finish shall be either an undisturbed gun finish as applied from the nozzle or a rod, broom, wood float, rubber float, steel trowel or rough screeded finish as shown on the Plans.

2. *Attachment of Nail Head Bearing Plate and Nut.* Attach a bearing plate, washers, and nut to each nail head as shown on the Plans. While the shotcrete construction facing is still plastic and before its initial set, uniformly seat the plate on the shotcrete by hand-wrench tightening the nut. Where uniform contact between the plate and the shotcrete cannot be provided, set the plate in a bed of grout. After grout has set for 24 hours, hand-wrench tighten the nut. Ensure bearing plates with headed studs are located within the tolerances shown on the Plans.

3. *Shotcrete Facing Tolerances.* Construction tolerances for the shotcrete facing from plan location and plan dimensions are as follows:

Horizontal location of welded wire mesh; reinforcing bars, and headed studs:	10 mm (0.4 in.)
Location of headed studs on bearing plate:	6 mm (¼ in.)
Spacing between reinforcing bars:	25 mm (1 in.)
Reinforcing lap, from specified dimension:	25mm (1 in.)
Complete thickness of shotcrete:	
If troweled or screeded:	15 mm (0.6 in.)
If left as shot:	30 mm (1.2 in.)
Planeness of finish face surface-gap under 3-m (10-ft) straightedge:	
If troweled or screeded:	15 mm (0.6 in.)
If left as shot:	30 mm (1.2 in.)
Nail head bearing plate deviation from parallel to wall face:	10 degrees

1.17 FORMS AND FALSEWORK

A. Prepare and submit forms and falsework drawings according to Section _____ *(use Agency standard specifications*).

1.18 REINFORCING STEEL

A. Submit all order lists and bending diagrams, fabricate reinforcing steel, ship and protect material, place, fasten, and splice reinforcing steel according to Section(s) _____. *(Agency standard specifications sections and subsection, if available, should be substituted).*

1.19 STRUCTURAL CONCRETE

A. Design concrete mixture, store, handle, batch, and mix material and deliver concrete, provide quality control, and construct concrete facing according to Section(s) _____. *(Agency standard specifications sections and subsection, if available, should be substituted).*

1.20 ARCHITECTURAL SURFACE FINISHES

A. Design and furnish textured form liners, install form liners, and apply a surface finish (color/stain application) that will duplicate the pattern shown on the plans. Submit detailed drawings of the form liner for approval by the Engineer at least

7 days before form liner work begins. Before production work begins, construct a 1-m (3-ft) high, by 0.5-m (1.5-ft) wide, by 3-m (10-ft) long test panel on site using the same forming methods, procedures, form liner, texture configuration, expansion joint, concrete mixture and color/stain application proposed for the production work.

1.21 BACKFILLING BEHIND WALL FACING UPPER CANTILEVER

A. Compact backfill within 1 m (3 ft) behind the wall facing upper cantilever using light mechanical tampers.

B. Backfill shall be relatively free draining granular material.

1.22 ACCEPTANCE

A. Material for the soil nail retaining wall will be accepted based on the manufacturer production certification or from production records. Construction of the soil nail retaining wall will be accepted based on visual inspection and the relevant production testing records.

PART 2 MEASUREMENT AND PAYMENT

2.01 SOIL NAILS

A. Measure production of soil nails by the linear meter (or foot). The length to be paid will be the length measured along the nail bar centerline from the back face of shotcrete to the bottom tip end of nail bar as shown on the Plans. No separate measurement will be made for proof test nails, which shall be considered incidental to production nail installation. Measure verification test nails by the each. Failed verification test nails or additional verification test nails installed to verify alternative nail installation methods proposed by the Contractor will not be measured.

2.02 STRUCTURE EXCAVATION

A. Measure excavation for the soil nail wall as the theoretical plan volume in cubic meters (feet) within the structure excavation limits shown on the plans. This will be the excavation volume within the zone measured from top to bottom of shotcrete wall facing and extending out 2 m (6 ft) horizontally in front of the plan wall final excavation line. Additional excavation beyond the Plan wall final excavation line resulting from irregularities in the cut face, excavation overbreak or inadvertent excavation, will not be measured. No measurement will be made for using temporary stabilizing berms. General roadway excavation will not be a separate

wall pay item but will be measured and paid as part of the general roadway excavation including hauling.

2.03 WALL FACE

A. Measure soil nail retaining walls by the square meter (foot) of wall face. Measurement will be made on the vertical plane of front face accepted in the final work. No measurement or payment will be made for additional shotcrete or CIP concrete needed to fill voids created by irregularities in the cut face, excavation overbreak or inadvertent excavation beyond the Plan final wall face excavation line, or failure to construct the facing to the specified line and grade and tolerances. The final pay quantity shall include all structural shotcrete, admixtures, reinforcement, welded wire mesh, wire holding devices, wall drainage materials, bearing plates and nuts, test panels and all sampling, testing and reporting required by the Plans and this Specification. The final pay quantity shall be the design quantity increased or decreased by any changes authorized by the Engineer.

2.04 PAYMENT

A. The accepted quantities, measured as provided above, will be paid for at the contract unit price per unit of measurement for the pay items listed below that are shown on the bid schedule. Payment will be full compensation for the work prescribed in this section. Payment will be made under:

Pay Item	**Pay Unit**
Permanent Soil Nails. No. ___ Bar (Grade ___)	Linear meter (or linear foot)
Permanent Soil Nails. No. ___ Bar (Grade ___)	Linear meter (or linear foot)
Verification Test Nails	Each
Structure Excavation-Soil Nail Wall	Cubic meter (or cubic foot)
Soil Nail Wall	Square meter (or square foot)

SOIL NAIL WALL

PERFORMANCE SPECIFICATIONS

PART 1 GENERAL

1.01 SECTION INCLUDES

A. 1.02 Scope of Work
B. 1.03 Pre-Approved List
C. 1.04 Available Information
D. 1.05 Soil Nail Wall Design Requirements
E. 1.06 Design Submittals

1.02 SCOPE OF WORK

A. Specifications under a performance type contracting method are identical to the Procedural Specifications except for the first section.

B. This work consists of designing and constructing permanent soil nail retaining wall(s) at the location shown on the drawings. The Contractor shall furnish all labor, plans, drawings, design calculations and all other material and equipment required to design and construct the soil nail wall(s) in accordance with this Specification.

1.03 PRE-APPROVED LIST

A. The prime Contractor must select one of the specialty contractors listed below and shall identify the specialty contractor on his proposal at the bid opening. No substitution will be permitted without written approval of the Engineer. Substitution after the bid opening will not be grounds for changes in bid prices.

1. Name and address of specialty contractor 1.
2. Name and address of specialty contractor 2, etc.

1.04 AVAILABLE INFORMATION

A. Available information developed by the (Agency) include the following items:

1. Contract Drawings titled, _____, dated _____.

2. Geotechnical Report *(Provide complete references for all available geotechnical data and reports).*

3. Other *(Agency-developed Inspector Information; Design Guidelines, etc.).*

(Note: These performance specifications must be completed by adding Subsections 1.03 to 1.21 and Section 2.0 from the Procedural Specifications.)

1.05 SOIL NAIL WALL DESIGN REQUIREMENTS

A. Design the soil nail walls using the Allowable Stress Design (ASD) method, also known as Service Load Method (SLD), as outlined in FHWA Geotechnical Engineering Circular No. 7. "Soil Nail Walls." Soil/rock design shear strength parameters, slope and external surcharge loads, seismic design coefficient, type of wall facing, architectural treatment, corrosion protection requirements, easements, and right-of-ways will be as shown on the Drawings.

1.06 DESIGN SUBMITTALS

A. At least 45 days before the planned start of the wall excavation, submit complete design calculations and working drawings to the Engineer for review and approval. Include all details, dimensions, quantities, ground profiles and cross-sections necessary to construct the wall. Verify the limits of the wall and ground survey data before preparing the drawings. The working drawings shall be prepared to the (*Agency*) standards. The drawings and calculations shall be signed and sealed by a Professional Engineer registered in *State of* [*Name of State/Commonwealth]* _____. The Engineer will approve or reject the Contractor's submittals within 30 calendar days after the receipt of the complete submission. The Contractor will not begin construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer.

SOIL NAIL WALL

SHOTCRETE SPECIFICATIONS

PART 1 GENERAL

1.01 SECTION INCLUDES

A. Scope of Work
B. Materials
C. References to other specifications *(e.g., Soil Nail Wall Specifications)*

1.02 SCOPE OF WORK

A. This work consists of constructing one or more courses of shotcrete on a prepared surface.

1.03 MATERIALS

A. *(Use agency standard specifications for the following:)*

1. Air-entraining admixture (wet mix only)
2. Chemical admixtures (wet mix only)
3. Concrete coloring agents
4. Curing material
5. Hydraulic cement
6. Pozzolans
7. Reinforcing steel

B. Shotcrete Aggregate

1. For fine aggregate, furnish rounded particles conforming to AASHTO M 6 Class B including the reactive aggregate supplementary requirement, except as amended or supplemented by the following:

   Material passing 75-µm sieve, AASHTO T 11 ..... 3.0 % max
   Sand equivalent value, AASHTO T 176..... 75 min. referee method

2. For coarse aggregate, conform to AASHTO M 80 class B, except as amended or supplemented by the following:

   Los Angeles abrasion, AASHTO T 96 ..... 40 % max.

Combine the aggregates to meet the designated gradation in Table 1.

Table 1: Shotcrete Gradation Limits for Combined Aggregates.

Sieve Size	Percent by Mass Passing Designated Sieve (AASHTO T 27)		
	Grading Designation		
	A	B	C
19 mm	100	100	100
12.5 mm	100	100	80-95
9.5 mm	100	90-100	70-90
4.75 mm	95-100	70-85	50-70
2.4 mm	80-100	50-70	35-55
1.2 mm	50-85	35-55	20-40
600 µm	25-60	20-35	10-30
300 µm	10-30	8-20	5-17
150 µm	2-10	2-10	2-10

1.04 REINFORCING FIBERS

A. Contractor may elect to use reinforcing deformed steel or fibrillated polypropylene fibers conforming to ASTM C 1116. The use of reinforcing fibers shall be pre-approved by the Engineer.

PART 2 CONSTRUCTION

2.01 GENERAL

A. Conform to the following:

1. ACI 506R Guide to Shotcrete.
2. ACI 506.1 State of the Art Report on Fiber Reinforced Shotcrete.
3. ACI 506.2 Specifications for Proportioning Application of Shotcrete.
4. AASHTO C 311 Method for Sampling and Testing Fly Ash or Natural Pozzolans for Use as a Mineral Admixture in Concrete.
5. ASTM C 1077 Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation.

2.02 PRECONSTRUCTION SUBMISSIONS

A. Shotcrete material, equipment, preparation, and application. Submit the following to the Engineer for acceptance at least 30 days before placing shotcrete:

1. Description of proposed equipment for mixing and applying shotcrete conforming to Subsection 2.03. Include the manufacturer instructions, recommendations, literature, performance, and test data.

2. Proposed shotcrete mix design conforming to Subsection 2.04 with mix proportions.

3. Representative samples of shotcrete material, if requested by the Engineer.

4. Results of all shotcrete preconstruction testing conforming to Subsection 2.05.

5. Proposed method for applying and curing shotcrete conforming to Subsections 2.06, 2.07, and 2.08.

6. Other information necessary to verify compliance with ACI 506.2.

7. Certification that shotcrete conforms to the standards specified herein.

8. Fiber samples, if used, with supplier or manufacturer recommendations for use.

B. Submit the following to the Engineer for acceptance at least 30 days before placing shotcrete:

1. *Project references.* Include project name, owner's name, and phone numbers from at least 3 projects of comparable nature completed in the last 2 years.

2. *Nozzle operator's experience and training.* For each nozzle operator, include shotcrete application experience on at least two projects of comparable nature.

3. *Shotcrete supervisor experience.* Include direct shotcrete application experience on comparable projects.

4. *Testing laboratory certification*. Include documentation that the strength-testing laboratory complies with ASTM C 1077 and has the experience to perform the tests specified in this Section. The testing laboratory shall be AASHTO-accredited for ASTM C 1077 or demonstrate the ability to perform the requisite tests.

2.03 EQUIPMENT

A. *Water Supply System.* For dry mix, provide a water storage tank at the job site. Provide a positive displacement pump with a regulating valve that is accurately

controlled to provide water in the pressures and volumes recommended by the delivery machine manufacturer.

B. *Mixing.* Use equipment capable of handling and applying shotcrete containing the specified maximum size aggregate and admixtures. Provide an air hose and blowpipe to clear dust and rebound during shotcrete application.

C. *Air Supply System.* Use an air supply system capable of supplying the delivery machine and hose with air at the pressures and volumes recommended by the machine manufacturer. Do not use air supply systems that deliver oil-contaminated air or are incapable of maintaining constant pressure.

D. *Delivery Machine.* Use a delivery machine capable of supplying material to the delivery hose at a uniform rate. The ejection from the nozzle must adhere to the treated surface with minimum rebound and maximum density when the nozzle is held in the range of 1 to 2 m (3 to 6 ft) from the target surface.

2.04 COMPOSITION (SHOTCRETE MIX DESIGN)

A. Design and produce shotcrete mixtures conforming to Table 2 for the type of shotcrete specified. Use the amount of water required to produce shotcrete of suitable strength, consistency, quality, and uniformity with the minimum amount of rebound. Use the same material types and sources as submitted with the mix design in the field trials and production work.

1. Fibers. If fibers are required, add them to the mix in the proportions recommended by the manufacturer.

2. Hydration stabilizing admixtures. Hydration stabilizing admixtures may be used to extend the allowable delivery time for shotcrete. Dosage is based on the time needed to delay the initial set of the shotcrete for delivery and discharge on the job. Design shall include discharge time limit in the dosage submittal. Dosage required to stabilize shotcrete shall be determined using job site material and field trial mixtures. The extended-set admixture shall control the hydration of all cement minerals and gypsum. The maximum allowable design discharge time is 3.50 hours.

3. If a hydration-stabilizing admixture is approved for use in the concrete mix, concrete shall be delivered and placed within the approved design discharge time limit. An approved and compatible hydration activator may be used at the discharge site to insure proper placement and testing.

4. Dosage and type of extended-set admixture shall be included with proposed mix design. When requested, the admixture manufacturer shall provide the service of a qualified person to assist in establishing the proper dose of extended-set admixture and make dosage adjustments required to meet changing job site conditions.

Table 2: Composition of Shotcrete.

Type of Shotcrete Process	Minimum Cement Content		Maximum W/C[(1)] Ratio	Air Content Range	Minimum 28-Day Compressive Strength[(3)]	
	(kg/m^3)	(lb/cy^3)		(%)	(MPa)	(psi)
Wet	325	550	0.55	NA	28	150
Dry	325	550	0.50	NA	28	150
Wet (w/EA)	325	550	0.45	5 min	28	150
Dry (w/EA)	325	550	0.45	5 min.	28	150

Notes: (1) W/C = Water/Cement (by weight).
(2) EA = Entrained Air.
(3) According to AASHTO T 23.

2.05 PRECONSTRUCTION TESTING

A. Conduct preconstruction shotcrete field trials before starting shotcrete production. Allow the Engineer the opportunity to witness all phases of the preconstruction testing.

1. Field Trials: Construct wood forms at least 150-mm (6-in.) thick by 1.0 m by 1.0 m (3 ft by 3 ft) in size. Have each proposed nozzle operator make test panels on two vertical wood forms. Cure the test panels according to AASHTO T 23, without immersing the panels.

2. Coring: Drill six 75-mm (3-in.) diameter cores from each test panel according to AASHTO T 24. Trim the ends of the cores according to AASHTO T 24 to make cylinders at least 75-mm (3-in.) long.

3. Compressive Strength Testing: Soak the cylinders in water for 40 hours immediately before testing. Test three cylinders from each test panel four days after field trial and test the remaining three cylinders 28 days after the field trial. Perform tests according to AASHTO T 23. All specified strength requirements shall be satisfied before the shotcrete mix design will be considered for acceptance.

4. Mix Design Acceptance: The Engineer will accept or reject the shotcrete mix design based on the results of the preconstruction field trials and testing. Before

approving any changes to a previously accepted mix design, the Engineer may require additional preconstruction testing at no additional cost to the agency.

2.06 SURFACE PREPARATION AND APPLICATION OF SHOTCRETE

A. Surface Preparation - Clean loose material, mud, rebound, and other foreign matter from all surfaces to receive shotcrete. Remove curing compound on previously placed shotcrete surfaces by sandblasting. Install approved depth gages to indicate the thickness of the shotcrete layers. Install depth gages on 2-m (6-ft) centers longitudinally and transversely with no less than two gauges per increment of surface area to receive the shotcrete. Moisten all surfaces.

B. Weather Limitations - Place shotcrete when the ambient temperature is 5ºC or higher. Do not perform shotcrete operations during high winds and heavy rains.

C. Shotcrete Application

1. Do not apply shotcrete to frozen surfaces.

2. Use acceptable nozzle operators who have fabricated acceptable test panels according to Subsection 2.02.

3. Apply shotcrete within 45 minutes of adding cement to the mixture. Apply shotcrete at a temperature between 10ºC and 30ºC.

4. Direct the shotcrete at right angles to the receiving surface except when shooting ground reinforcing bars. Apply shotcrete in a circular fashion to build up the required layer thickness. Apply shotcrete in a steady uninterrupted flow. If the flow becomes intermittent, direct the flow away from the work area until it becomes steady.

5. Make the surface of each shotcrete layer uniform and free of sags, drips, or runs.

6. Limit the layer thickness of each shotcrete application to 50 mm (2 in.). Thicker applications may be approved if the contractor can demonstrate that no sloughing or sagging is occurring. If additional thickness is required, broom or scarify the applied surface and allow the layer to harden. Dampen the surface before applying an additional layer.

7. Remove laitance, loose material, and rebound. Promptly remove rebound from the work area.

8. Taper construction joints to a thin edge over a distance of at least 0.3 m (1 ft). Wet the joint surface before placing additional shotcrete on the joint. Do not use square construction joint.

D. Production Summary - Prepare and submit a summary of shotcrete production application for each shift. Furnish the summary to the Engineer within 24 hours. Include the following information in the report:

1. Quantity and location of shotcrete applied including sketches.

2. Observations of success or problems of equipment operation, application, final product conditions, and any other relevant issues during production and application.

3. Description of placement equipment.

4. Batch number(s) if applicable.

2.07 QUALITY CONTROL RECORDS

A. Submit field quality control test reports within two working days of performing the tests. Include the following information in the reports:

1. Sample identification including mix design and test panel number and orientation.

2. Date and time of sample preparation including curing conditions and sample dimensions.

3. Date, time, and type of test.

4. Complete test results including load and deformation data during testing, sketch of sample before and after testing, and any unusual occurrences observed.

5. Names and signature of person performing the test.

6. Location of steel reinforcement, if used, covered by shotcrete.

7. Name of nozzle operator.

2.08 PROTECTION AND CURING

A. Protect and cure the surface according to _____ *(Use agency specifications for concrete curing*). For intermediate shotcrete surfaces or if a stained or finished final surface is required, cure the shotcrete according to _____(*Use agency specifications for water curing).* If no stained or finished surface is required, apply curing compound to the final exposed shotcrete surface according to (*Use agency specifications curing compound methods).* Protect and maintain shotcrete at a temperature above 5ºC until shotcrete has achieved a minimum strength of 5.2 MPa (750 psi).

2.09 ACCEPTANCE

A. Material for concrete will be evaluated by visual inspection of the work, conformance testing and by certification for materials manufactured off-site. Compressive strength will be evaluated by conformance testing using Table 2 for specification limits. See Table 3 for minimum sampling and testing requirements and acceptance quality category.

Table 3: Sampling and Testing of Shotcrete.

Material or Product	Property or Characteristic	Category	Test Methods or Specifications	Frequency	Sampling Point
Shotcrete	Air content	–	AASHTO T 152 or AASHTO T 196	1 per load(1)	Truck, mixer or agitator(2)
	Unit mass	–	AASHTO T 121	1 per load(1)	Truck, mixer or agitator(2)
	Compressive strength	II	AASHTO T 23	1 set per 25 m^3 (33 cy), but not less than 1 set each day (3)	Production test panels(3)

Notes: (1) When continuous mixing is used sample every 7.5 m^3 (10 cy).
(2) Sample according to AASHTO T 141.
(3) Prepare production test panels according to Subsection 2.05. Obtain two 75-mm (3-in.) diameter core specimens from each panel according to AASHTO T 24. A single compressive strength test result is the average result from two 75-mm (3-in.) diameter core specimens from the same test panel tested according to AASHTO T 23 at 28 days.

2.10 MEASUREMENT

A. Measurement for payment for shotcrete shall be measured by the square meter (square foot).

2.11 PAYMENT

A. The accepted quantities, measured as provided above, will be paid at the contract price per unit of measurement for the pay item listed below that is shown in the bid schedule. Payment will be full compensation for the work prescribed in this Section.

Payment will be made under:

Pay Item	**Pay Unit**
Shotcrete	Square meter (or square foot)

SOIL NAIL WALL

GEOTECHNICAL INSTRUMENTATION

PART 1 GENERAL

1.01 SECTION INCLUDES

A. Scope of Work
B. Materials
C. References to other specifications *(e.g., Soil Nail Wall Specification; Survey Specification)*
D. Other? _____

1.02 SCOPE OF WORK

A. This work consists of furnishing all instruments, tools, materials, and labor and performing all work necessary to install soil nail wall instrumentation and record the initial readings. The Contractor shall maintain and protect all instruments for the duration of the Contract. The Contractor shall repair or replace damaged or inoperable instruments within 72 hours after the damage has been discovered and the contractor informed.

1.03 GENERAL CONSTRUCTION REQUIREMENTS

A. Notify the Engineer prior to any work on instrumentation installation and monitoring. All instrumentation field installations shall be performed in the presence of the Engineer.

B. Install electrical readout instruments and wire to a readout panel. Complete wiring to the readout panel after installation of each instrument, after each instrument is tested by the Contractor to the satisfaction of the Engineer, and prior to excavation of subsequent soil nail lifts. Demonstrate that the system is working according to the manufacturer's specifications. Immediately repair or replace any monitoring device components that are damaged or failed, for whatever reason, to perform the intended function, to the satisfaction of the Engineer and at no additional cost.

C. Furnish and install inclinometers at the locations shown on the Plans. Install inclinometers and take initial readings prior to soil nail wall construction. Adjust soil nail installations at these locations as necessary to avoid damaging the inclinometer casing.

D. Protect all instrumentation during the term of the contract and replace or restore them at the Contractor's expense and to the satisfaction of the Engineer if delivered defective or damaged during construction. Damaged or inoperable instruments shall be repaired or replaced within 72 hours after damage has been discovered and the Contractor informed.

1.04 EXPERIENCE REQUIREMENTS AND SUBMITTALS

A. At least 15 days prior to start of the soil nail wall instrumentation installation, submit in writing to the Engineer five copies of: (1) a list of proposed instruments including instrument and readout unit specifications; (2) complete and detailed installation procedures, including both the manufacturer's recommendations and the Contractor's step-by-step field procedures; (3) a wiring diagram detailing the wiring of the instruments to the central readout panels; and (4) shop drawings and specifications for ancillary equipment such as readout panels, load cell blockouts and covers, other protective covers, conduit, and enclosures.

B. Install the instruments utilizing a qualified geotechnical instrumentation specialist having experience in the design and installation of similar instrumentation systems on a minimum of 3 similar projects. At least 15 days prior to start of the soil nail wall instrumentation installation, submit the resume of the individual(s) responsible for instrument installation and testing. The submittal shall include at least three references, with current telephone numbers, of persons who can verify the experience requirements.

1.05 QUALITY ASSURANCE

A. Factory calibration shall be conducted on all instruments prior to shipment from the manufacturing locations. Certification shall be provided to indicate that the test equipment used for this purpose is calibrated and maintained in accordance with the test equipment manufacturer's calibration requirement and that, where applicable, calibration are traceable to the National Institute of Standards and Technology. The manufacturer prior to shipment shall make a final quality assurance inspection with results of the inspection recorded on a checklist. A copy of the completed checklist shall be included with each instrument shipped.

B. The Contractor shall provide the manufacturer's warranty for each piece of equipment furnished for the monitoring program and such warranty shall be in place for the duration of the contract.

1.06 INCLINOMETERS

A. Install inclinometer casing in vertical drill holes within 3 degrees of vertical, as measured by the initial inclinometer reading, and fully grouted in place to the depths and at the locations shown on the plans. Drillhole diameter shall be not less than 150 mm (6 in.) Complete inclinometer installation at least one week prior to the beginning of wall excavation. One of the casing grooves shall be aligned normal to the wall to a tolerance of +/- 5 degrees throughout the length. Casings adjacent to the soil nail wall shall be installed to a minimum penetration of 5 m (15 ft) below the wall base.

B. Protect the top of all inclinometer casings with a locking metal protective cover of sufficient size to allow monitoring the installation with the inclinometer wheel attached to the inclinometer casing. After installation, survey the top of each inclinometer casing and determine the coordinates and elevation with an accuracy of 3 mm (0.1 in.).

C. All instruments shall be compatible with and calibrated using readout devices approved by the Engineer.

D. *Inclinometer Casing.* Inclinometers shall consist of internally grooved plastic, aluminum, fiberglass, or steel casing in 3-m (10-ft) lengths, provided with all necessary end plugs, and caps. Casing shall be the snap-together self-aligning type that does not require couplings. The spiral twist of casing grooves in one 3-m (10-ft) section of casing shall not exceed one degree. The top of each casing shall be provided with plastic cap and locking steel protective monument cover cap. Casing shall be as manufactured by Slope Indicator Company of Seattle, Washington; Carlson/R.S.T. Instruments Inc. of Yakima, Washington; Roctest Inc., of Plattsburgh, New York; Geokon, Inc. of Lebanon, New Hampshire; or an approved equal.

E. *Inclinometer Grout.* Backfill around the inclinometer casing shall be a pumpable mix of water-cement-lime grout consisting of one bag [43 kg (95 lbs)] of cement to three bags [68 kg (150 lbs)] of hydrated lime. Other mixes may be used, if approved by the Engineer.

F. *Inclinometer Probe.* The inclinometer probe shall be a biaxial sensor such as the Digitilt manufactured by Slope Indicator Company; the inclinometer probe manufactured by Carlson/R.S.T. Instruments, Inc. of Yakima, Washington; the Accutilt instrument manufactured by Roctest, Inc. of Champion, New York; the inclinometer manufactured by Geokon, Inc. of Lebanon, New Hampshire; or an approved equal. The probe cable shall be heavy duty, waterproof, and designed to support the weight of the probe without stretching, slipping, or creeping. The cable

shall be clearly marked at 300-mm (12-in.) intervals. The readout unit shall be compatible with the inclinometer probe. The probe and cable shall be serviced by the manufacturer as a unit at least 30 days prior to construction.

1.07 STRAIN GAUGES

A. The strain gauges shall be weldable vibrating wire gauges manufactured by Geokon, Inc., Roctest, Inc., or Geo Group, Inc., or approved equal. Install and protect the strain gauges and connections in accordance with the manufacturer's specifications. Field test each strain gauge to verify that it is fully operational prior to mounting on the nail. Defective gauges shall be rejected.

B. Encapsulation corrosion protection shall not be required for the instrumented nails. Each instrumented nail shall be epoxy coated per the soil nail wall materials specification. The epoxy coating shall be removed as necessary to install the gauges. Mount the gauges to the bar in pairs at each location shown on the plans. Mount all gauge pairs on opposite sides of the bar 180 degrees apart. Mount all gauge pairs on the same plane. Inscribe the end of each nail bar along the plane of orientation of the strain gauges.

C. Protect all gauges, sensors, and wire assemblies from moisture. All wire connections shall be of an approved waterproof type and shall be fitted with at least two waterproof, tamper-resistant labels spaced 3 m (10 ft) apart at the readout panel end of the wire. Signal cables shall not be spliced unless approved by the Engineer. Use centralizers on the instrumented nails to ensure that the bar is located within 25 mm (1 in.) of the center of the drillhole. Install the nail so that the final locations of the gauge pairs are at the 6- and 12-o'clock positions with a tolerance of 10 degrees.

1.08 NAIL LOAD CELLS

A. Soil nail load cells shall have an ultimate capacity not to exceed 450 kN (100 kips) with an accuracy of 2.25 kN (500 lbs). The load cells shall be center-hole load cells with a minimum hole diameter of 38 mm (1½ in.). The load cells shall be the center-hole load cell manufactured by Slope Indicator Company; Geokon, Inc.; Geo Group, Inc. of Gaithersburg, Maryland; Carlson/R.S.T. Instruments, Inc.; Roctest, Inc.; or approved equal. Load cells shall be temperature compensated or provided with temperature sensors as recommended by the manufacturer.

B. Mount the load cell on the nail between the bearing plate and the nut as shown on the Plans. All bearing surfaces shall be clean. Spherical bearings shall be well lubricated with suitable grease. Attach the cells and protect the connections according to the manufacturer's specifications. All wire connections shall be of an approved waterproof type.

C. Provide a 300-mm^2 (0.5-in^2) circular blockout in the cast-in-place wall facing for the load cell assembly. Install a steel cover plate over the blockout to protect the load cell. Paint, galvanize, or otherwise protect the cover plate from corrosion. Install in such a manner as to allow easy future access to the load cell.

1.09 READOUT PANELS

A. The readout panels shall be of sufficient size and capacity to handle the specified number of instruments for each instrumented section. Each instrument shall have an isolated channel and shall be readily identified by waterproof labels resistant to vandalism and tampering.

B. Locate one readout panel at each instrumentation section unless otherwise approved by the Engineer. Attach the readout panel to a steel or treated wooden post that is firmly secured in the ground and located a distance of approximately 1 m (3 ft) behind the top of the nailed wall or at another convenient location as directed or approved by the Engineer. Wire all instrumentation to the readout panel in accordance with the manufacturer's recommendations. The readout panel shall be securely sealed and shall be rated NEMA 4X or better. Protect the readout panel from vandalism and tampering. Enclose all above-ground wiring in a steel conduit that is firmly attached to the readout panel. Do all wiring of the instrument readout panel during instrument installation.

C. Provide data logger devices compatible with the instrumentation for acquisition of strain gage and load cell data. The data logger shall be compatible with strain gages and load cells installed without degrading the accuracy of the instruments. The data logger shall have programmable reading intervals, data storage, and capability of downloading to a computer. Software to communicate with the data logger, and for downloading data, shall also be provided. The data logger shall be fully programmed for the project with software customized to this particular system and application and shall be compatible with the Owner's portable PC system. The Owner's personnel shall be trained in the use of the data acquisition system to the satisfaction of the Engineer.

1.10 WALL SURVEY

A. Install reflective survey prisms at the locations shown on the plans in the shotcrete face while shotcreting the initial soil nail wall lift.

1.11 MONITORING AND REPORTING

A. For each inclinometer installed, take initial inclinometer readings at least 48 hours after installation and before the beginning of wall excavation. Monitor in

accordance with the inclinometer probe manufacturer's recommended procedures and AASHTO T-254. The Owner's personnel shall be trained in the use of the inclinometer data acquisition system to the satisfaction of the Engineer.

B. Provide a hard copy of the data and a graph of movement (mm or in.) versus depth (m or ft) for each inclinometer in both directions to the Engineer within 24 hours of monitoring the inclinometers. Include all sets of readings. Summarize and include the survey data with the submittal. Submit an electronic copy of the data in a format compatible with the software GTILT.

PART 2 MEASUREMENT AND PAYMENT

2.01 MEASUREMENT

A. No separate measurement will be made for the materials and work specified in this Section. The unit of measurement for Soil Nail Wall Instrumentation will be lump sum.

2.02 PAYMENT

A. Soil nailing instrumentation will be paid for at the contract lump sum amount for the item Soil Nail Wall Instrumentation. Payment will be full compensation for furnishing all materials, labor, equipment, tools, and incidentals necessary to complete the work as specified in this Specification and as shown on the Plans.

B. Upon satisfactory installation and final acceptance by the Engineer, all instruments and readout units furnished and installed under this Section shall become the property of the Owner.

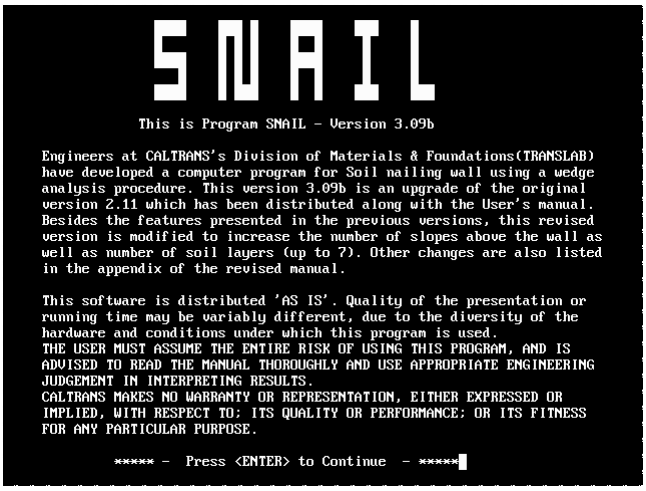
Pay Item	**Pay Unit**
Soil Nail Wall Instrumentation	Lump Sum

(Alternately and where the scope of the instrumentation during design is not well developed, payment can be made on an each basis for each instrument provided and installed. In addition payment can be made for each readout instrument should the agency not own any.)

APPENDIX F

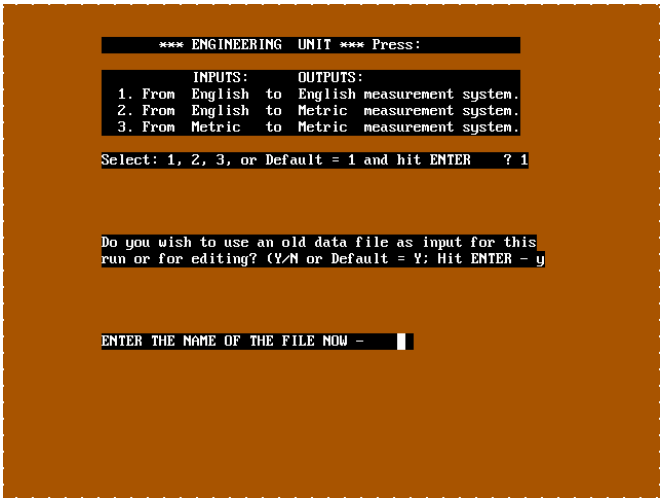
USER'S GUIDE FOR SNAIL, VERSION 3.09

SNAIL is executed by typing the command **nail.exe** at the DOS prompt in a directory where the executable file resides. The following screen appears:



Press <**ENTER**> to Continue.

The following screen asks for type of units to be used:



1. English to English
2. From English to Metric
3. From Metric to Metric

If user desires to <u>create a new file</u>, enter **N,**
screen prompts:
"Type in the name of your Data File now"
for new filename,
Use up to a maximum of 32 characters
Press **<ENTER>** to continue,

If user desires to <u>open an existing file</u>, enter **Y** or <**Enter>,**

then user is prompted, **"Enter the Name of the File Now"**
then user must enter filename
Maximum: 32 characters;
filename entered should include disk designation (e.g., C:\ \design\data input1;
no extension is needed,
Press **<ENTER>** to continue,
Press <**Backspace**> to erase

The next screen asks:
Do you want to change the title? (Y/N)

If **Y**, the next message asks:
Type in your new title
Use up to a maximum of 32 characters
Press **<ENTER>** to continue,
Press <**Backspace**> to erase

If **N**, ready to input data.

Data for SNAIL should be entered for each of the parameters requested in different input panels, which are organized by category and are presented below. To move from one panel to another, the **Page Down** and **Page Up** keys can be used. Once created, the data is saved in data files that can be reopened and modified.

The data input panels are:
- Project Description (Optional)
- Wall Geometry
- Reinforcement Parameters
- Soil Parameters
- Search Limit

Depending on the Options selected, additional data may be required, including:
- Surcharge
- Earthquake Acceleration
- Water Surface
- Search Limits Specification
- Slope Below Wall
- Varying Reinforcement Parameters
- External Horizontal Force
- Specified Failure Plane

All data must be entered using the units requested.

INPUT PANELS

Project Description

Data identifying the project consist of characters and numbers. Characters following a comma are not considered in this panel.

Wall Geometry (Panel 1)

Required data include angles (I1, I2,…) and lengths (S1, S2,…) to define wall geometry above the wall toe. Coordinate (0,0) is toe of wall. Enter all data in blank spaces. A blank entry means zero.

The following data can be edited at any time.

1. Vertical Wall Height
2. Wall Batter from the vertical axis
3. First Slope Angle
4. First Slope Distance from Wall Crest
5. Second Slope Angle
6. Second Slope Distance from First Slope
7. Third Slope Angle
8. Third Slope Distance from Second Slope
9. Fourth through Seventh Slope Angle, and Slope Lengths

Panel 1

```
1.-WALL GEOMETRY:
 H =       ft-------Vertical Wall Height.
 B =       Degree---Wall Batter from Vertical Line.
 I1=      Degree| S1=      ft---1st Slope Angle and Distance.
 I2=      Degree| S2=      ft---2nd Slope Angle and Distance.
 I3=      Degree| S3=      ft---3rd Slope Angle and Distance.
 I4=      Degree| S4=      ft---4th Slope Angle and Distance.
 I5=      Degree| S5=      ft---5th Slope Angle and Distance.
 I6=      Degree| S6=      ft---6th Slope Angle and Distance.
 I7=      Degree-----7th Slope Angle.
  2.-REINFORCEMENT INPUTS:(Use OPTION 5 if LE, AL, SV, D, or BSF* varies.)
 N =       ---------Number of Reinforcement Levels.
 LE=       ft-------Reinforcement Length.
 AL=       Degree---Reinforcement Inclination.
SV1=       ft-------Vertical Distance to first Level.
 SV=       ft-------Vertical Spacing from second to N level.
 SH=       ft-------Horizontal Spacing
 PS=       Kips-----Punching Shear at reinforcement head.
 FY=       Ksi------Yield Stress of Reinforcement.
 D =       in-------Diameter of Reinforcement.
 DD=       in-------Diameter of Grouted Hole.
 Use Arrow and Return Keys to move around, Backspace and Delete Keys to edit
 When data entry finished, press Page Up, or Down, or Esc Key to Run program.
```

Press **PAGE UP** or **PAGE DOWN** to Continue

Figure F.1 shows the sign convention of angles and lengths.

<u>Reinforcement Parameters (Panel 1)</u>

Nail parameters can be modified simultaneously in all nails (through Panel 1) or <u>directly</u> modified at every nail level in Panel 5. The exception are parameters marked with *, which can only be modified <u>indirectly</u> in Panel 5.

Constant Parameters are:

1. Number of nail (reinforcement) levels (Maximum of 30)
2. Nail (Reinforcement) length (*)
3. Reinforcement inclination (*)
4. Nail vertical distance to first level (can be negative)
5. Vertical spacing (*)
6. Horizontal spacing
7. Punching shear capacity
8. Nail bar yield strength
9. Diameter of nail bar (*)
10. Diameter of drillhole
11. Bond strength*

Figure F.2 shows the definition of constant reinforcement parameters

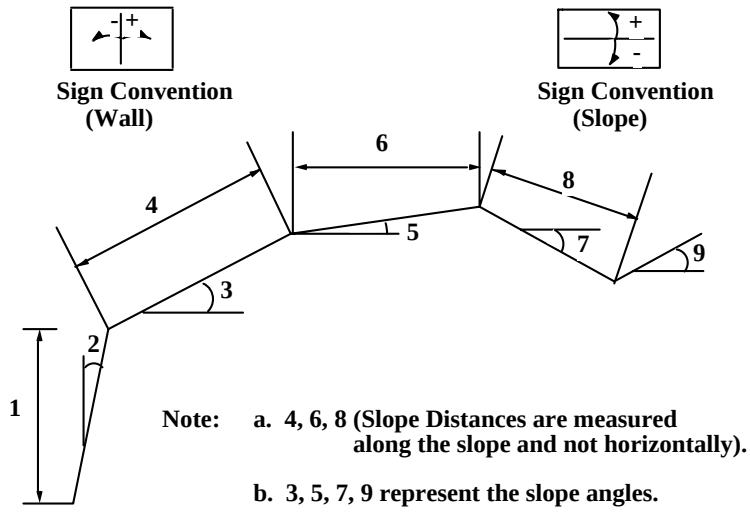


Figure F.1: Wall Geometry Definition.

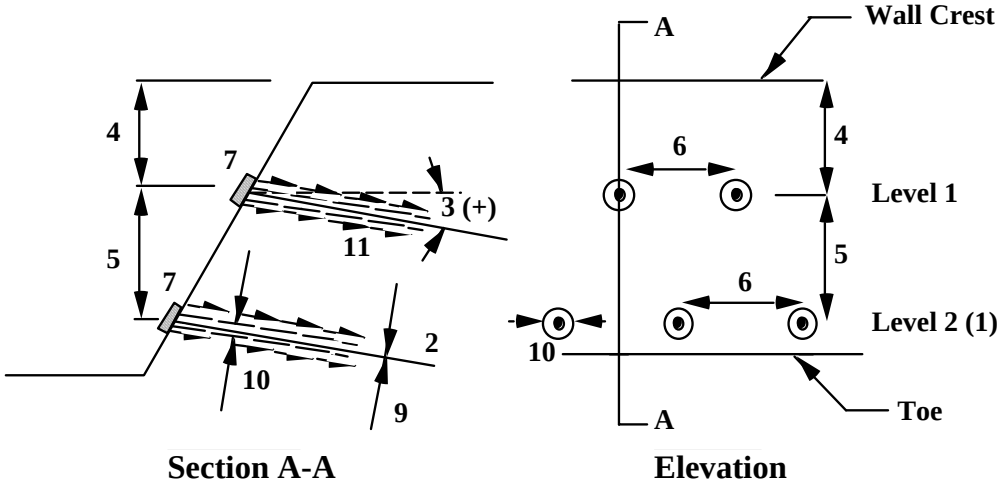


Figure F.2: Reinforcement Parameters.

Soil Parameters (Panel 2)

A maximum of seven soil layers can be specified with the following variables:

1. Unit Weight, GAM.
2. Friction Angle, PHI.
3. Cohesion, COH.
4. Bond Stress, SIG.

The boundary between soils layers is defined by specifying the coordinates of two points behind the wall. Layer order is from left to right, or from top to bottom.

Search Limit (Panel 2)

The search area limits are defined along the backslope (Figure E-3) by the x-coordinates of points LS and LN. The left limit can start at the wall or at a user-selected location. If LS is not entered, the program uses LS=0 and the search starts at the wall crest. The right limit is always user-defined. The program automatically divides the search distance into ten equal length segments.

Panel 2

```
3.-SOIL PARAMETERS:
 NS =      Number of soil types.(1=Top layer to 7=Bottom layer-
         Layers must not intersect within limits of search).
     Weight| Angle|Cohes.| Bond*|  XS  |  YS  |  XE  |  YE
LAYER   Pcf | Deg. |  Psf |  Psi | (ft) | (ft) | (ft) | (ft)
  1 ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  ___ ___ ___ ___ ___ ___ ___ ___
  4.-SEARCH LIMIT:
 LS=       ft-Begin Search.If LS=0, Search starts at wall crest.
 LN=       ft-End Search.(Horizontal Distance From Wall Toe).
++++++++++ End of Data Inputs required to run SNAIL.++++++++++++
  5.-SURCHARGE: Maximum of 2 different surcharges are entered.
     First  | Second
 XL=        |        ft-----Begin Surcharge: Dist. from Toe.
 XR=        |        ft-----End Surcharge:   Dist. from Toe.
 PL=        |        psf/ft-Loading At Begin Surcharge.
 PR=        |        psf/ft-Loading At End Surcharge.
+++++ Use 'UP'or'DOWN'arrows to scroll. Hit 'Q' or 'q' to quit.+++++
```

Press **PAGE UP** or **PAGE DOWN** to Continue

Surcharge (Panel 2)

Surcharge cases that can be considered with SNAIL are:

No load,
Uniformly distributed load, and
Uniformly varying load.

Up to two sets of surcharges, defined from left to right (Figure E-4), can be considered in SNAIL. Surcharges can be placed in front and behind the wall (0,0 is at wall toe). If the surcharge in front of the wall extends to the wall, use XR= -0.1.

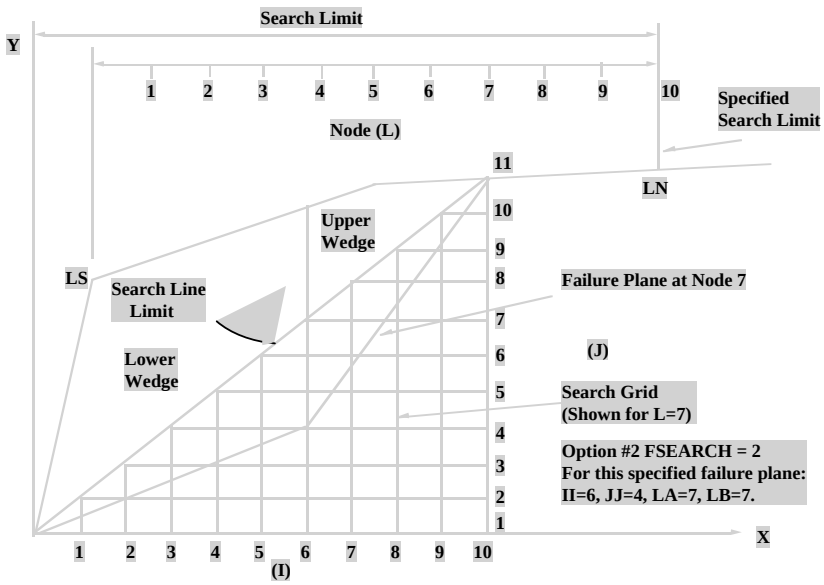


Figure F.3: Search Grid Definition.

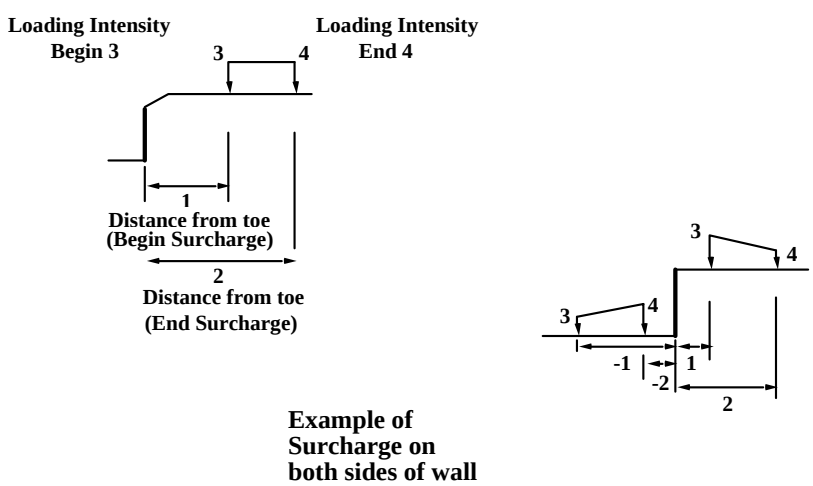


Figure F.4: Surcharge Definition.

<u>Earthquake Acceleration (Panel 3)</u>

Enter:

1. Horizontal Earthquake Coefficient
2. Vertical Earthquake Coefficient

The vertical earthquake coefficient (if used) must be entered as a decimal fraction of the horizontal earthquake coefficient.

Water Surface (Panel 3)

A water surface, if used, is defined using three coordinate points. Enter coordinate points with 0,0 always at the wall. For analysis beyond wall toe, the water surface is specified as follows: (1) The coordinates of the left point are on the wall face, the water surface coincides with the wall behind and the ground surface below; (2) the y-coordinate of left point is negative, the water surface flat or coincident with ground surface. Water table points must form a concave surface (i.e., dips toward the face), otherwise, an error is generated.

Panel 3

```
6.-EARTHQUAKE ACCELERATION:
 KH=       A/G------Horizontal Earthquake Coefficient.
PKH=       %KH/100--Vertical Earthquake Coefficient.
  7.-WATER:
 FLAGW=     ==> 0= not Used. 1= Piezometric. 2= Phreatic
            1st Point     2nd Point     3rd Point
X-Coor.===> XW1=      ft  XW2=      ft  XW3=      ft
Y-Coor.===> YW1=      ft  YW2=      ft  YW3=      ft
 ************************* OPTION  #1 ******************************
 FLAGT=     ==> 0= Ultimate Bond, Yield, &Punching Shear values.
            1= Factored Bond, Yield, &Punching Shear values.
            2= Tie-back Wall only (with Soldier pile wall).
 ************************* OPTION  #2 ******************************
 FSEARCH=   ==> 0= The Search is Routinely from Nodes 1 to 10.
            1= The Search is conducted from nodes LA to LB.
            2= For Specified Failure Plane. Input II And JJ.
   LA=     Beginning at node 'LA'.       II =       Horizontal
   LB=     Ending at node 'LB'.          JJ =       Vertical
 ************************* OPTION  #3 ******************************
 FLAG =     ==> 0= There is no TOE; 1= There is TOE. Enter DATA:
 1st Slope Angle|1st Slope Length| 2nd Slope Angle|2nd Slope Length
 I8=      Degree| S8=        Feet| I9=      Degree| S9=        Feet
 SD=     Ft, Vertical Depth of search.| NTS=     No. of Searches.
```

Press **PAGE UP** or **PAGE DOWN** to Continue

Before running SNAIL, the user must decide which type of analysis is required for the case being considered. SNAIL allows the user to select the analysis type using the following options:

Option 1: Used this option to define the maximum reinforcement strength. The option is recognized through the field FLAGT.

FLAGT = 0. Values of bond strength, reinforcement yield strength, and punching shear capacity are entered with ultimate values. Program uses these strength parameters divided by the indicated factor of safety.

FLAGT = 1. Values of bond strength, reinforcement yield strength, and punching shear capacity are entered as factored values. SNAIL uses these values to calculate the factor of safety.

FLAGT = 2. Use only for tieback walls (with soldier piles). No vertical component of the reinforcement is transmitted to the failure wedge.

Option 2: Search is specified within limits provided by points LS to LN (x-coordinate of left and right boundaries).
FSEARCH = 0. Search is performed for all nodes, from 1 to 10.
FSEARCH = 1. Search is performed between specified nodes LA and LB.
FSEARCH = 2. Factor of Safety of a specified failure surface is computed. Enter points II and JJ. Results are limited to selected range.

Option 3: Toe geometry is specified (see Figure F.5)
FLAG=0. No toe is considered
FLAG=1. Toe is considered and geometry must be input:

(1) First slope angle (positive counter clockwise)
(2) Length of first slope
(3) Slope angle below toe (positive counter clockwise)
(4) Length of slope extension
(5) Maximum depth of search below toe
(6) Number of searches (maximum of five) below toe

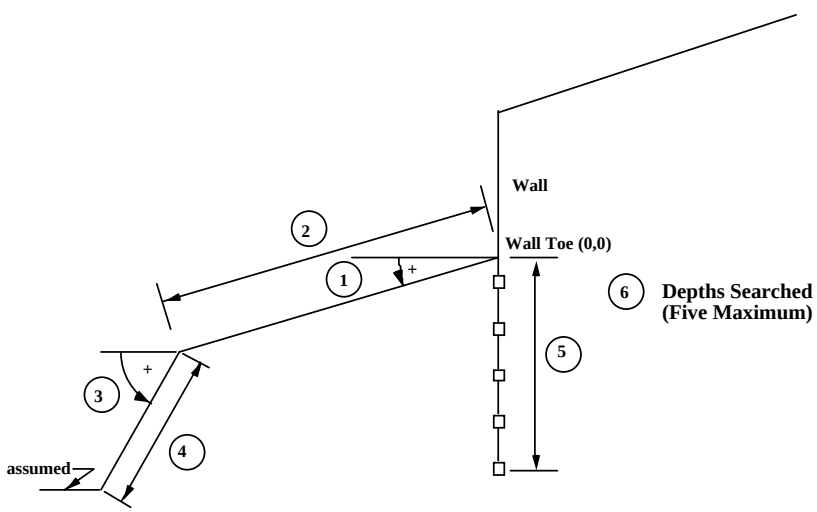


Figure F.5. Slope Geometry below Wall Toe.

Option 4: An external horizontal force, PD, which acts on the wall is specified. Force is positive when pointing toward the wall and negative when pointing away from the wall. Moments due to force are not considered. Force is transmitted to lower wedge.

Option 5: FLAGN = 0. Unused variable.
FLAGN = 1. Data for varying nail parameters is entered (Panel 5).
OPTION 5 allows modifying the nail parameters listed below (Nos. 13-20) at each nail level.

13. Total nail length
14. Nail angle of inclination
15. Nail vertical spacing
16. Nail bar diameter

17. Bond Strength
18. Nail horizontal spacing
19. Drillhole diameter
20. Nail bar yield strength

Options 4 and 5 can be modified directly in Panel 4. The nail vertical spacing and the bond strength must be modified outside instead so as to obtain equivalent parameters 18-20.

Panel 4

```
************************ OPTION  #4 ******************************
 PD=      Kips/ft-Width. External force on Wall. -->(+)|(-)<--
 AN=      Degrees from horizontal. Positive = Counterclockwise.
************************ OPTION  #5 ******************************
 FLAGN=     ==> 0= OPTION #5 is not Used; 1= Used. Enter DATA:
   Reinf.        Reinf.        Vert.         Bar         Bond
  Length      Inclination    Spacing     Diameter     Stress
   (ft)        (Degree)       (ft)        (inch)      Factor*
 LE(01)=     AL(01)=     SV(01)=     D(01)=      SIG(01)=
 LE(02)=     AL(02)=     SV(02)=     D(02)=      SIG(02)=
 LE(03)=     AL(03)=     SV(03)=     D(03)=      SIG(03)=
 LE(04)=     AL(04)=     SV(04)=     D(04)=      SIG(04)=
 LE(05)=     AL(05)=     SV(05)=     D(05)=      SIG(05)=
 LE(06)=     AL(06)=     SV(06)=     D(06)=      SIG(06)=
 LE(07)=     AL(07)=     SV(07)=     D(07)=      SIG(07)=
 LE(08)=     AL(08)=     SV(08)=     D(08)=      SIG(08)=
 LE(09)=     AL(09)=     SV(09)=     D(09)=      SIG(09)=
 LE(10)=     AL(10)=     SV(10)=     D(10)=      SIG(10)=

*NOTES: The Bond Stress Factor (BSF) is a mutiplier of Bond applied
 throughout a bar, regardless of soil parameters. (Default = 1.00)

+++++ Use 'UP'or'DOWN'arrows to scroll. Hit 'Q' or 'q' to quit.+++++
```

```
LE(11)=     AL(11)=     SV(11)=     D(11)=      SIG(11)=
 LE(12)=     AL(12)=     SV(12)=     D(12)=      SIG(12)=
 LE(13)=     AL(13)=     SV(13)=     D(13)=      SIG(13)=
 LE(14)=     AL(14)=     SV(14)=     D(14)=      SIG(14)=
 LE(15)=     AL(15)=     SV(15)=     D(15)=      SIG(15)=
 LE(16)=     AL(16)=     SV(16)=     D(16)=      SIG(16)=
 LE(17)=     AL(17)=     SV(17)=     D(17)=      SIG(17)=
 LE(18)=     AL(18)=     SV(18)=     D(18)=      SIG(18)=
 LE(19)=     AL(19)=     SV(19)=     D(19)=      SIG(19)=
 LE(20)=     AL(20)=     SV(20)=     D(20)=      SIG(20)=
 LE(21)=     AL(21)=     SV(21)=     D(21)=      SIG(21)=
 LE(22)=     AL(22)=     SV(22)=     D(22)=      SIG(22)=
 LE(23)=     AL(23)=     SV(23)=     D(23)=      SIG(23)=
 LE(24)=     AL(24)=     SV(24)=     D(24)=      SIG(24)=
 LE(25)=     AL(25)=     SV(25)=     D(25)=      SIG(25)=
 LE(26)=     AL(26)=     SV(26)=     D(26)=      SIG(26)=
 LE(27)=     AL(27)=     SV(27)=     D(27)=      SIG(27)=
 LE(28)=     AL(28)=     SV(28)=     D(28)=      SIG(28)=
 LE(29)=     AL(29)=     SV(29)=     D(29)=      SIG(29)=
 LE(30)=     AL(30)=     SV(30)=     D(30)=      SIG(30)=

 Use Arrow and Return Keys to move around, Backspace and Delete Keys to edit
 When data entry finished, press Page Up, or Down, or Esc Key to Run program.
```

Once all data panels are complete, press <**Esc**> key

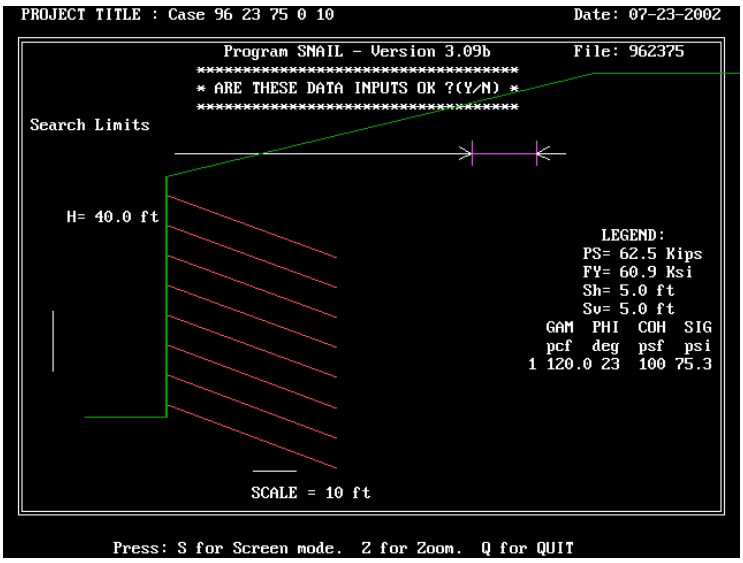
User is asked if input needs to be saved.

If **N** is entered, the program returns to data input panels for further modifications.

If **Y** is entered, input file must be saved. If the selected filename is the same as an existing file, the existing file will be overwritten. File automatically assigned the extension "inp."

Press <**ENTER**> to continue

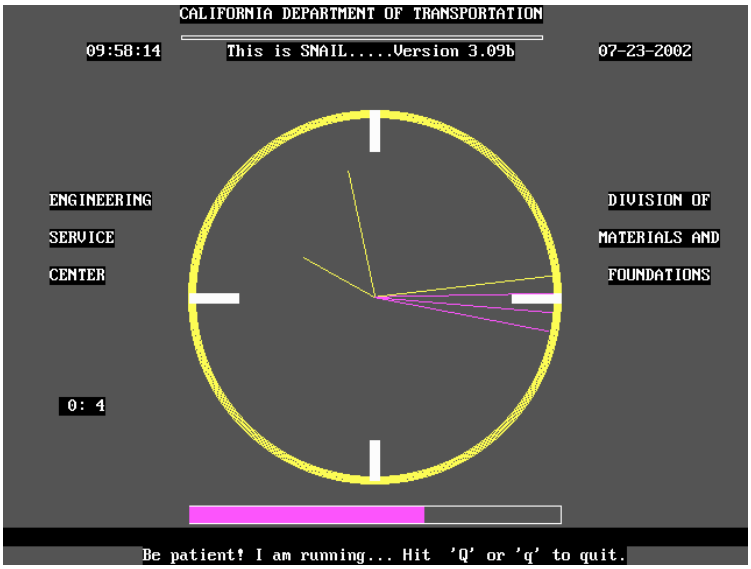
Entered data and problem geometry can be checked one more time through the screen.



If data requires additional modification, press <**N**>
If data is correct, press **Y.**

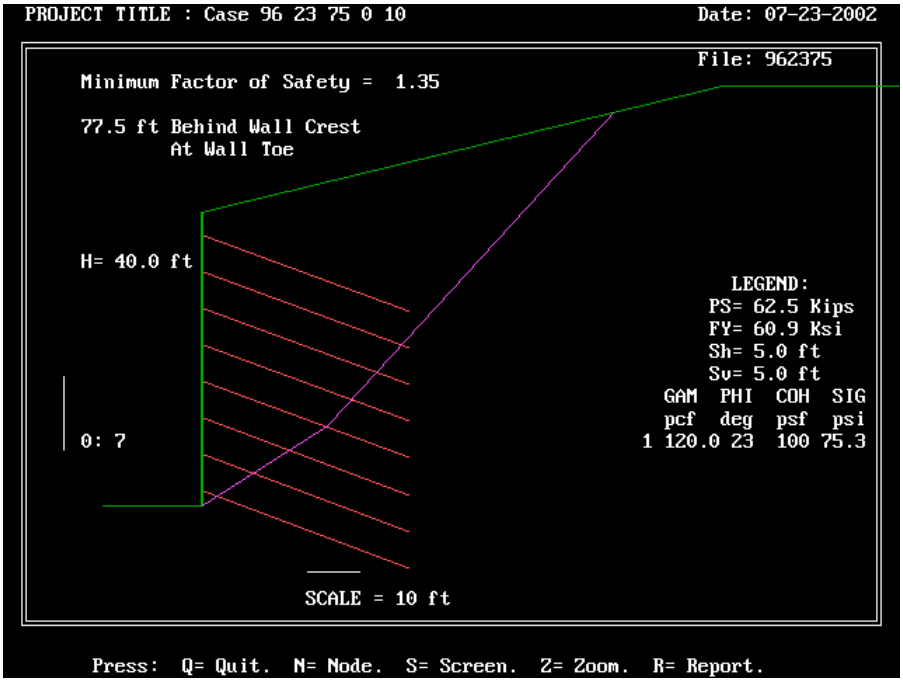
The solution is initiated.

The following screen is displayed during the solution:



Run can be interrupted by pressing <**Q**>.

A summary of the solution is shown graphically as follows:



The following elements are shown:

1. Problem title, filename, and run date;
2. Wall geometry with nails, boundary lines between soil layers, and water surface;
3. Critical failure surface and associated minimum global factor of safety;
4. Distance behind the wall where the failure surface daylights;
5. Search depth below wall toe (if 3-wedge option is used);
6. Surcharge, water table and the horizontal force (PD) on facing boundary lines will be shown;

7. Legend summarizing soil unit weight, strength parameters, water table surface (if used), soil layers ID, vertical and horizontal nail spacing, nail bar yield strength, and facing punching shear capacity (PS on output screen);
8. Height of the wall (H), and reinforcement length (L); and
9. If seismic loading was present, K_H (horizontal seismic coefficient), ratio of vertical-to-horizontal seismic coefficient, and A_c: yield acceleration that gives a FS = 1.0.

Then the user is asked:
Do you want to print the Minimum factors of safety? (Y/N)
If **Y,** analysis results is printed in a report
If **N,** user is asked if previous data needs to be modified

Do you want to edit previous data file for another run? (Y/N)
If **N,** user is asked if another existing file or a new file will be modified

Do you want to edit another file? (Y/N)
If **Y,** filename must be entered. If **N,** the program stops
Nail forces at every nail level is shown for the 10 most critical failure surfaces along with the associated factors of safety and are saved to an output file (ASCII or text file). Additionally, the average nail force that yields a global factor of safety=1.0 is also included in the output file. An abridged output file of an example run is presented below:

```
          RESULTS OF THE MINIMUM SAFETY FACTOR
       FOR SPECIFIED FAILURE PLANES AT THE TOE AND BELOW

          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY     BEHIND        PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                (ft)     (deg)   (ft)    (deg)   (ft)

    Toe    1.354      71.5      26.1   23.9     40.1   65.4

 Reinf. Stress at Level  1 =   0.000 Ksi
                 2 =   4.490 Ksi (Pullout controls...)
                 3 =  10.470 Ksi (Pullout controls...)
                 4 =  16.449 Ksi (Pullout controls...)
                 5 =  22.429 Ksi (Pullout controls...)
                 6 =  30.767 Ksi (Pullout controls...)
                 7 =  39.508 Ksi (Pullout controls...)
                 8 =  43.524 ksi (Punching Shear controls..)

Press <ENTER> to continue...
```

```
          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY     BEHIND        PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                (ft)     (deg)   (ft)    (deg)   (ft)

   NODE  2    1.352      73.0      25.8   24.3     39.6   66.3

 Reinf. Stress at Level  1 =   0.000 Ksi
                 2 =   3.878 Ksi (Pullout controls...)
                 3 =   9.929 Ksi (Pullout controls...)
                 4 =  15.980 Ksi (Pullout controls...)
                 5 =  22.031 Ksi (Pullout controls...)
                 6 =  30.552 Ksi (Pullout controls...)
                 7 =  39.383 Ksi (Pullout controls...)
                 8 =  43.559 ksi (Punching Shear controls..)

Press <ENTER> to continue...
```

...skips intermediate results

```
          MINIMUM    DISTANCE    LOWER FAILURE   UPPER FAILURE
          SAFETY     BEHIND        PLANE           PLANE
          FACTOR     WALL TOE    ANGLE  LENGTH   ANGLE  LENGTH
                      (ft)      (deg)   (ft)    (deg)   (ft)

   NODE  6    1.353       79.0       27.1   35.5     38.5   60.6

 Reinf. Stress at Level  1 =   0.000 Ksi
                    2 =   0.682 Ksi (Pullout controls...)
                    3 =   6.911 Ksi (Pullout controls...)
                    4 =  14.343 Ksi (Pullout controls...)
                    5 =  22.843 Ksi (Pullout controls...)
                    6 =  31.344 Ksi (Pullout controls...)
                    7 =  39.845 Ksi (Pullout controls...)
                    8 =  43.427 ksi (Punching Shear controls..)

Press <ENTER> to continue...

    ***********************************************************************
    *              For Factor of Safety = 1.0                             *
    *    Maximum Horizontally Average Reinforcement Working Force:        *
    *                 22.323 Kips/level                                   *
    ***********************************************************************

Press <ENTER> to continue...
```