STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
16 STATE HOUSE STATION
AUGUSTA, MAINE 04333-0016

Janet T. Mills
GOVERNOR

Bruce A. Van Note
COMMISSIONER

July 17, 2023

Subject: Bridge Rehabilitation
State WIN: 018984.00 & 022362.00
Location: **Caribou & the Forks**
Amendment No. 1

Dear Sir/Ms.:

Make the following changes to the bid documents:

Remove the Geotechnical report dated September 16, 2016 and **Replace** with the attached Geotechnical Reports dated May 26, 2023.

Add the attached Soil Nail Walls Reference Manual

Consider these changes and information prior to submitting your bid on **August 2, 2023**.

Sincerely,

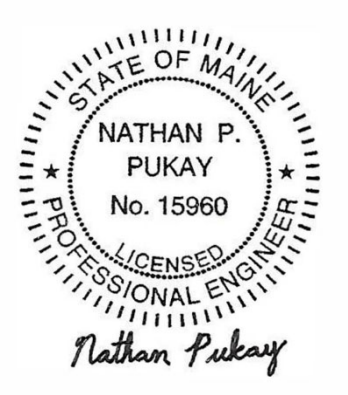
George M. A. Macdougall P.E.
Contracts & Specifications Engineer

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DATA REPORT

For the Rehabilitation of:

CARIBOU STREAM BRIDGE
BRIDGE STREET OVER CARIBOU STREAM
CARIBOU, MAINE



Prepared by:

Nathan Pukay, P.E.
Transportation Engineer II

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Aroostook County
WIN 18984.00

Soils Report 2023-11
Bridge No. 0181

Federal Project No. 1898400
May 26, 2023

Table of Contents

1.0 INTRODUCTION 1

2.0 GEOLOGIC SETTING 1

3.0 SUBSURFACE INVESTIGATIONS 1

4.0 LABORATORY TESTING 2

5.0 SUBSURFACE CONDITIONS 2

5.1 FILL 2
5.2 GLACIAL TILL 3
5.3 BEDROCK 3
5.4 GROUNDWATER 4

6.0 SOIL PROPERTIES 4

7.0 CLOSURE 4

Sheets

Sheet 1 – Location Map
Sheet 2 – Boring Location Plan

Appendices

Appendix A – Boring Logs
Appendix B – Laboratory Test Results
Appendix C – Historical Soils Report

1.0 INTRODUCTION

The purpose of this Geotechnical Data Report is to present subsurface information obtained by Kleinfelder on behalf of Maine Department of Transportation (MaineDOT) for the rehabilitation of a steel bin wall at Caribou Stream Bridge in Caribou, Maine. Additionally, this report provides historical soils information obtained in 1982 by the MaineDOT Soils Section for the construction of the existing bridge structure.

The existing Caribou Stream Bridge carries Bridge Street over Caribou Stream and was constructed in 1983. The structure consists of a single 25-foot span by 20-foot rise, concrete rigid frame bridge bearing on spread footings cast on bedrock. According to the 2020 MaineDOT Bridge Inspection Report, the bridge is in overall good condition, but an adjacent steel bin wall supporting the Northwest embankment is in serious condition with extensive rusting, holes, and areas of settlement. The bin wall predates the bridge, but the exact construction date is not known. The scope of the project is to stabilize the existing bin wall with a soil nail wall, cover the bin wall face with concrete (or similar), install additional riprap at the base of the wall for scour protection, and make other minor improvements at the project site.

2.0 GEOLOGIC SETTING

Caribou Stream Bridge carries Bridge Street over Caribou Stream as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the Presque Isle 1° x 2° Quadrangle, Maine, Open File No. 87-15 (1987) indicates the surficial soils in the vicinity of the project site consist primarily of glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones, may include many boulders, generally massive, but in many places contains beds and lenses of variably washed and stratified sediments.

The MGS Bedrock Geology of Maine (1985) maps the bedrock in the vicinity of the project as interbedded pelite and limestone and/or dolostone of the Spragueville Formation.

3.0 SUBSURFACE INVESTIGATIONS

Two contemporary test borings explored subsurface conditions at the project location. The borings were drilled between the existing steel bin wall and Water Street. Boring RW-CAR-101 was drilled approximately 10 feet behind the face of the bin wall, adjacent to an existing chain-link fence, and terminated after coring bedrock. Boring RW-CAR-102 was drilled approximately 10 feet further from the bin wall, near the end of the guardrail off Water Street. The boring was advanced to 24 feet below the ground surface before ending the exploration after refusal of the hollow stem auger. The test boring locations are shown on Sheet 2 – Boring Location Plan.

The test borings were drilled in July 2016 by New England Boring Contractors of Hermon, Maine. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs.

The borings were performed by using a combination of solid stem auger, hollow stem auger, cased wash boring and rock coring techniques. The borings were completed by backfilling and compacting the borehole with drill cuttings. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in February 2016. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.57 to the raw field N-values. This hammer efficiency factor (0.57) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

A Kleinfelder geoprofessional selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field-testing requirements, and reviewed the field logs for accuracy. The borings were located in the field using taped measurements at the completion of the drilling program, and ground surface elevations were determined by interpolating between contour lines provided by MaineDOT Survey.

An additional eleven (11) test borings were drilled in 1982 by the MaineDOT Soils Section for the construction of the existing Caribou Stream Bridge. Subsurface conditions encountered in the 1982 test borings, boring details, boring locations and interpretive subsurface profiles and cross sections are provided in MaineDOT Soils Report 82-32. This soils report is provided in Appendix C – Historical Soils Report.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the 2016 test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of four standard grain size analysis and one unconfined compressive strength rock test. The results of soil and rock tests are included as Appendix B – Laboratory Test Results. Soil and rock test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the 2016 test borings generally consisted of Fill, Glacial Till, and Limestone. The boring logs are provided in Appendix A – Boring Logs. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill

A layer of fill was encountered in the test borings. The thickness of the fill material was approximately 5 to 22 feet. The fill materials encountered were variable and consisted of:

- Grey-brown and brown, moist, SAND, some to trace gravel, trace to little silt;
- Grey-brown, moist, GRAVEL, trace sand, trace silt;
- Grey-brown, wet, Sandy GRAVEL, little silt;
- Asphalt pieces and wood fragments.

Corrected SPT N-values in the fill layer ranged from 13 to 30 blows per foot (bpf), indicating the fill is medium dense in consistency. One grain size analysis conducted on samples of the fill resulted in the material being classified as A-1-b under the AASHTO Soil Classification System and GM under the Unified Soil Classification System (USCS).

5.2 Glacial Till

Glacial Till was encountered in the test borings beneath the fill soils. The encountered thickness was approximately 5 to 19 feet. The deposit consisted of:

- Grey-brown, moist to wet, Sandy GRAVEL, some to little silt;
- Grey-brown, moist, SAND, some gravel, trace silt;
- Light brown-grey, moist, Silty SAND, some gravel;
- Grey-brown, wet, Silty SAND, little gravel.

Corrected SPT N-values ranged from 22 to in excess of 50 bpf, indicating the glacial till deposit is medium dense to very dense in consistency. Three grain size analyses resulted in the material being classified as A-1-b and A-4 under the AASHTO Soil Classification System and GM and SM under the USCS.

5.3 Bedrock

Bedrock was encountered and cored in boring RW-CAR-101 at 27.0 feet below the ground surface. Boring RW-CAR-102 was terminated at 24.0 feet below the ground surface after refusal of the hollow stem auger, indicating possible bedrock. The following table summarizes approximate depth to bedrock, corresponding approximate top of bedrock elevation, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)	RQD (%)	
RW-CAR-101	1+82.44	35.72 Rt	27.0	381.1	R1	0
					R2	0
					R3	0
					R4	27
					R5	0

The bedrock at the site is identified as grey, fine-grained, Limestone, with moderate to severe weathering, and steeply dipping joints with calcite infilling. The RQD's of the bedrock cores ranged from 0 to 27 percent corresponding to a rock quality of very poor to poor. Detailed bedrock descriptions are provided on the boring logs in Appendix A – Boring Logs.

5.4 Groundwater

Groundwater was measured at depths ranging from 22 to 24 feet below the ground surface upon completion of the borings. Note that water was introduced into the boreholes during drilling operations and the measured levels may not represent stabilized groundwater elevations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 SOIL PROPERTIES

The following table provides recommended soil properties. These soil properties are provided for informational purposes. Soil properties for the soil nail wall design shall be determined by the soil nail wall designer based on the available subsurface information and previous design experience.

Soil Unit	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Granular Fill	120	34	0
Glacial Till	132	36	0

7.0 CLOSURE

This Geotechnical Data Report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed bin wall rehabilitation project at the Caribou Stream Bridge in Caribou, Maine. The report was prepared in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

MaineDOT shall not be responsible for the bidder's or contractor's interpretations of, estimates or conclusions derived from the geotechnical information. Data provided may not be representative of the subsurface conditions between boring locations.

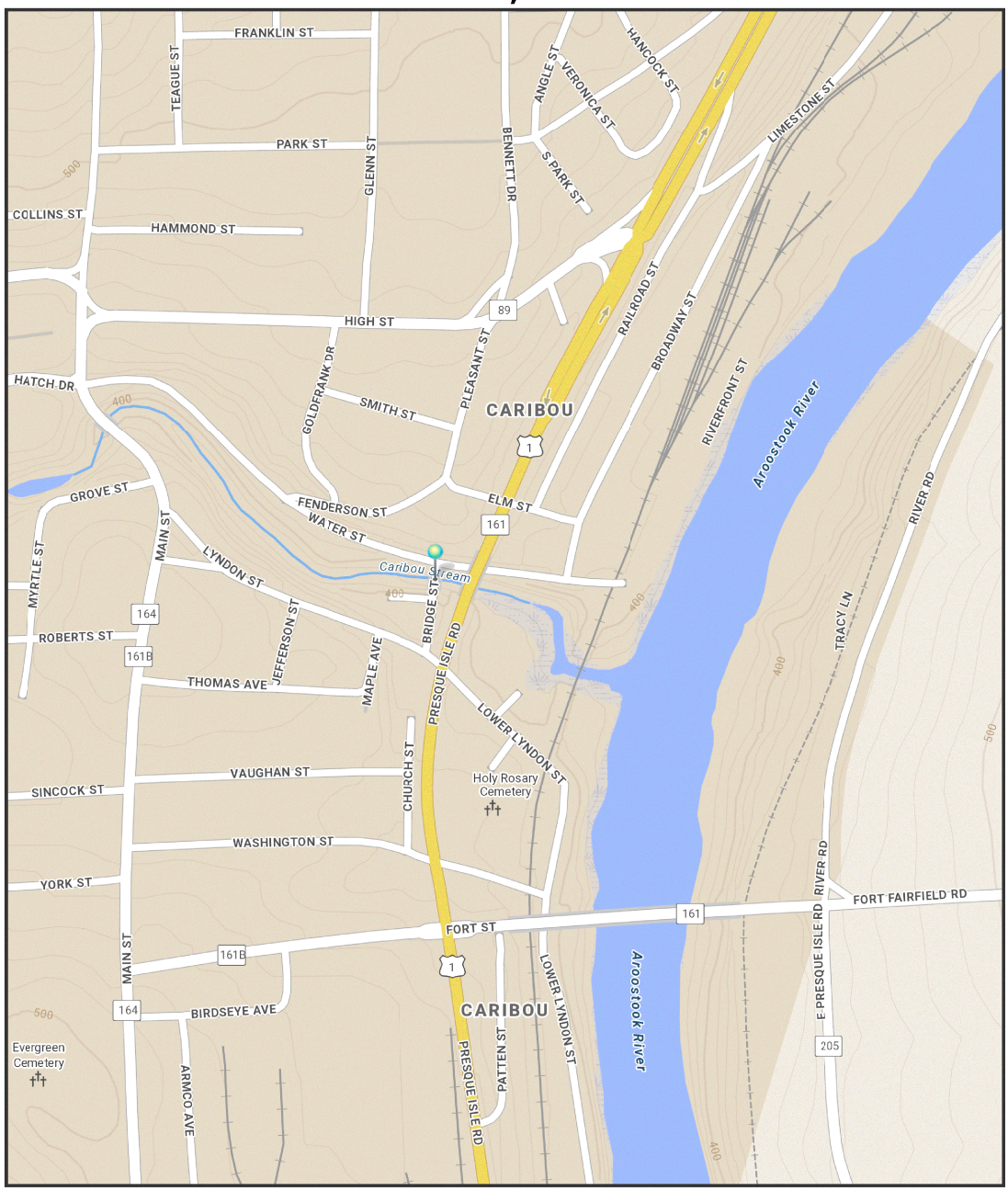
Sheets

Date:5/10/2023

Division: GEOTECH

Username: Nathan.P.Pukay

CARIBOU, MAINE



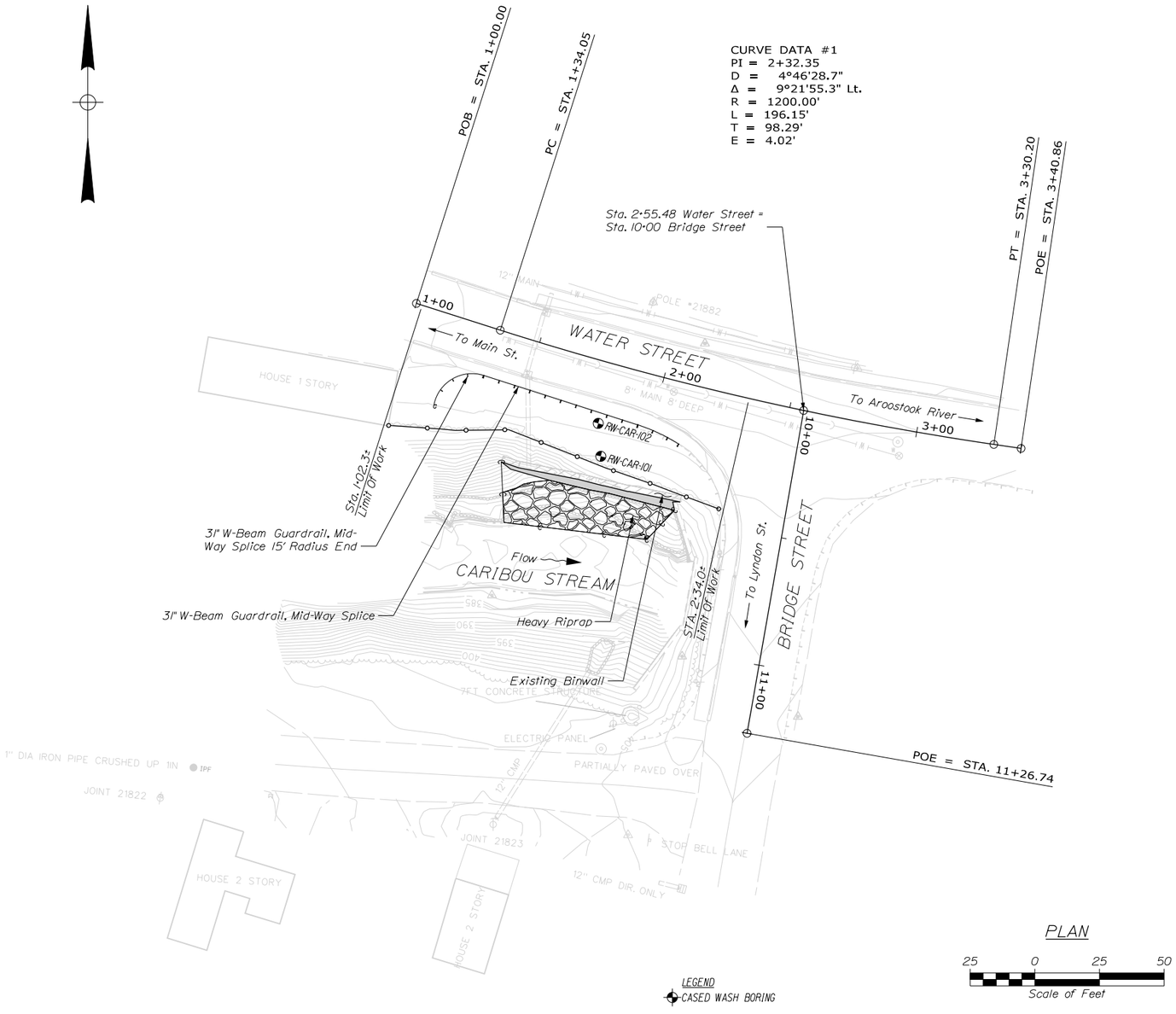
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0.1 Miles

1 inch = 0.14 miles

Date: 5/8/2023
Time: 8:21:29 AM

SHEET NUMBER 1 OF 2	CARIBOU STREAM BRIDGE CARIBOU STREAM CARIBOU AROOSTOOK COUNTY LOCATION MAP	STATE OF MAINE DEPARTMENT OF TRANSPORTATION 18984.00 WIN BRIDGE NO. 0181 18984.00 BRIDGE PLANS



PROJ. MANAGER	MIKE WIGHT	BY	DATE
DESIGN-DETAILED	M. GRAY	R. MAYER	JUNE 2023
CHECKED-REVIEWED			SIGNATURE
DESIGN2-DETAILED2	N.PUKAY	T. WHITE	MAY 2023
DESIGN3-DETAILED3			P.E. NUMBER
REVISIONS 1			
REVISIONS 2			DATE
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

1898400

WIN 18984.00

BRIDGE NO. 0181 BRIDGE PLANS

CARIBOU STREAM BRIDGE
CARIBOU STREAM
AROOSTOOK COUNTY
CARIBOU

BORING LOCATION PLAN

SHEET NUMBER

2

OF 2

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Nathan.P.Pukay Date:5/30/2023

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Retaining Wall Replacement, Adjacent to Bridge #0181
Location: Bridge St @ Water St Caribou, ME

Boring No.: RW-CAR-101
WIN: 018984.00

Driller:	N.E.B.C	**Elevation (ft.)**	408.05	**Auger ID/OD:**	4" Solid Stem
Operator:	Chris/Devon	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Madden	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	7/26/2016-7/27/2016	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	NX Core
Boring Location:	1+82.44, 35.72R	**Casing ID/OD:**	4.5"/4"	**Water Level*:**	24'3"
Hammer Efficiency Factor:	0.572	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
				Sample Information							
0	S1	24/6	0.00 - 2.00	1-7-10-14	17	16	SSA			Top 2" grass/topsoil Brown, moist, medium dense SAND, some subangular gravel, trace silt, fine-coarse grained sand, fine-coarse grained gravel, non-plastic fines, SW-SM, trace root material and wood (FILL)	
5	S2	24/14	5.00 - 7.00	5-14-10-14	24	23				Brown, moist, medium dense SAND, some subangular gravel, trace silt, fine-coarse grained sand, fine-coarse grained gravel, non-plastic fines, SW-SM, asphalt pieces, wood (FILL)	
	S3	24/8	7.00 - 9.00	12-10-9-10	19	18				Greyish brown, moist, medium dense GRAVEL, trace sand, trace silt, angular to subangular, fine-coarse grained gravel, fine sand, non-plastic fines, GW, (FILL)	
10	S4	24/6	10.00 - 12.00	13-7-9-8	16	15	Wash			Greyish brown, moist, medium dense GRAVEL, trace silt, trace sand, angular to subangular, fine-coarse grained gravel, fine sand, non-plastic fines, GW, (FILL)	
15	S5	24/6	15.00 - 17.00	11-7-7-9	14	13				Greyish brown, wet, medium dense, sandy GRAVEL, little silt, angular to subangular, fine-coarse grained gravel, fine sand, non-plastic fines, GM, (FILL)	G#386514 A-1-b, GM
20	S6	24/6	20.00 - 22.00	7-13-9-9	22	21				Greyish brown, wet, medium dense GRAVEL, trace silt, trace sand, angular to subangular, fine grained gravel, non-plastic fines, GP, majority of sample is wood fragments, up to 3" long (FILL),	
								386.1		22.0	
	S7	24/3	22.00 - 24.00	19-19-21-20	40	38				Brownish grey, wet, dense, sandy GRAVEL, some silt, fine-coarse grained gravel, angular to subangular, fine-medium sand, non-plastic fines, GM (GLACIAL TILL)	
25											

Remarks:

Water level measured at 7:15 AM on 7/27/16 with casing installed to 27 feet.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Retaining Wall Replacement, Adjacent to Bridge #0181
Location: Bridge St @ Water St Caribou, ME

Boring No.: RW-CAR-101
WIN: 018984.00

Driller:	N.E.B.C	**Elevation (ft.)**	408.05	**Auger ID/OD:**	4" Solid Stem
Operator:	Chris/Devon	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Madden	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	7/26/2016-7/27/2016	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	NX Core
Boring Location:	1+82.44, 35.72R	**Casing ID/OD:**	4.5"/4"	**Water Level*:**	24'3"
Hammer Efficiency Factor:	0.572	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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WO1P = Weight of One Person
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	S8	9/6	25.00 - 25.75	50-100/3"	>100					Brownish grey, wet, dense, sandy GRAVEL, some silt, fine-coarse grained gravel, angular to subangular, fine-medium sand, non-plastic fines, GM (GLACIAL TILL)	G#386515 A-1-b, GM
								381.1		27.0	
	R1	30/30	27.00 - 29.50	RQD = 0%			NX			R1: Grey, fine-grained, <1" to 8" fragments of fractured limestone, moderate to severe weathering, steep dipping (60°), calcite infilling, rock mass quality: very poor. 100% Recovery. R1: Core Times (min/ft) 1:45/4:43/3:56	
	R2	18/18	29.50 - 31.00	RQD = 0%						R2: Grey, fine-grained, <1" to 5" fragments of fractured limestone, moderate to severe weathering, steep dipping (60°), calcite infilling, clay/silt infilling, rock mass quality: very poor. 100% Recovery. R2: Core Times (min/ft) 1:37/1:52	
30	R3	18/18	31.00 - 32.50	RQD = 0%						R3: Grey, fine-grained, <1" to 9" fragments of fractured limestone, moderately severe weathering, steep dipping (60°), calcite infilling, clay/silt infilling, rock mass quality: very poor. 100% Recovery. R3: Core Times (min/ft) 3:14/2:05	qp = 1981.87 ksf
	R4	33/33	32.50 - 35.25	RQD = 27%						R4: Grey, fine-grained, <1" to 10" fragments of fractured limestone, moderate to severe weathering, steep dipping (60°), calcite infilling, rock mass quality: poor. 100% Recovery. R4: Core Times (min/ft) 4:42/3:35/2:12	
35	R5	21/21	35.25 - 37.00	RQD = 0%						R5: Grey, fine-grained, <1" to 8" fragments of fractured limestone, moderate to severe weathering, steep dipping (60°), calcite infilling, clay infilling, rock mass quality: very poor. 100% Recovery. R5: Core Times (min/ft) 2:14/2:52	
								371.1		37.0	
										Bottom of Exploration at 37.0 feet below ground surface.	
40											
45											
50											

Remarks:

Water level measured at 7:15 AM on 7/27/16 with casing installed to 27 feet.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: RW-CAR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Retaining Wall Replacement, Adjacent to Bridge #0181
Location: Bridge St @ Water St Caribou, ME

Boring No.: RW-CAR-102
WIN: 018984.00

Driller:	N.E.B.C	**Elevation (ft.)**	409.30	**Auger ID/OD:**	2.25"/5.625"
Operator:	Chris/Devon	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Madden	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	7/27/2016	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	
Boring Location:	1+80.20, 23.69R	**Casing ID/OD:**		**Water Level*:**	22'
Hammer Efficiency Factor:	0.572	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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WOR/C = Weight of Rods or Casing
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
				Sample Information							
0	S1	24/12	0.00 - 2.00	1-6-12-11	18	17	HSA			Top 3" grass/topsoil Greyish brown, moist, medium dense, SAND, little silt, little subrounded gravel, fine-coarse grained sand, fine grained gravel, slightly plastic fines, SM, (POSSIBLE FILL)	
	S2	24/14	2.00 - 4.00	15-14-17-22	31	30				Greyish brown, moist, medium dense, SAND, little silt, trace subrounded gravel, fine-coarse grained sand, fine gravel, slightly plastic fine, SM, (POSSIBLE FILL)	
5	S3	24/11	5.00 - 7.00	30-19-18-12	37	35		404.3		5.0 Greyish brown, moist, dense, sandy GRAVEL, little silt, fine subrounded gravel, fine-coarse sand, non-plastic fines, GM, (GLACIAL TILL), broken cobbles fragments at 5 feet	G#386516 A-1-B, GM
	S4	24/NR	7.00 - 9.00	10-11-12-12	23	22	HSA			Recovered only broken cobble fragments (also in sampler tip)	
10	S5	24/6	10.00 - 12.00	12-14-20-20	34	32		399.3		10.0 Greyish brown, moist, dense SAND, some subrounded gravel, trace silt, fine-coarse sand, fine-coarse gravel, non-plastic to slightly plastic fines, SW-SM, (GLACIAL TILL)	
15	S6	24/6	15.00 - 17.00	9-12-14-16	26	25	HSA	394.3		15.0 Light brownish grey, moist, medium dense, silty SAND, some gravel, fine-coarse grained sand, fine-coarse grained gravel, slightly plastic fines, SM (GLACIAL TILL)	
20	S7	24/14	20.00 - 22.00	19-21-32-49	53	51				Greyish brown, moist, wet, very dense, silty SAND, little subangular to subrounded gravel, fine-coarse sand, fine-coarse gravel, slightly plastic fines, SM (GLACIAL TILL)	G#386517 A-4, SM
25	S8	0/0	24.00 - 24.00	N/A				385.3		24.0 **Bottom of Exploration at 24.0 feet below ground surface.** Split spoon bouncing on possible bedrock at 24 feet	

Remarks:

Water level measured at 22 feet B.G.S. on 7/27/16 following auger refusal at 24 feet.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Appendix B

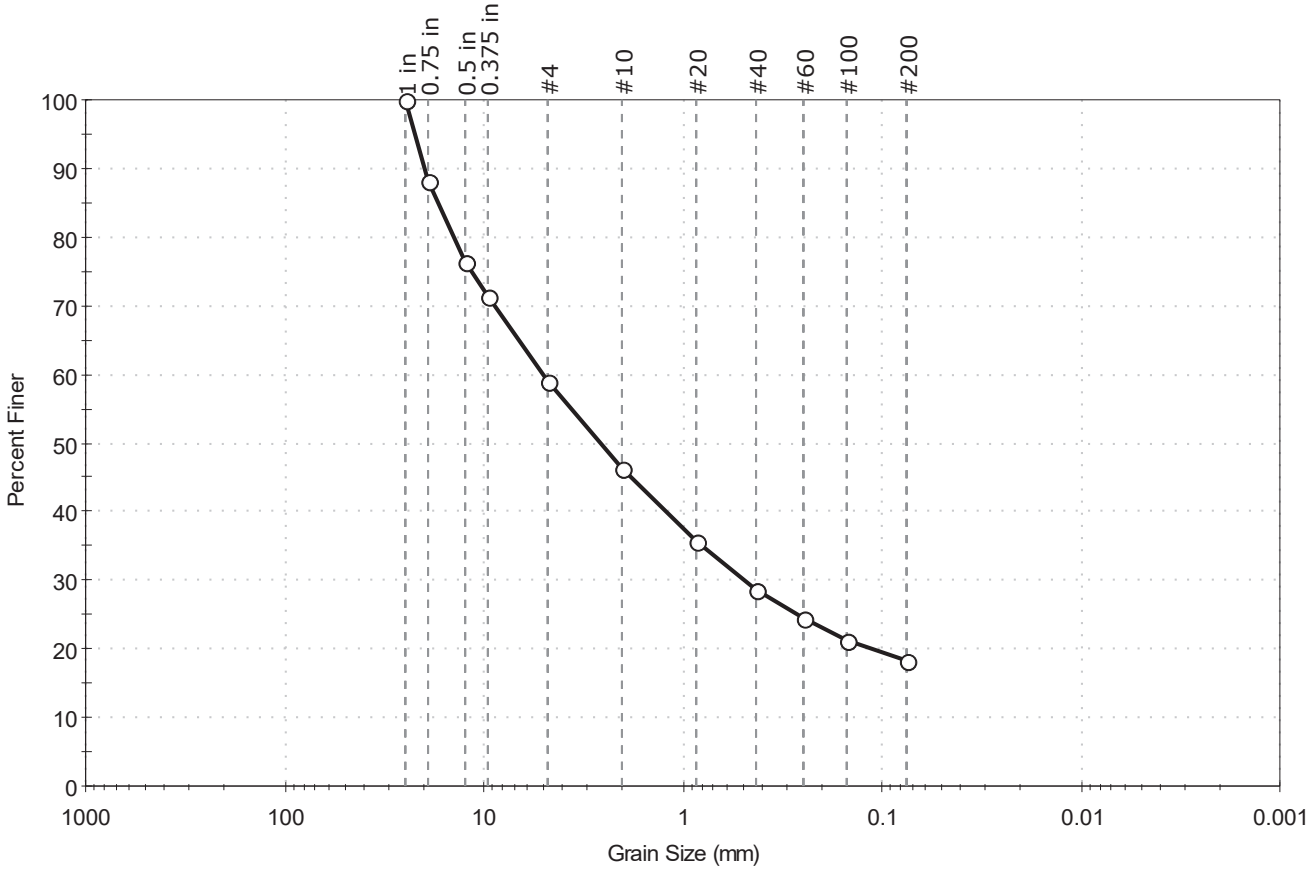
Laboratory Test Results

GeoTesting EXPRESS

Client:	Kleinfelder				
Project:	Caribou Retaining Wall Replacement				
Location:	Caribou, ME			Project No:	GTX-305151
Boring ID:	RW-CAR-101	Sample Type:	bag	Tested By:	jbr
Sample ID:	S5	Test Date:	08/11/16	Checked By:	jsc
Depth :	15-17 ft	Test Id:	386514		

Test Comment: ---
Visual Description: Moist, olive brown silty gravel with sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	41.0	40.7	18.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	88		
0.5 in	12.50	76		
0.375 in	9.50	71		
#4	4.75	59		
#10	2.00	46		
#20	0.85	36		
#40	0.42	29		
#60	0.25	24		
#100	0.15	21		
#200	0.075	18		

Coefficients

D_{85} = 16.9233 mm
D_{30} = 0.4833 mm
D_{60} = 5.0277 mm
D_{15} = N/A
D_{50} = 2.5853 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM N/A

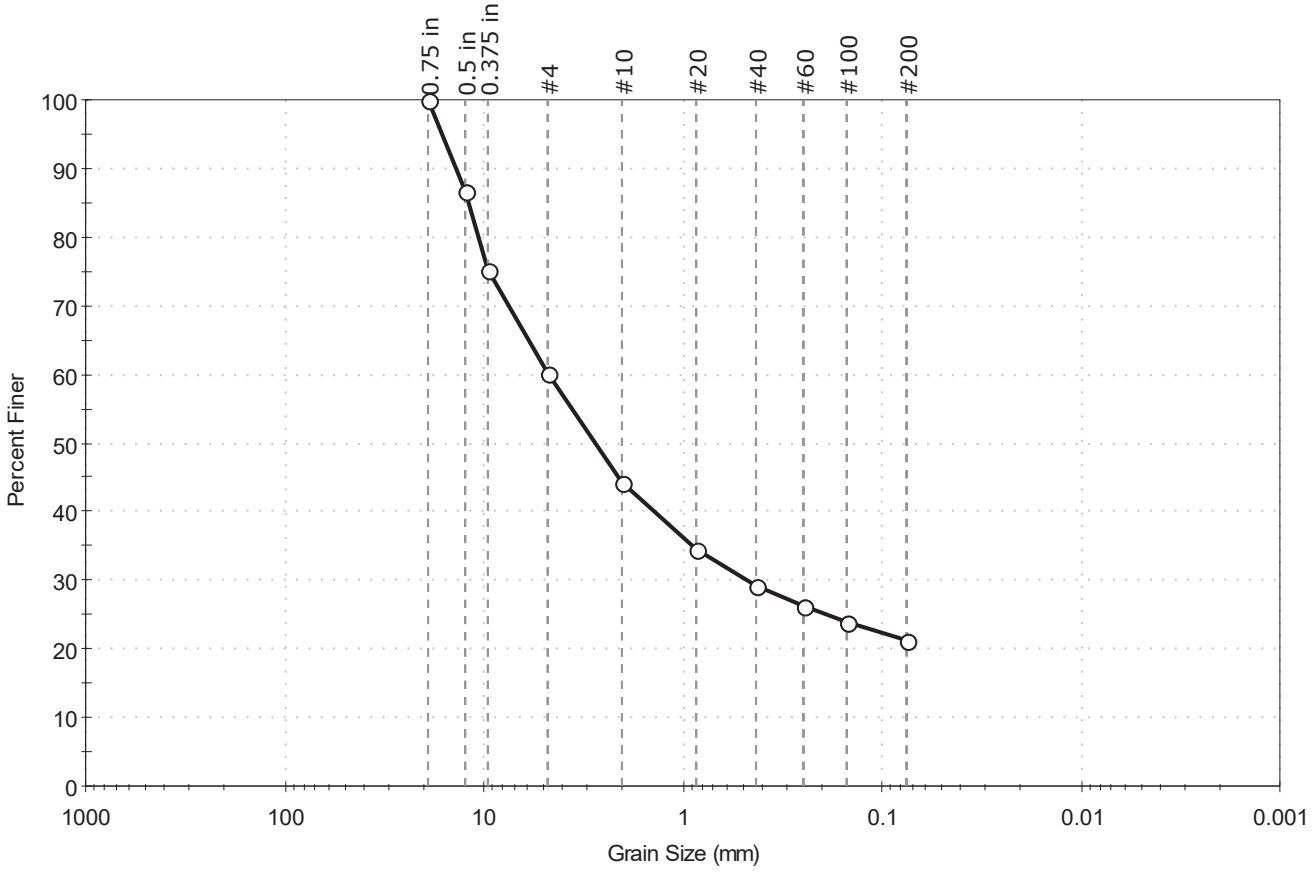
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD

Client:	Kleinfelder				
Project:	Caribou Retaining Wall Replacement				
Location:	Caribou, ME			Project No:	GTX-305151
Boring ID:	RW-CAR-101	Sample Type:	bag	Tested By:	jbr
Sample ID:	S8	Test Date:	08/11/16	Checked By:	jsc
Depth :	25-25.8 ft	Test Id:	386515		
Test Comment:	---				
Visual Description:	Moist, olive silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.7	39.1	21.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	87		
0.375 in	9.50	75		
#4	4.75	60		
#10	2.00	44		
#20	0.85	35		
#40	0.42	29		
#60	0.25	26		
#100	0.15	24		
#200	0.075	21		

Coefficients

D_{85} = 11.9728 mm D_{30} = 0.4644 mm

D_{60} = 4.6884 mm D_{15} = N/A

D_{50} = 2.7371 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

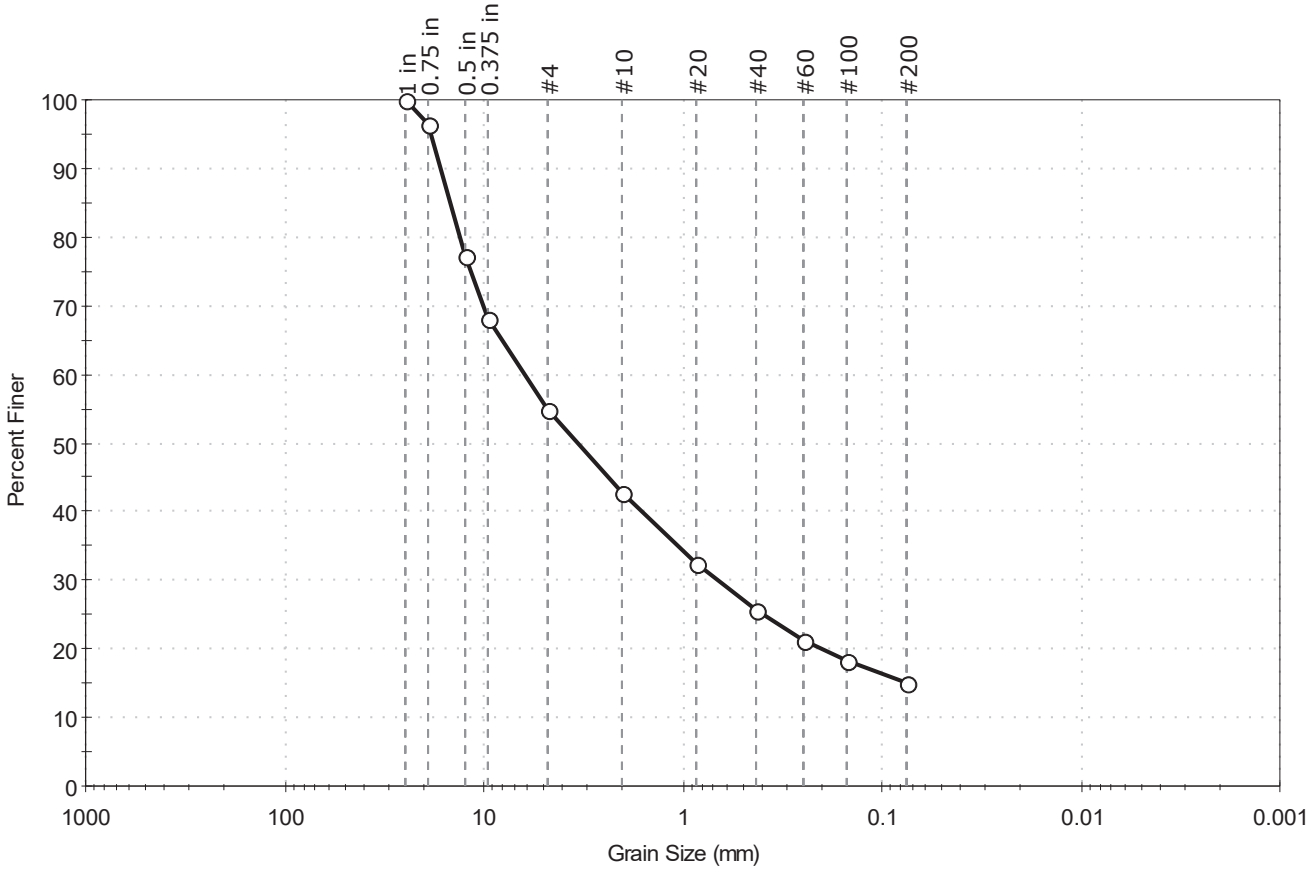
Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

Client:	Kleinfelder				
Project:	Caribou Retaining Wall Replacement				
Location:	Caribou, ME			Project No:	GTX-305151
Boring ID:	RW-CAR-102	Sample Type:	bag	Tested By:	jbr
Sample ID:	S3	Test Date:	08/11/16	Checked By:	jsc
Depth :	5-7 ft	Test Id:	386516		
Test Comment:	---				
Visual Description:	Moist, olive silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	45.3	39.7	15.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	77		
0.375 in	9.50	68		
#4	4.75	55		
#10	2.00	43		
#20	0.85	32		
#40	0.42	26		
#60	0.25	21		
#100	0.15	18		
#200	0.075	15		

Coefficients

D_{85} = 14.7803 mm	D_{30} = 0.6697 mm
D_{60} = 6.2393 mm	D_{15} = N/A
D_{50} = 3.3827 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

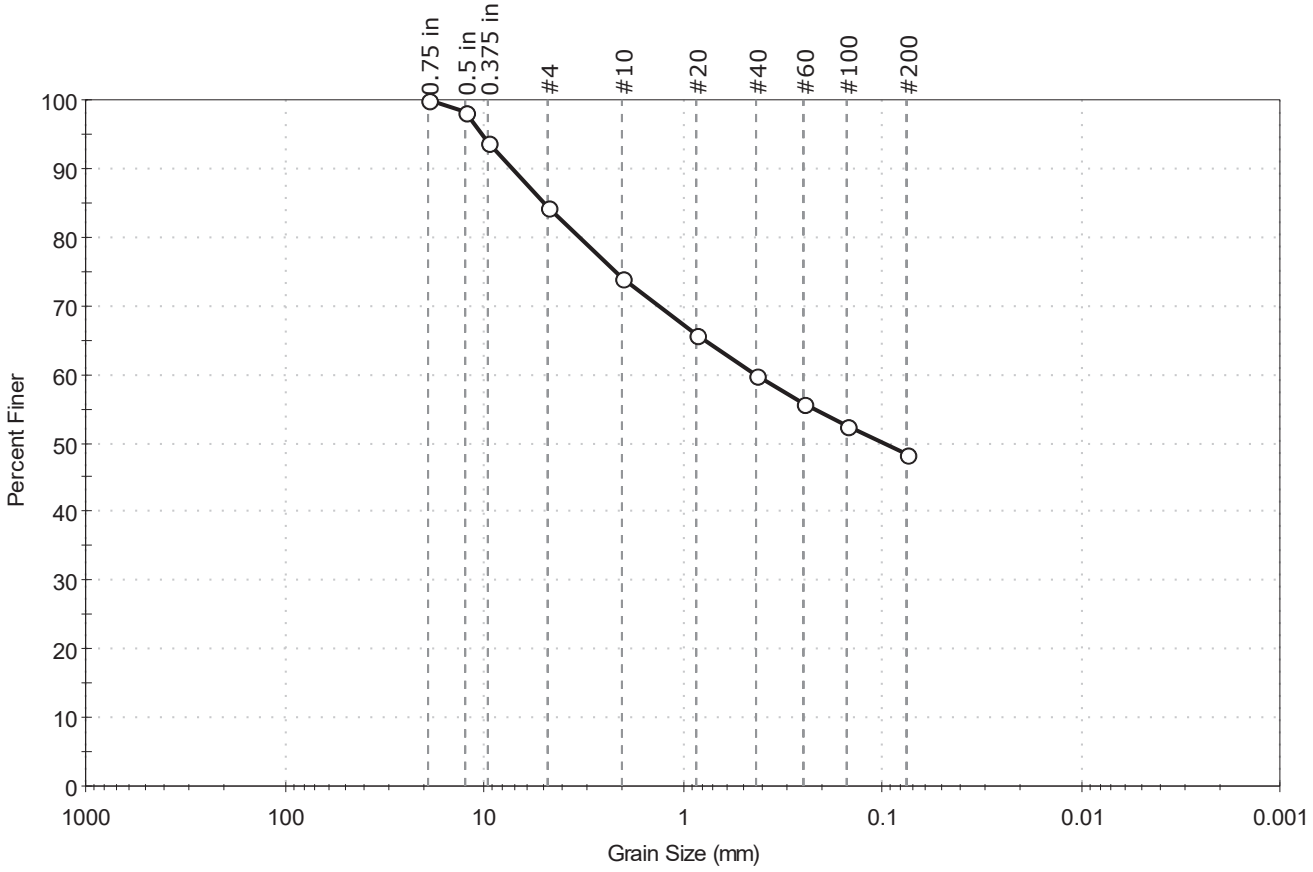
Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

Client:	Kleinfelder				
Project:	Caribou Retaining Wall Replacement				
Location:	Caribou, ME			Project No:	GTX-305151
Boring ID:	RW-CAR-102	Sample Type:	bag	Tested By:	jbr
Sample ID:	S7	Test Date:	08/11/16	Checked By:	jsc
Depth :	20-22 ft	Test Id:	386517		
Test Comment:	---				
Visual Description:	Moist, olive clayey sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	15.6	35.9	48.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	94		
#4	4.75	84		
#10	2.00	74		
#20	0.85	66		
#40	0.42	60		
#60	0.25	56		
#100	0.15	52		
#200	0.075	49		

Coefficients

D_{85} = 4.9616 mm D_{30} = N/A

D_{60} = 0.4266 mm D_{15} = N/A

D_{50} = 0.0977 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

Client:	Kleinfelder				
Project:	Caribou Retaining Wall Replacement				
Location:	Caribou, ME			Project No:	GTX-305151
Boring ID:	RW-CAR-101	Sample Type:	cylinder	Tested By:	daa/rlc
Sample ID:	R4	Test Date:	08/12/16	Checked By:	jsc
Depth :	33'9"-34'8"	Test Id:	386490		
Test Comment:	---				
Visual Description:	See photograph(s)				
Sample Comment:	---				

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
RW-CAR-101	R4	33'9"-34'8"	170	13,763	1	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens tested at the approximate as-received moisture content and at standard laboratory temperature.

The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure (See attached photographs)

Client:	Kleinfelder	Test Date:	8/10/2016
Project Name:	Caribou Retaining Wall Replacement	Tested By:	daa/rlc
Project Location:	Caribou, ME	Checked By:	jsc
GTX #:	305151		
Boring ID:	RW-CAR-101		
Sample ID:	R4		
Depth:	33'9"-34'8"		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.25	4.25	4.25
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	590.82		
Bulk Density, lb/ft^3	170	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00040
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00010

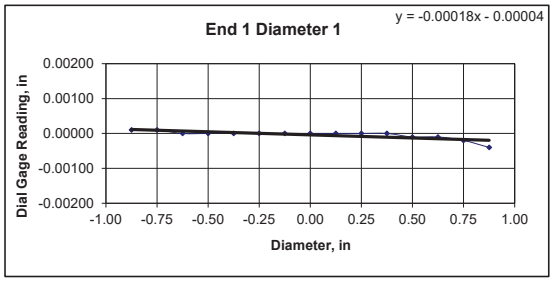
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

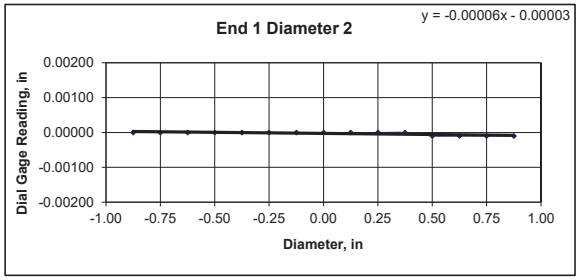
Difference between max and min readings, in:

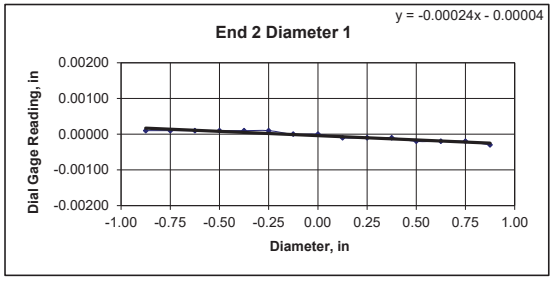
0° = 0.0004 90° = 0

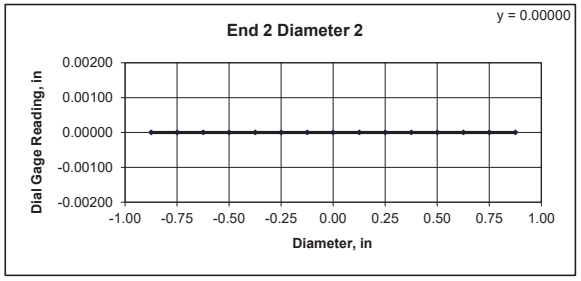
Maximum difference must be < 0.0020 in. Difference = ± 0.00025

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line -0.00018

Angle of Best Fit Line: -0.01031

End 2:

Slope of Best Fit Line -0.00024

Angle of Best Fit Line: -0.01375

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line -0.00006

Angle of Best Fit Line: -0.00344

End 2:

Slope of Best Fit Line 0.00000

Angle of Best Fit Line: 0.00000

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES

Spherically Seated

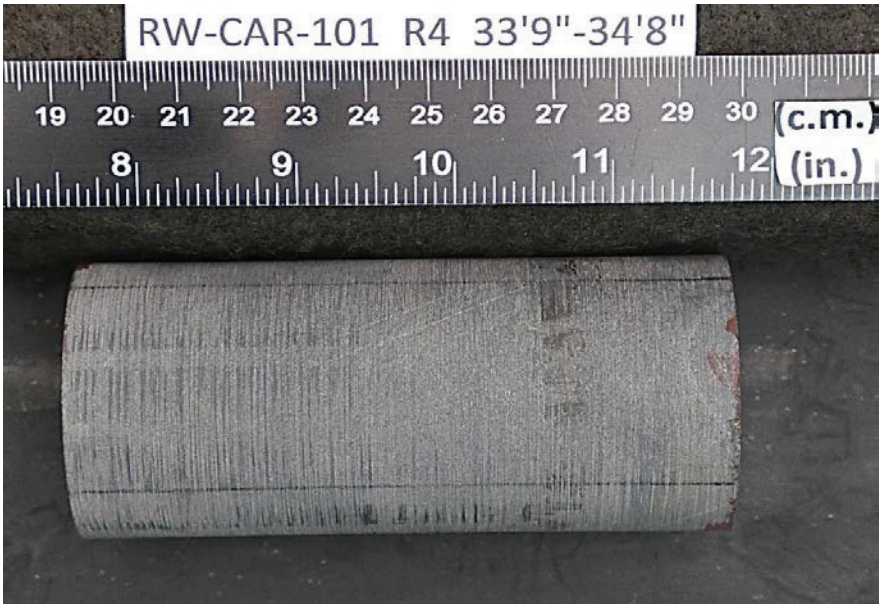
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.990	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00000	1.990	0.00000	0.000	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Kleinfelder
Project Name:	Caribou Retaining Wall Replacement
Project Location:	Caribou, ME
GTX #:	305151
Test Date:	8/12/2016
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	RW-CAR-101
Sample ID:	R4
Depth:	33'9"-34'8"



After cutting and grinding



After break

Appendix C

Historical Soils Report

CARIBOU 99 3 32

Soils Report 82-32

Caribou - Aroostook County
Project 99-3(32)
Cross Street Over Caribou Stream

September 1982

82 32

Maine Department of Transportation

Materials & Research Division

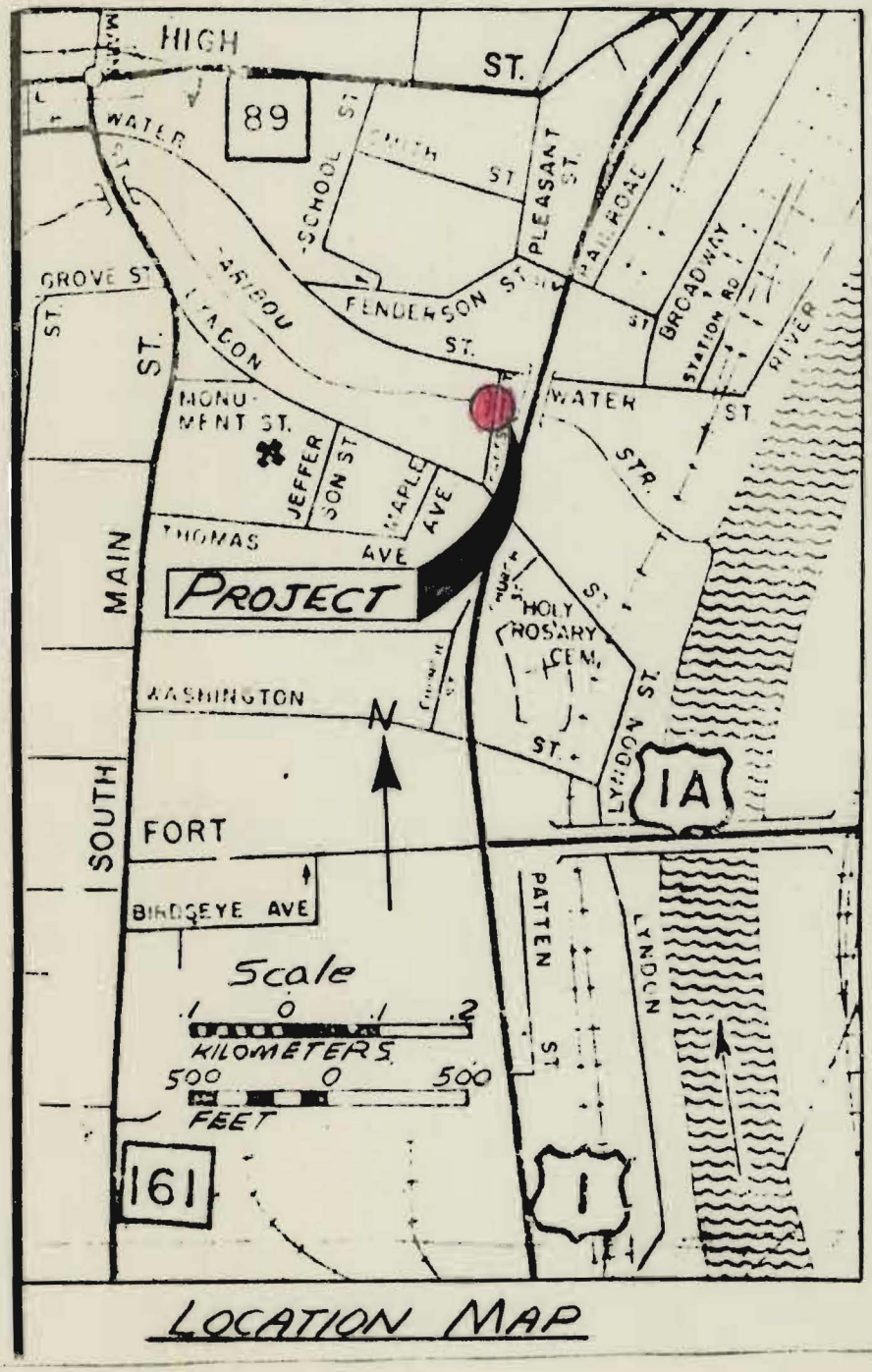
Soils Section

SUBSURFACE INVESTIGATION FOR THE PROPOSED CONSTRUCTION OF A STRUCTURE TO CARRY CROSS STREET OVER CARIBOU STREAM IN THE TOWN OF CARIBOU

Aroostook County

Project 99-3(32)
September 1982

Soils Report 82-32



LOCATION MAP

TABLE OF CONTENTS

Text	Page No.
INTRODUCTION.	1
GENERAL CONDITIONS.	1
SUBSURFACE CONDITIONS	2
CONCLUSIONS AND RECOMMENDATIONS	3
SUMMARY	4

Illustrations	Sheet No.
Allowable Bearing Capacity Chart.	1
Plan, Profile and Transverse Sections	2
Washboring Details.	3

INTRODUCTION

Several washborings have been completed at the site of Caribou Stream Bridge in Caribou where it is proposed to replace a structure which carried Cross Street over Caribou Stream until it was washed out during a flood. A total of eleven washborings were made at this site by Gary Paine's boring crew. Initially, four washborings were made; two near each abutment location in mid March. Because of the significant difference in ledge elevation from one abutment location to the other it was believed additional borings would be required but high fast water rendered additional washborings impossible at that time. In mid July the crew returned to the site and made several additional washborings to better define the ledge surface.

All soil samples and rock cores obtained in the washborings were forwarded to the Central Laboratory in Bangor for testing and identification. Foundation survey sheets were prepared indicating boring locations, a stratified soils profile along centerline and several stratified transverse sections plus details of the individual washborings. The originals of these sheets will be provided to the Design Division for inclusion in the construction plans, and copies are included in the illustrations of this report.

GENERAL CONDITIONS

Ledge is near streambed elevation on the south side of Caribou Stream, but drops to 15 feet below streambed beneath the north side. The southerly abutment of the old structure still stands, but the northerly abutment was washed out by flood waters.

The native soil above the ledge surface which varies in thickness from only one foot to 12.5 feet was described in the field as brown gravel and rocks or more often as brown silty till. Samples were predominantly described as brown silty sand or silty sand and gravel. Some samples contained some clay.

The volume and velocity of water through this bridge site varies considerable through the year. When the washborings were made in mid July, there was scarcely any water in the stream; yet last year there was sufficient volume and velocity to wash out the bridge.

SUBSURFACE CONDITIONS

Five transverse sections were developed using the washboring results to interpret the soils stratification and ledge elevation. What these transverse sections reveal is that the ledge elevation is quite uniform across each section, but slopes down from south to north.

At Station 12+70 (Section A-A) behind the abutment of the old bridge, there is about 10 feet of fill consisting of loose to medium density brown silty sand and gravel which is underlain by loose brown silty gravel. Ledge underlies this silty gravel being virtually flat between centerline and 50 feet right at elevation 389.5, and sloping down to elevation 387.5 at 50 feet left.

At Station 12+95 (Section B-B) about twelve feet in front of the old abutment the ledge surface is nearly level at elevation 383± and is covered by only a foot, more or less, of sandy gravel and rocks. It was reported that ledge can be seen beneath the old abutment which was apparently built directly on ledge.

At Station 13+07 (Section C-C) the ledge surface was encountered at elevation 75± beneath five feet of brown silty pebbly till.

At Station 13+17 (Section D-D) there is 12± feet of dense brown till over the ledge surface. Again, the ledge surface is virtually flat across this section at elevation 368.5±.

The two borings made at Station 13+26 (Section E-E) encountered very rocky material described as brown gravel and rocks, and the rock core bit was used to drill more than five feet of rock in each of these holes. However, most of the

rock core from these two borings was described by our Materials Geologist as assorted rocks and cobbles and boulders with only the bottom portion of the rock core being perceived as possibly being ledge. The ledge surface appears to slope down from elevation 373.5 fifteen feet left of centerline to elevation 371.5 thirteen feet right of centerline.

The five transverse sections discussed above are shown on sheet 2, and the details of the individual washborings are shown on sheet 3. Using these transverse sections, a stratified profile along centerline was developed and is also shown on sheet 2. These sections and profile indicate that at any given station along the centerline the ledge surface could be expected to be relatively uniform in elevation across the section. The ledge surface apparently slopes downward from south toward north across Caribou Stream, and the set of borings made at Station 13+26 on the north side of the stream indicate that the ledge surface may be coming back up.

CONCLUSIONS AND RECOMMENDATIONS

Because of the slope of the ledge surface, it appears that one abutment of the structure can be supported directly on ledge, but that the other cannot be built on ledge. Options for support of the northerly abutment include: a spread footing in the till, short heavy cross section steel H-piles driven to ledge, concrete pedestals to ledge, or removal of the till above ledge and replacement with concrete fill.

Though the option of a spread footing is believed least desirable, allowable bearing capacity curves are included on sheet 1. As can be seen from these curves, the footing pressure on such a spread footing could not be much higher than 2 tons per square foot. Also, with one side of the structure supported directly on ledge and the other supported on soil the potential for differential movement and the cracking of the structure is high. Thus, it is recommended that the northerly

abutment of this structure be supported on the ledge surface either through the use of short piles or columns or the use of a concrete fill extending from footing elevation to ledge surface.

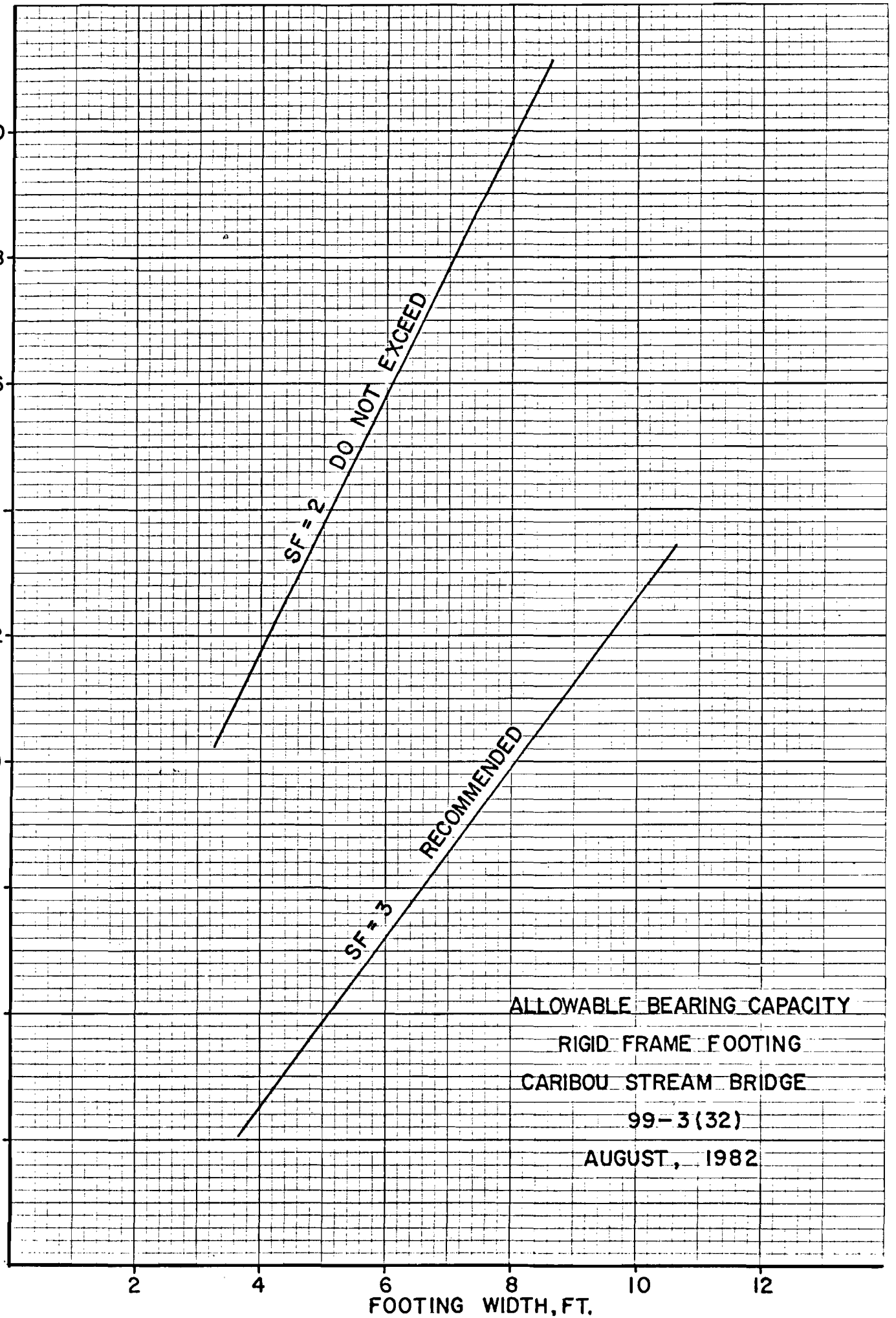
SUMMARY

Eleven washborings were made at a bridge site in Caribou where Cross Street crosses Caribou Stream. The proposed structure for this site will replace a single span bridge which was washed out in August, 1980. A relatively large number of boring were done at this site primarily to define the ledge surface which slopes downward from the south side of Caribou Stream to the north side. The soil overburden above the ledge was described as either gravel and rocks or brown silty till, and the density of it varies from loose to very dense. The fill behind the still standing southerly abutment of the old bridge consists of loose to medium brown silty sand and gravel.

The southerly end of this structure can be built directly on the ledge surface, but the northerly end will have to be connected to ledge through the use of short piles or columns or by excavating the soil between the footing and the ledge surface and replacing it with concrete fill. If the footing pressures could be kept low enough, the northerly abutment could be supported on a spread footing in the soil above ledge; however, this design, one end of the structure on ledge and the other on soil invites differential movement and resultant cracking of the structure.

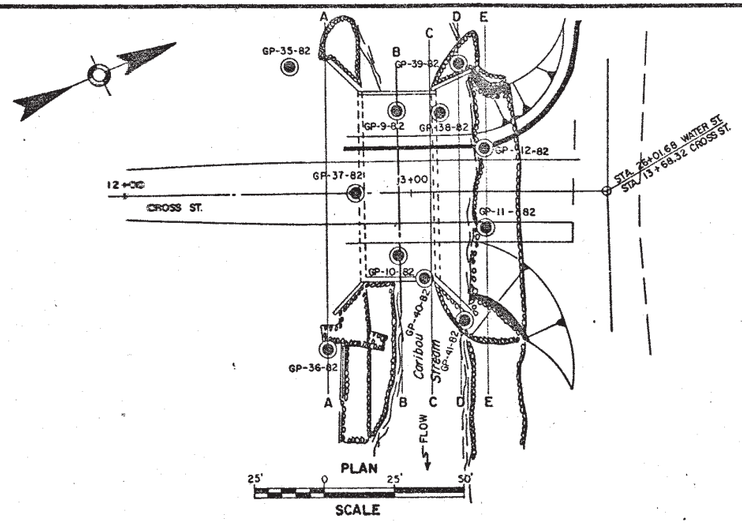
Prepared by Guy Baker

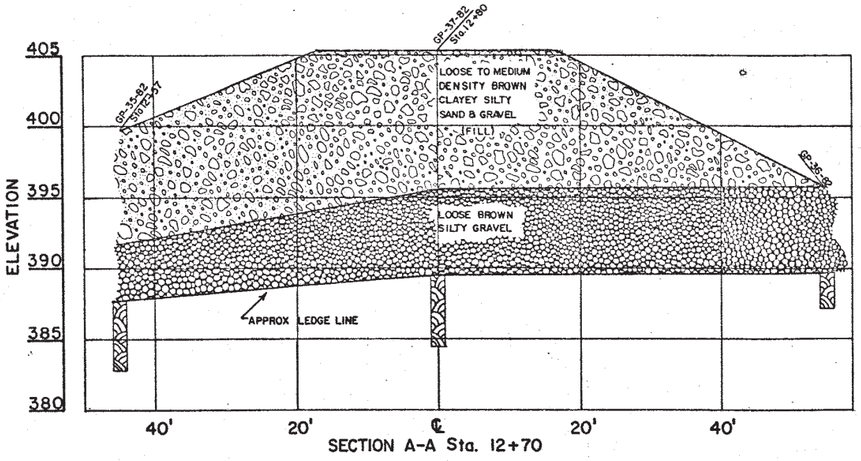
Guy Baker
Assistant Soils Engineer



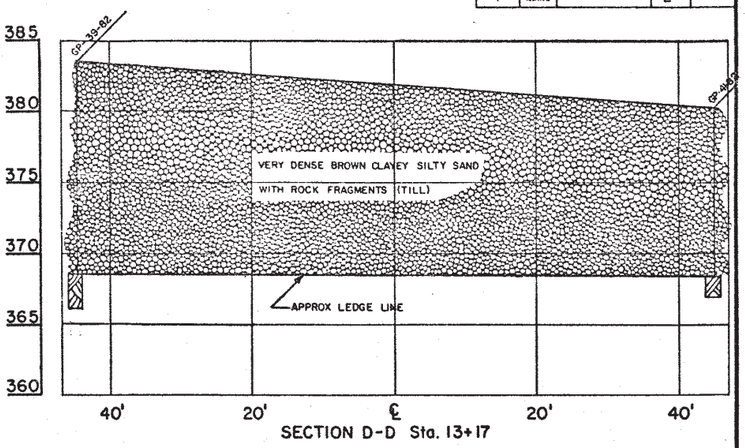
46 0782
10 X 10 TO THE INCH • 7 X 10 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A.

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE		2	

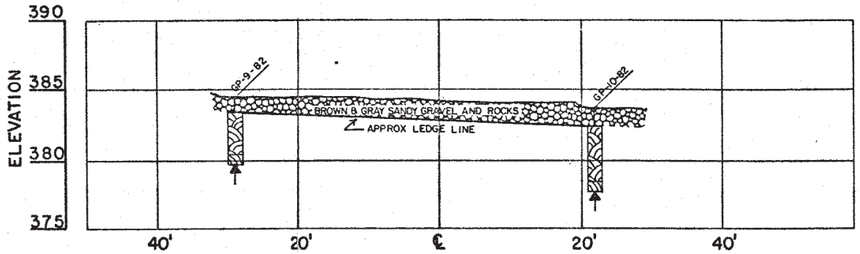




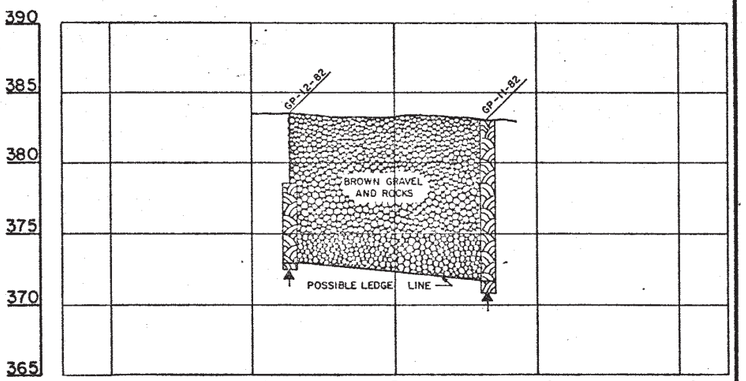
SECTION A-A Sta. 12+70



SECTION D-D Sta. 13+17



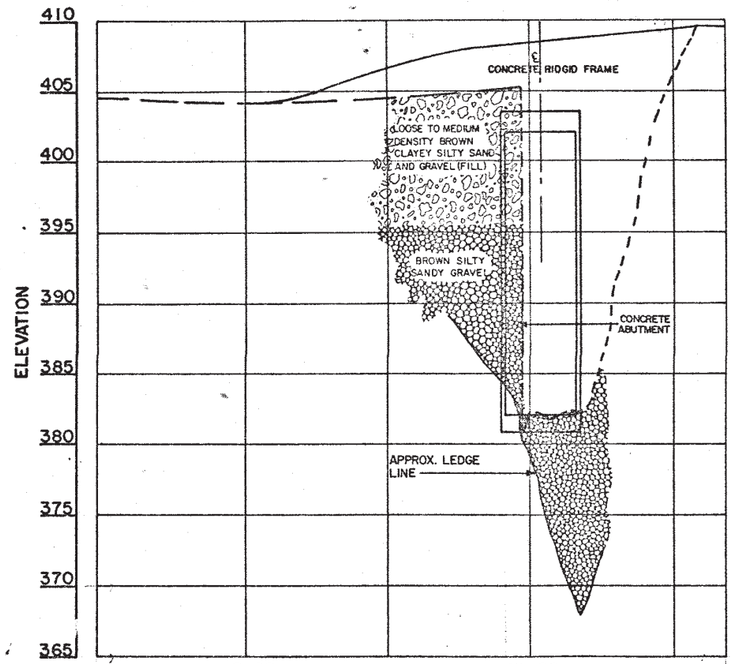
SECTION B-B Sta. 12+95



SECTION E-E Sta. 13+26

SECTION C-C Sta 13+07

TRANSVERSE SECTIONS
HORIZ. 10' 0 10' 20'
VERT. 5' 0 5' 10'
SCALE



PROFILE
HORIZ. 25' 0 25' 50'
VERT. 5' 0 5' 10'
SCALE

PROJECT DESIGN ENGINEER	BY	DATE
PLANS	DESIGN - DETAILED	
	CHECKED	
	REVISIONS	
	FIELD CHANGES	

BRUNING 44-132 45710

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

CROSS ST. BRIDGE
OVER
CARIBOU STREAM
IN THE TOWN OF
CARIBOU
AROOSTOOK COUNTY

FOUNDATION SURVEY

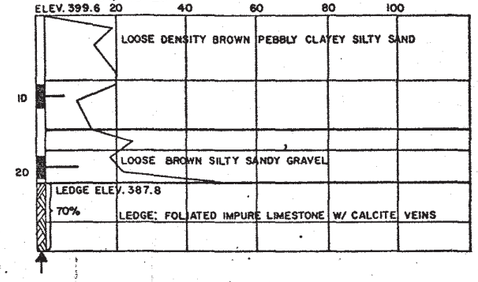
SHEET OF AUGUSTA, MAINE

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
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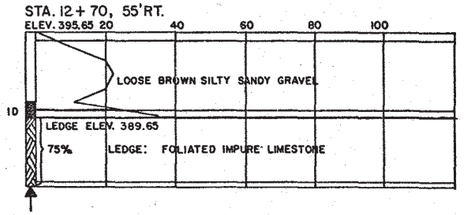
ELEVATION

405
400
395
390
385
380

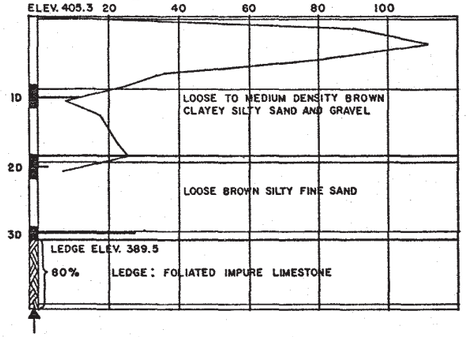
BORING GP-35-82
STA. 12+57, 45' LT.
ELEV. 399.6 20 40 60 80 100



BORING GP-36-82
STA. 12+70, 55'RT.
ELEV. 395.65 20 40 60 80 100

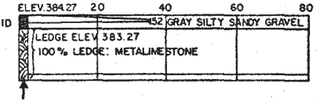


BORING GP-37-82
STA. 12+80 ℄
ELEV. 405.3 20 40 60 80 100

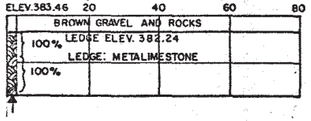


385
380
375
370

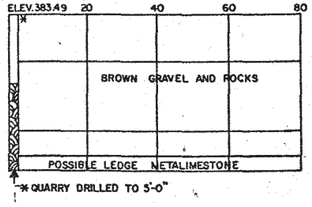
BORING GP-9-82
STA. 12+95, 29' LT.
ELEV. 384.27 20 40 60 80



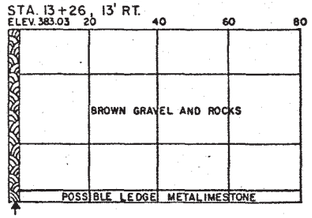
BORING GP-10-82
STA. 12+95, 22' RT.
ELEV. 383.46 20 40 60 80



BORING GP-12-82
STA. 13+26, 15 LT.
ELEV. 383.49 20 40 60 80

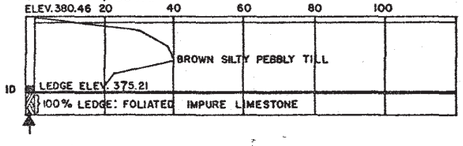


BORING GP-11-82
STA. 13+26, 13' RT.
ELEV. 383.03 20 40 60 80

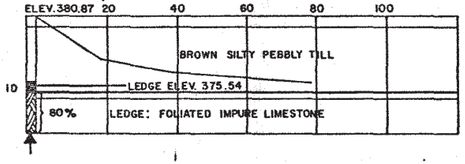


385
380
375
370
365

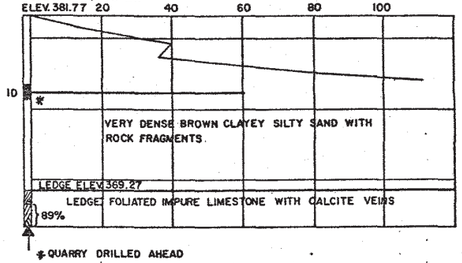
BORING GP-40-82
STA. 13+04, 30 RT.
ELEV. 380.46 20 40 60 80 100



BORING GP-38-82
STA. 13+10, 28'LT.
ELEV. 380.87 20 40 60 80 100

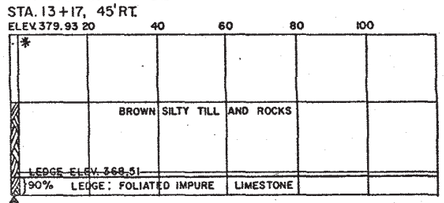


BORING GP-39-82
STA. 13+17, 45'LT.
ELEV. 381.77 20 40 60 80 100



*QUARRY DRILLED AHEAD

BORING GP-41-82
STA. 13+17, 45'RT.
ELEV. 379.93 20 40 60 80 100



*QUARRY DRILLED WITHOUT CASING

*BORING NOTES

All samples and vanes are made ahead of casing

N — Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow

Location of sample or sample attempt

ID — Number and type of dry sample
S & H Sampler #1290's

Number of blows required to drive spoon or tubing one foot with 350 ft lbs. of energy per blow

Bottom of boring (may not be bottom of soil strata)

71% Location cored by diamond bit and percent recovery of rock.

PROJECT DESIGN ENGINEER
PLANS
DESIGN - DETAILED
CHECKED
REVISIONS
FIELD CHANGES
BY
DATE

BRUNING 44-132-6710

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

CROSS ST. BRIDGE
OVER
CARIBOU STREAM
IN THE TOWN OF
CARIBOU
AROOSTOOK COUNTY

BORING DETAILS

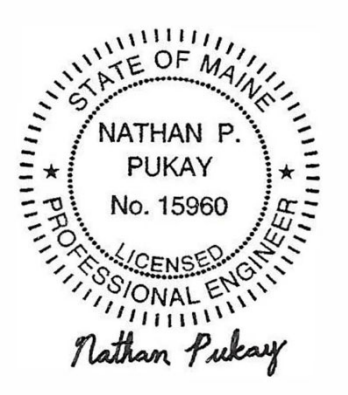
SHEET OF AUGUSTA, MAINE

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DATA REPORT

For the Rehabilitation of:

THE FORKS BRIDGE
ROUTE 201 OVER KENNEBEC RIVER
THE FORKS PLANTATION – WEST FORKS PLANTATION, MAINE



Prepared by:

Nathan Pukay, P.E.
Transportation Engineer II

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Somerset County
WIN 22362.00

Soils Report 2023-10
Bridge No. 2841

Federal Project No. 2236200
May 26, 2023

Table of Contents

1.0 INTRODUCTION 1
2.0 GEOLOGIC SETTING 1
3.0 SUBSURFACE INVESTIGATION 1
4.0 SOIL PROPERTIES 2
5.0 CLOSURE 2

Sheets

Sheet 1 – Location Map

Appendices

Appendix A – Existing Subsurface Information
Appendix B – Maine Department of Transportation Standard Specifications, Revision of July 1988, Selected Pages.

1.0 INTRODUCTION

The purpose of this Geotechnical Data Report is to present available geotechnical information obtained by the Maine Department of Transportation (MaineDOT) at the Forks Bridge site in The Forks Plantation-West Forks Plantation, Maine.

The existing Forks Bridge carries Route 201 over the Kennebec River and was constructed in 1990. The structure is a 165-foot span, steel rigid frame bridge supported on spread footings bearing on soil, with independent cantilever-type concrete wingwalls also bearing on soil. According to the 2019 MaineDOT Bridge Inspection Report, the superstructure is rated an 8 (very good condition), the bridge deck is rated a 6 (satisfactory condition), and the substructure is rated a 5 (fair condition). Three out of four of the wingwalls show signs of severe cracking due to alkali-silica reaction (ASR). The fourth wingwall is in overall good condition as it was rehabilitated by MaineDOT Maintenance approximately ten years ago. The scope of the project is to reinforce the three damaged wingwalls with soil nail walls, perform concrete repairs on the curb and breastwalls, and rehabilitate the bridge drains with drain liners.

2.0 GEOLOGIC SETTING

The Forks Bridge carries Route 201 over Kennebec River as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Reconnaissance Surficial Geology Map of The Forks Quadrangle, Open-File No. 76-26 (1976), indicates the surficial soils in the vicinity of the bridge project consist of glacial-stream deposits and glacial till. Glacial-stream deposits consist of sand and gravel. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones.

The MGS Bedrock Geology of Maine (1985) maps the bedrock in the vicinity of the project as argillaceous limestone and/or dolostone of The Forks Formation.

3.0 SUBSURFACE INVESTIGATION

The available subsurface information can be found in MaineDOT Soils Report 86-24, dated May 1986. This report is provided in Appendix A – Existing Subsurface Information. Additionally, the historical bridge plans and as-builts provide information related to structural backfill and embankment fill limits. Information related to the gradations of these soils is provided in Appendix B – Maine Department of Transportation Standard Specifications, Revision of July 1988, (Select Pages).

4.0 SOIL PROPERTIES

The following table provides recommended soil properties. These soil properties are provided for informational purposes. Soil properties for the soil nail wall design shall be determined by the soil nail wall designer based on the available subsurface information and previous design experience.

Soil Unit	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Gravel Borrow	125	34	0
Common Borrow	120	30	0

5.0 CLOSURE

This Geotechnical Data Report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed bridge rehabilitation project at the The Forks Bridge in The Forks Plantation-West Forks Plantation, Maine. The report was prepared in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

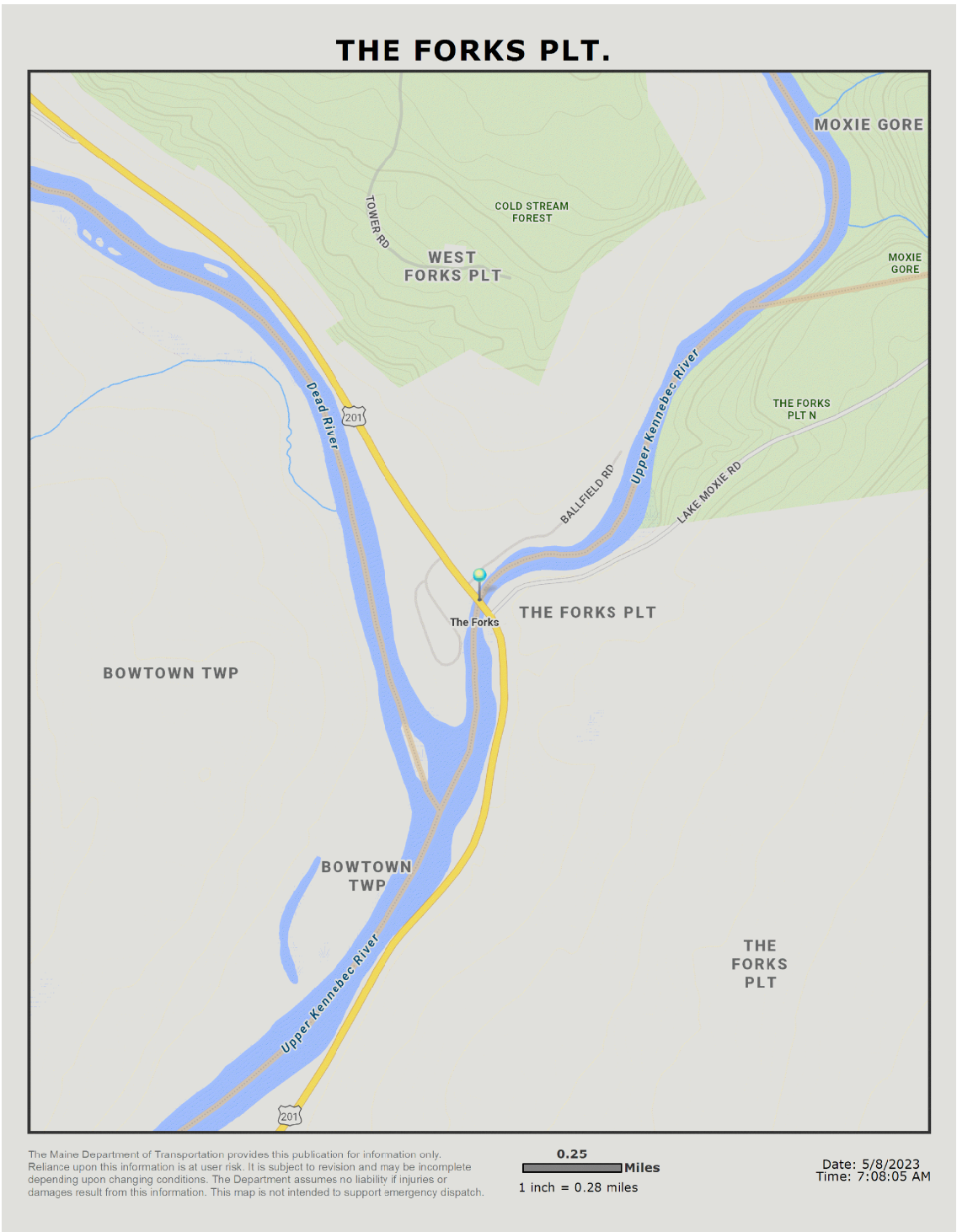
MaineDOT shall not be responsible for the bidder's or contractor's interpretations of, estimates or conclusions derived from the geotechnical information. Data provided may not be representative of the subsurface conditions between boring locations.

Sheets

Date:5/10/2023

Division: GEOTECH

Username: Nathan.P.Pukay



SHEET NUMBER 1 OF 1	THE FORKS BRIDGE KENNEBEC RIVER THE FORKS PLT. SOMERSET COUNTY LOCATION MAP	STATE OF MAINE DEPARTMENT OF TRANSPORTATION 2236200 WIN BRIDGE NO. 2841 22362.00 BRIDGE PLANS

Appendix A

Existing Subsurface Information

THE FORKS 1862.00

Soils Report 86-24
The Forks - Somerset County
1862.00 - Fed. No. 0332008
The Forks Bridge
May, 1986

86.24

Maine Department of Transportation

Technical Services Division

Geotechnical Section

SUBSURFACE SOILS INVESTIGATION FOR THE PROPOSED REPLACEMENT OF THE FORKS BRIDGE OVER THE KENNEBEC RIVER IN THE TOWNS OF THE FORKS AND WEST FORKS, SOMERSET COUNTY

Somerset County

PIN 1862.00
Fed. No. 0332008
May, 1986

Soils Report 86-24

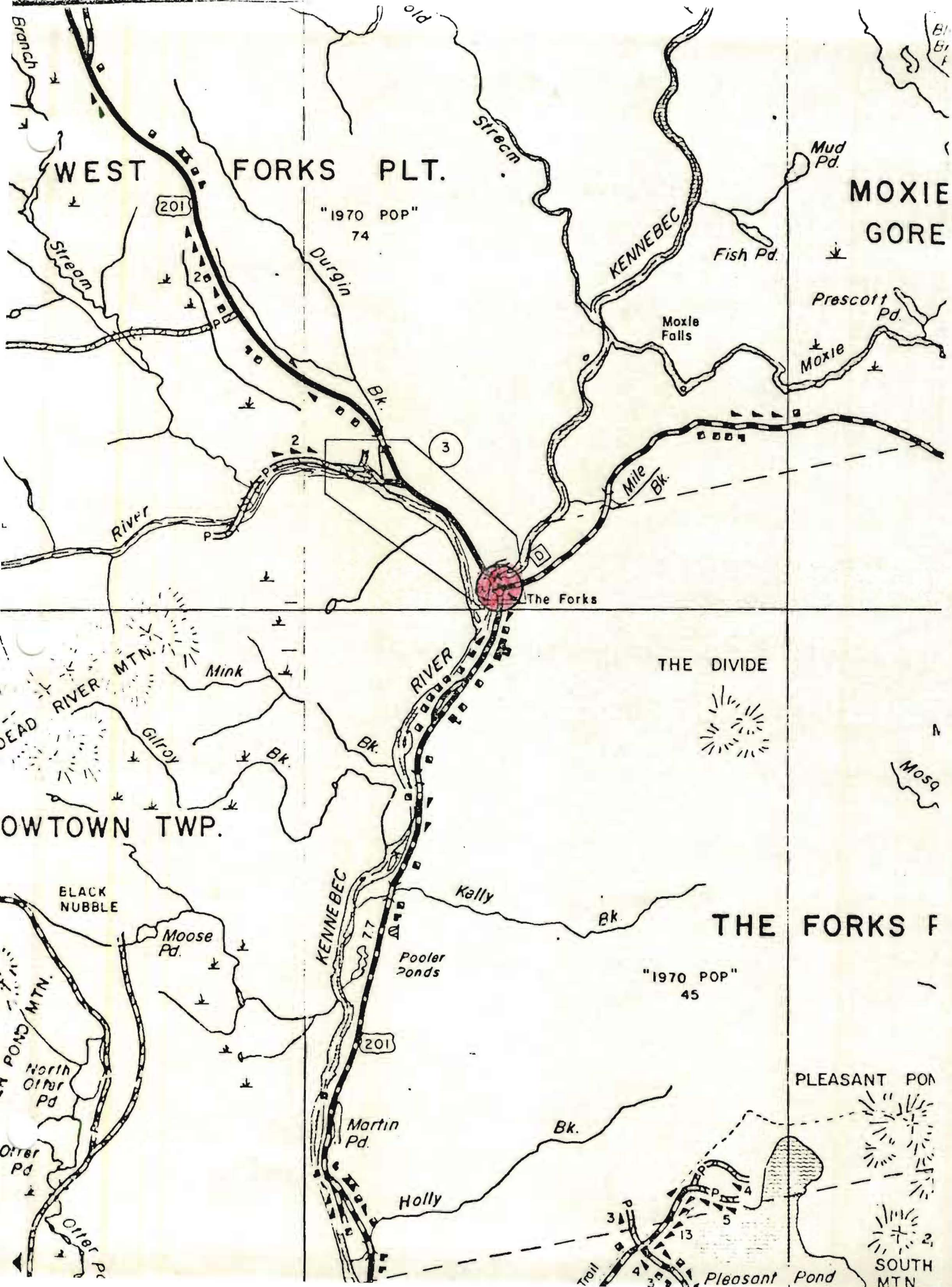

WEST FORKS PLT.
"1970 POP"
74
201
Stream
Durgin
Old
Stream
KENNEBEC
Mud Pd.
Fish Pd.
MOXIE GORE
Prescott Pd.
Moxie Falls
Moxie
Bk.
Mile Bk.
River
The Forks
THE DIVIDE
RIVER
Mink
Gilroy
Bk.
Bk.
MTN.
OWTOWN TWP.
BLACK NUBBLE
Moose Pd.
KENNEBEC
Kally Bk
Pooler Ponds
THE FORKS P
"1970 POP"
45
201
North Otter Pd
Otter Pd
Martin Pd.
Bk.
Holly
PLEASANT POND
SOUTH MTN.
Pleasant Pond

TABLE OF CONTENTS

Text	Page No.
INTRODUCTION.	1
GENERAL CONDITIONS.	1
DETAILED CONDITIONS	
Abutment No. 1.	2
Abutment No. 2.	2
Lateral Stresses.	3
SUMMARY	4

Illustration	Sheet No.
Bearing Capacity Chart.	1
Foundation Survey	2

INTRODUCTION

A subsurface soils investigation has been completed for The Forks Bridge over the Kennebec River in the Towns of The Forks and West Forks. This proposed bridge will replace an existing narrow through-truss structure.

Soils investigation consisted of five washborings. Two washborings were made during March and April, 1985 by a crew under the supervision of Mr. Gary Paine. The three remaining washborings were made by Maine Test Borings, Inc. during April and May, 1986. The locations of the five washborings are shown on a plan of the proposed bridge on Sheet 2 of the illustrations following the text of this report. Also shown on this sheet is a centerline profile of the proposed bridge depicting the soils stratification and the details of the five washborings. The original for Sheet 2 is being provided to the Bridge Design Section for inclusion in the construction plans.

Soil samples and rock cores were transported to the Central Laboratory in Bangor where standard identification tests were performed.

GENERAL CONDITIONS

The proposed project is located in the Towns of The Forks and West Forks, Somerset County. The existing bridge which carries Route 201 over the Kennebec River is a single span through-truss structure with a length between abutments of approximately 170 feet. The proposed bridge will be a single span rigid frame structure with a total length between abutments of 165 feet. The new structure will be located approximately 65 feet downstream of the existing bridge.

Soils at the bridge site generally consist of dense brown sandy gravel with many rocks. Washboring B-3 made at Station 118+16, 33 feet

right on the south shore of the Kennebec River encountered solid ledge at a depth of 24.1 feet. Ledge was not encountered by the borings made on the north shore of the river. Local well information indicates that ledge may be found at a depth of 100 feet or greater on the north shore of the river.

DETAILED CONDITIONS

Abutment No. 1:

The centerline of bearing for this abutment intersects the proposed centerline of the new bridge at Station 118+60 and is skewed counterclockwise 20°. Three washborings were made within the approach to this abutment location. Due to high water, no borings could be made at the exact location of the proposed abutment. All three washborings made near proposed Abutment No. 1 encountered dense brown sandy gravel with many rocks. Only one boring, B-3, was able to penetrate to what is believed to be the ledge surface. The ledge, described as calcareous metasiltstone (impure meta limestone) with a high angle of foliation, was encountered at a depth of 24.1 feet at Elevation 550.5. It is believed that piles would have difficulty penetrating to ledge. It is recommended to support the proposed Abutment No. 1 on spread footings with a footing elevation below the lowest elevation of the river bed. A bearing capacity chart for this footing is shown on Sheet 1. Adequate protection against scour should be provided for this footing.

Abutment No. 2:

The centerline of bearing for this abutment intersects the proposed centerline of the new bridge at Station 120+25 and is skewed as Abutment No. 1. The washborings were made as near to the ends of the proposed abutment as possible. Both borings also encountered the dense brown sandy gravel with many rocks. Washboring B-1, made at Station 120+42, 25 feet

left penetrated to a depth of 52.2 feet before being stopped by rocks. Due to the large number of rocks encountered, this boring was advanced by core drilling and spinning the casing. Apparently the subsurface material is similar to that exposed on the north shore of the river, i.e. gravel with numerous rocks 1 to 2 feet in diameter. Information from local residents indicates that the ledge surface may be at a depth of 100 feet or more at this location. As with Abutment No. 1, it is recommended to support the proposed Abutment No. 2 on a spread footing located below the lowest elevation of the river bed. The bearing capacity chart for this footing is shown on Sheet 1. Adequate protection against scour should be provided.

Lateral Stresses:

The unusual design of this bridge will result in 2± inches of movement at each abutment due to thermal expansion and contraction. This movement raises questions regarding the lateral stresses developed within the approach embankments. Movement of the abutment walls toward the fill should result in lateral earth pressures greater than the active case and possibly approaching the passive case while movement of the walls away from the fill may result in reduced lateral pressures approaching the active case. In reality the pressure distribution against the abutment walls would vary with depth as well as abutment movement. It is believed unreasonable to design the abutments based on full passive pressures. It is suggested that a lateral earth pressure coefficient (K) of 5.5 be used for design. This value corresponds to an equivalent fluid weight of 690 pcf. Another consideration is settlement/heave behind the abutment. It is believed that this effect will not be a problem especially if approach slabs are utilized.

At the time of this writing, it has been proposed to incorporate

a retaining wall behind the integral abutments. This would isolate the bridge movement from the approach fills. If the retaining wall is prevented from any movement away from the fill, the higher at-rest lateral earth pressures should probably be used in its design. (K=0.40 or 50 pcf equivalent fluid weight.)

Due to the unusual design of the structure it may be desirable to monitor the performance of the completed bridge. It is suggested that bridge movement, approach heave/settlement and lateral earth pressures be instrumented and monitored.

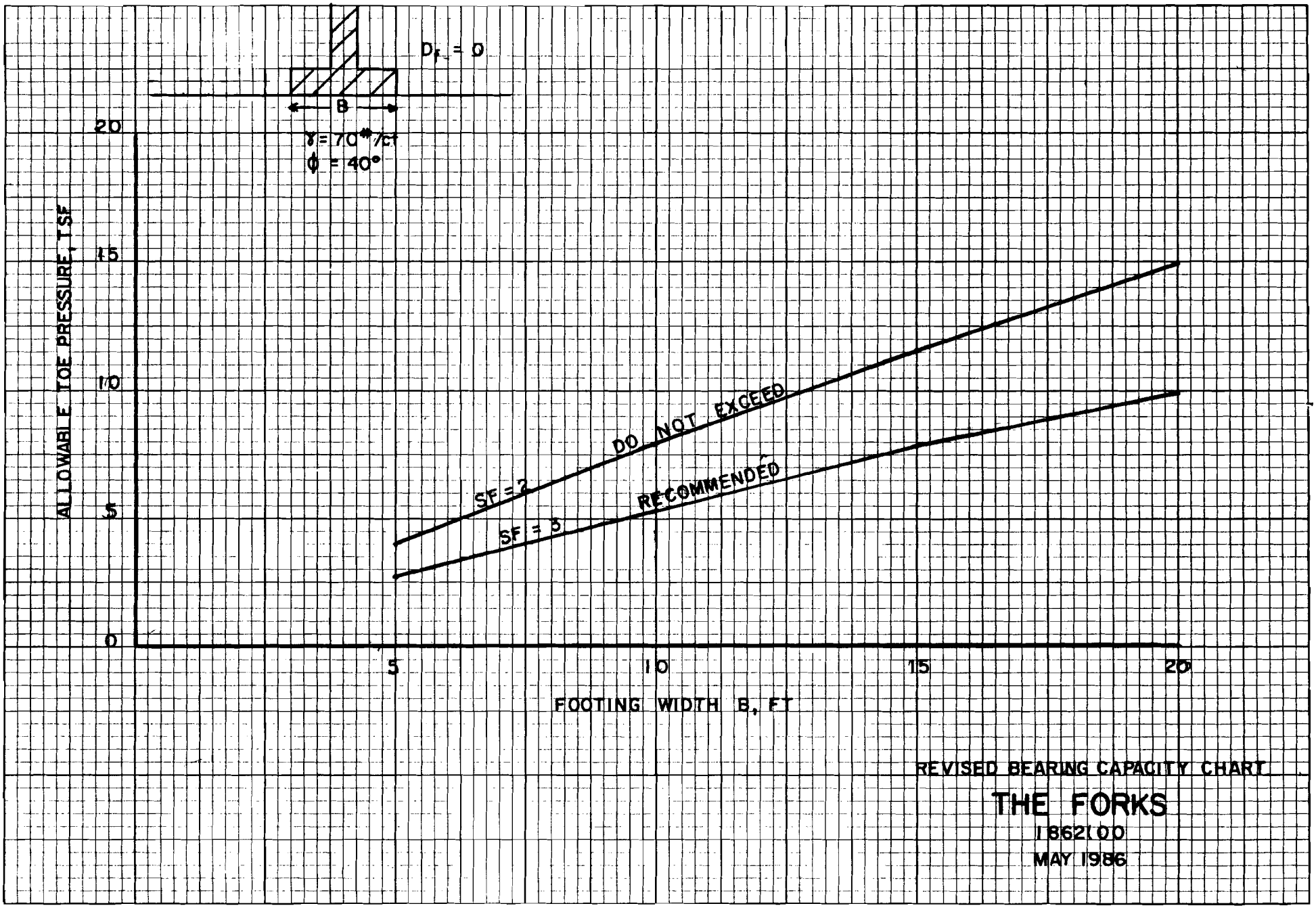
SUMMARY

A total of five washborings, three made in the vicinity of Abutment No. 1 and two near the proposed abutment No. 2 were made for the proposed project. The locations of these explorations are shown on a plan on Sheet 2. Also shown on this sheet are a centerline profile depicting the soils stratification at the proposed bridge site and the details of the five washborings. Soils at the bridge site generally consist of dense brown sandy gravel with many rocks. Apparent ledge is located at a depth of 24.1 feet in the vicinity of Abutment No. 1 and slopes steeply downward to the north. It is recommended to support the abutments on spread footings. The footing elevations should be located below the lowest elevation of the river bed and adequate protection against scour must be provided. A discussion dealing with lateral earth pressures and the unusual design of the proposed bridge is given in the Detailed Conditions section.

Prepared by: _____
David B. Miller
Associate Geotechnical Engineer

Approved by: _____
Gerald A. Waldo
Geotechnical Engineer

K&E 10 X 10 TO THE INCH • 7 X 10 INCHES
KEUFFEL & ESSER CO MADE IN U.S.A.



REVISED BEARING CAPACITY CHART

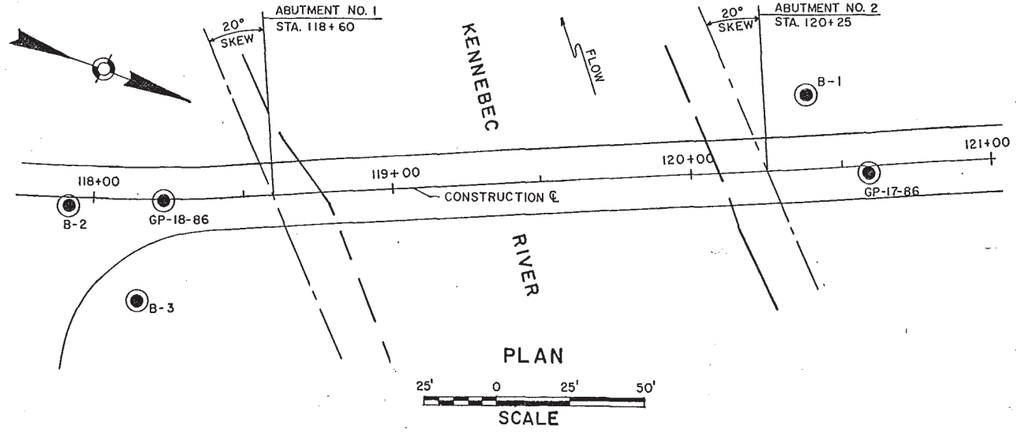
THE FORKS

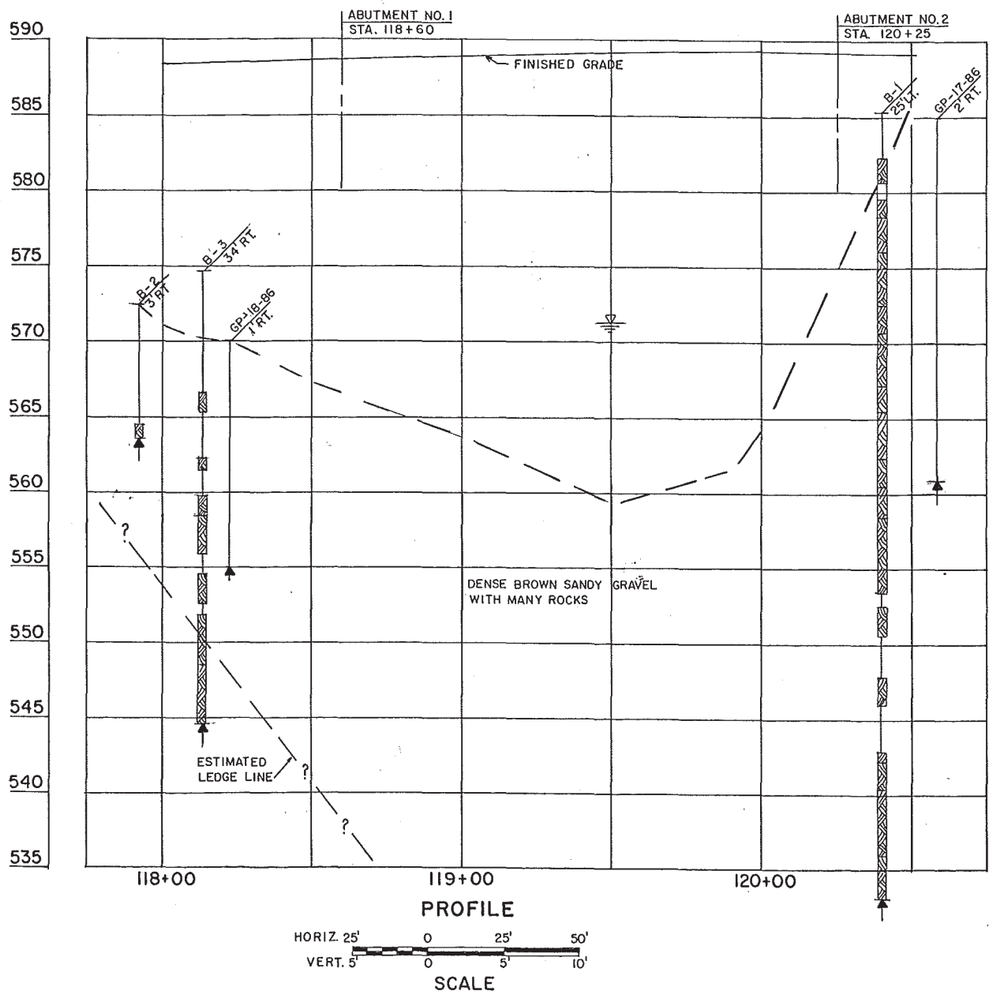
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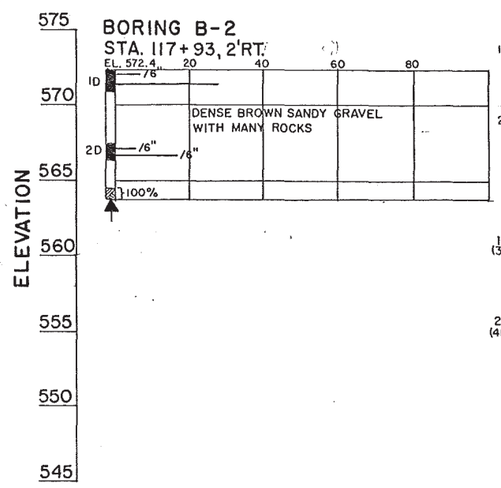
MAY 1986

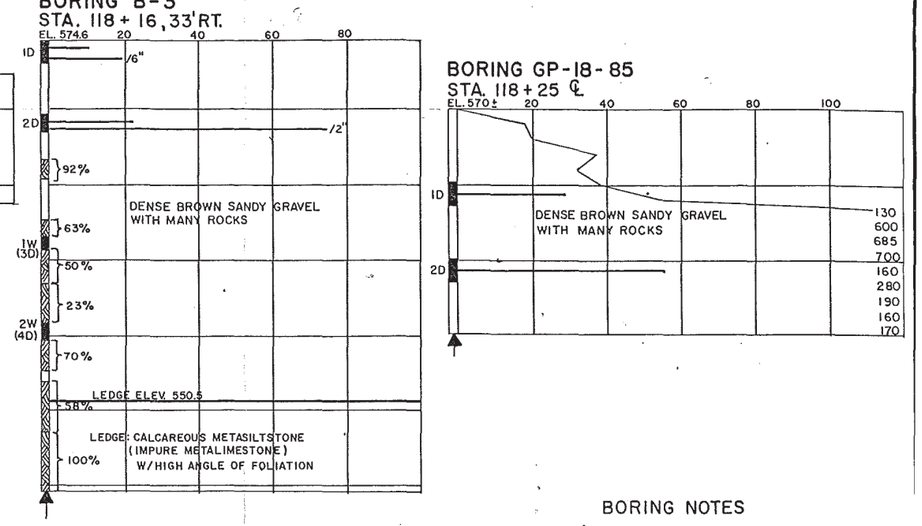
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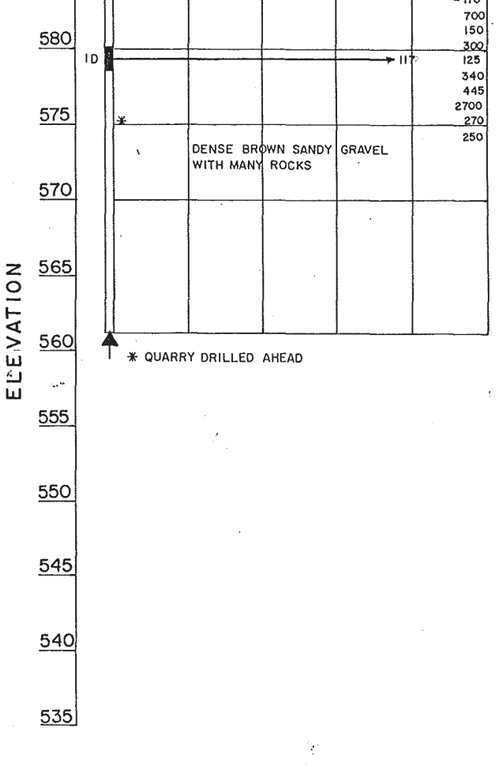
F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE		2	

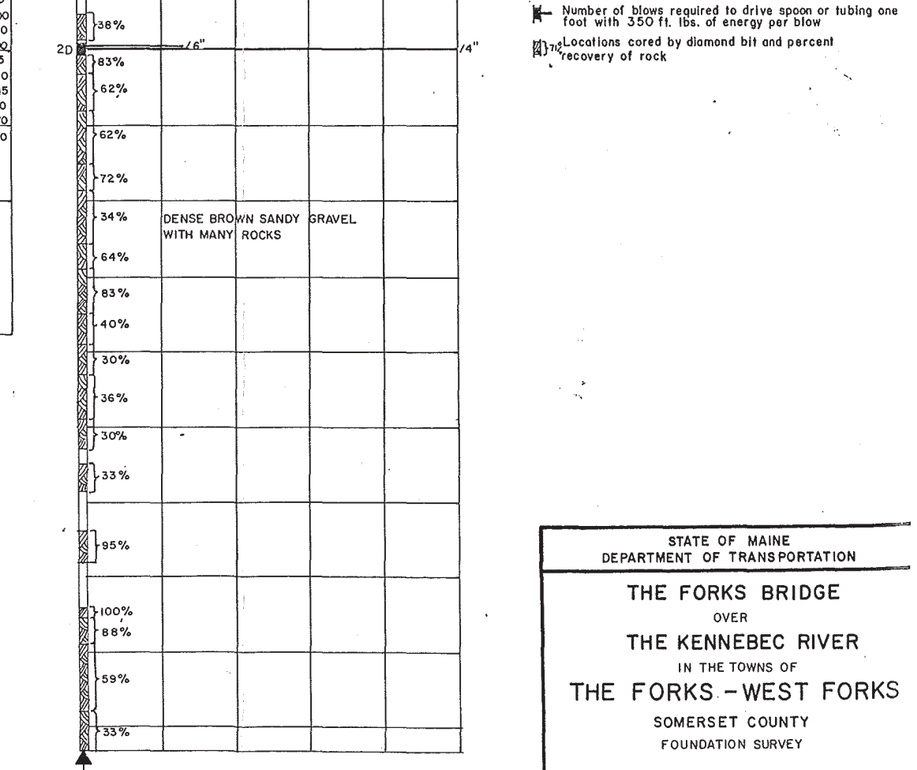












BORING NOTES

- All samples and vones are made ahead of casing
- N — Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow
- Location and type of dry sample
- 1D S & H Sampler # 1290's
- 1W Wash sample and number
- N — Number of blows required to drive spoon or tubing one foot with 350 ft. lbs. of energy per blow
- 71% Locations cored by diamond bit and percent recovery of rock

PLANS CHECKED REVISIONS FIELD CHANGES

BRUNING 44-132-45710-1

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

THE FORKS BRIDGE
OVER
THE KENNEBEC RIVER
IN THE TOWNS OF
THE FORKS - WEST FORKS
SOMERSET COUNTY
FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE

Appendix B

Maine Department of Transportation Standard Specifications,
Revision of July 1988, Selected Pages.

STATE OF MAINE



Department of Transportation

Standard Specifications

Highways and Bridges

Revision of July, 1988

Power Company,
Before me,

When tested in accordance with AASHTO T182, (ASTM D1664) the stone chip aggregate shall have a retained bituminous film above 95 percent. The stripping tests may be omitted if the total quantity of stone chip aggregate is less than 50 cubic yards or if it comes from a source which has given satisfactory performance.

703.14 Blotter. Aggregate for blotter material shall consist of sharp durable particles of sand conforming to the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
3/4 inch	100
No. 4	40-100
No. 40	10-50
No. 200	0-5.0

The aggregate shall be free from vegetable and other deleterious material.

703.15 Filler. These materials shall conform to the following specification requirements for the designated materials.

Mineral filler shall conform to the requirements of AASHTO M17.

Ground limestone shall meet the following gradation requirements:

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
No. 8	100
No. 50	80-100
No. 200	30.0-70.0

703.16 Vacant

703.17 Aggregate for Stabilized Base. Aggregates for bituminous stabilized base course shall consist of granular material containing hard, durable particles conforming to the gradation requirements of the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
1 1/2 inch	100
3/8 inch	60-100
No. 4	50-85
No. 8	38-74
No. 50	8-34
No. 200	0-10.0

The gradation shall be uniform within the limits specified and shall not vary from the low limit on one sieve size to the high limit on the adjacent sieve size. The material passing the No. 200 sieve shall be not more than 1/2 of the fraction passing the No. 50 sieve.

703.18 Common Borrow. Common borrow shall consist of earth, suitable for embankment construction. It shall be free from frozen material, perishable rubbish, peat and other unsuitable material.

The moisture content shall be sufficient to provide the required compaction and stable embankment. In no case shall the moisture content exceed 4 percent above optimum.

The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D.

703.19 Granular Borrow. Granular borrow shall consist of sand or gravel of hard durable particles free from vegetable matter, lumps or balls of clay and other deleterious substances. The gradation of that portion passing a 3 inch sieve shall meet the gradation requirements of the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves: Material for Underwater Backfill	Material for Embankment Construction
No. 40	0-70	0-70
No. 200	0-10.0	0-20.0

Granular borrow shall contain no particles or fragments with a maximum dimension in excess of the compacted thickness of the layer being placed.

703.20 Gravel Borrow. Gravel borrow shall consist of uniformly graded granular material having no rocks with a maximum dimension of over 6 inches and that portion passing a 3 inch square mesh sieve shall contain not more than 70 percent passing a 1/4 inch mesh sieve and not more than 10 percent passing a No. 200 mesh sieve.

703.21 Rock Borrow. Rock borrow shall consist of hard durable rock broken to various sizes that will form a compact embankment with a minimum of voids. The maximum size for any rock shall be 3 feet in its greatest dimension.

703.22 Underdrain Backfill Material. Granular material for Underdrain Type B shall be free from organic matter and shall conform to the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
1 inch	95-100
1/2 inch	75-100
No. 4	50-100
No. 20	15-80
No. 50	0-15
No. 200	0-5.0

Crushed or uncrushed material for Underdrain Type C shall conform to the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
1 inch	100
3/4 inch	90-100
3/8 inch	0-75
No. 4	0-25
No. 10	0-5.0

703.23 Vacant

703.24 Stone for French Drains. Stones for French drains shall consist of hard, durable rock and shall conform to the following table.

Sieve Designation	Percentage by Weight Passing Square Mesh Sieves
6 inch	90-100
1 1/2 inch	0-40
No. 4	0-5

Gradation test shall conform to AASHTO T27 except that the total sample shall be sieved and the minimum weight of the sample will be 120 pounds.

703.25 Stone Fill. Stones for stone fill shall consist of sound durable rock which will not disintegrate by exposure to water or weather. Either field stone or rough, unhewn quarry stone may be used. Stones shall weigh from 10 pounds to a maximum of 500 pounds or larger if approved by the Engineer. Fifty percent by weight of the stones, shall be approximately 200 pounds.

703.26 Plain and Hand Laid Riprap. Stones shall consist of sound durable rock which will not disintegrate by exposure to water or weather. Either field stone or rough, unhewn quarry stone may be used. Exposed stones shall be angular and as nearly rectangular in cross-section as practicable. Rounded boulders or cobbles will not be permitted. Stones shall weigh from 10 pounds to 200 pounds except that when available suitable stones weighing more than 200 pounds may be used. Approximately 50 percent of the stones by volume, shall exceed a unit weight of 50 pounds.

703.27 Stone Blanket. Stones shall consist of sound durable rock which will not disintegrate by exposure to water or wind. Either field stone or rough, unhewn quarry stone may be used. Stones shall weigh from 300 pounds to 3,000 pounds. Approximately 50 percent of the stones, by volume, shall exceed a unit weight of 1,000 pounds.

703.28 Heavy Riprap. Stones shall consist of sound, durable rock, resistant to the action of air and water. Either field stone or rough, unhewn quarry stone may be used. The exposed stones shall be angular. Round or thin, flat stones will not be permitted. Stones shall have a minimum weight of 500 pounds each and at least 50 percent of the stones, by volume, shall exceed 1,000 pounds each.

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7th Edition.



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16. Abstract

This document presents information on the analysis, design, and construction of permanent 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, construct, and inspect safe and economical structures. This document updates the information contained in FHWA0-IF-03-017 (Lazarte et al. 2003). The focus is on soil nailing techniques that are commonly used in U.S. practice. The contents of this document include: an introduction; chapters on applications and feasibility, construction materials and methods, information required for design, analysis and design of soil nail walls, corrosion protection; and chapters on contracting approach, technical specifications and design examples. This manual introduces a framework for the design of soil nail walls that takes into account factors of safety used in the ASD method while integrating LRFD principles.

17. Key Word	18. Distribution Statement
Soil nail, soil nail wall design, LRFD, ASD, earth-retaining structures, construction, soil nail load test, facing, shotcrete, specifications.	No restrictions.

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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When You Know	**Multiply By**	**To Find**	**When You Know**	**Multiply By**	**To Find**
(a) Length					
inch (in.)	25.4	millimeter (mm)	millimeter (mm)	0.039	inch (in.)
foot (ft)	0.305	meter (m)	meter (m)	3.28	foot (ft)
yard (yd)	0.914	meter (m)	meter (m)	1.09	yard (yd)
mile (mi)	1.61	kilometer (km)	kilometer (km)	0.621	mile (mi)
(b) Area					
square inches (in^2)	645.2	square millimeters (mm^2)	square millimeters (mm^2)	0.0016	square inches (in^2)
square feet (ft^2)	0.093	square meters (m^2)	square meters (m^2)	10.764	square feet (ft^2)
Acres (ac)	0.405	hectares (ha)	hectares (ha)	2.47	Acres (ac)
square miles (mi^2)	2.59	square kilometers (km^2)	square kilometers (km^2)	0.386	square miles (mi^2)
square inches (in^2)	645.2	square millimeters (mm^2)	square millimeters (mm^2)	0.0016	square inches (in^2)
(c) Volume					
fluid ounces (oz)	29.57	milliliters (mL)	milliliters (mL)	0.034	fluid ounces (oz)
Gallons (gal)	3.785	liters (L)	liters (L)	0.264	Gallons (gal)
cubic feet (ft^3)	0.028	cubic meters (m^3)	cubic meters (m^3)	35.32	cubic feet (ft^3)
cubic yards (yd^3)	0.765	cubic meters (m^3)	cubic meters (m^3)	1.308	cubic yards (yd^3)
(d) Mass					
ounces (oz)	28.35	grams (g)	grams (g)	0.035	ounces
pounds (lb)	0.454	kilograms (kg)	kilograms (kg)	2.205	pounds
short tons (2000 lb) (T)	0.907	megagrams (tonne) (Mg)	megagrams (tonne) (Mg)	1.102	short tons (2000 lb)
(e) Force					
pound (lb)	4.448	Newton (N)	Newton (N)	0.2248	pound (lb)
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot (psf)	47.88	Pascals (Pa)	Pascals (Pa)	0.021	pounds per square foot (psf)
pounds per square inch (psi)	6.895	kiloPascals (kPa)	kiloPascals (kPa)	0.145	pounds per square inch (psi)
(g) Density					
pounds per cubic foot (pcf)	16.019	kilograms per cubic meter (kgm^3)	kilograms per cubic meter (kgm^3)	0.0624	pounds per cubic feet (pcf)
(h) Temperature					
Fahrenheit temperature (^oF)	5/9(^oF- 32)	Celsius temperature (^oC)	Celsius temperature (^oC)	9/5(^oC)+ 32	Fahrenheit temperature (^oF)

Notes:

1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second (s), Newton (N), and Pascal ($Pa=N/m^2$).

2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.

3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

PREFACE

The purpose of this manual is to provide updated, state-of-the-practice information for selecting, designing, and constructing soil nail walls for roadway projects. The information contained herein is aimed at producing safe and cost-effective designs, and to help owners identify and manage the risks associated with soil nail wall projects. This manual focuses solely on soil nail systems for long-term support of excavations and does not specifically address the use of soil nails as temporary structures. This GEC serves as the FHWA reference document for highway projects involving permanent soil nail walls.

Recent advances in the state-of-the-practice for this system addressed in the document include:

- The implementation of the Load and Resistance Factor Design (LRFD) platform.
- Information on new or emerging soil nailing technologies.
- The inclusion of design examples based upon the SNAP-2 computer program.

The primary audience for this document is: agency and consulting engineers specialized in bridge, structural, geotechnical, and roadway design; and engineering geologists and consulting engineers providing technical reviews, or who are engaged in the design, procurement, and construction of permanent soil nail systems. This document is also intended for management, specification and contracting specialists, as well as for construction engineers interested in design and contracting aspects of soil nail systems. In this document, the term "soil nail" refers to inclusions in soils and soft and weathered rock. The term "soil nail" is used in this document regardless of the material supported by the soil nail wall. Although this manual is focused on solid bar soil nails, Chapter 10 presents necessary considerations for the design and construction of Hollow Bar Soil Nails (HBSNs).

This document draws descriptions and basic information from earlier FHWA publications in this field; in particular, the predecessor manual entitled "Soil Nail Walls," Report FHWA0-IF-03-017 (Lazarte et al. 2003). Valuable information was also obtained from the publication entitled "Hollow Bar Soil Nails Pullout Test Program," Publication No. FHWA-CFL/TD-10-001 (Cadden et al. 2003).

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Table of Contents

Cover
Notice Page
Technical Report Documentation Page
Conversion Factors
Preface
Acknowledgements
Table of Contents i
List of Figures xiii
List of Tables xviii
List of Equations xxi
List of Symbols xxv
List of Acronyms xxxiii

Chapter 1: Introduction 1

1.1 Overview 1
1.2 Purpose 1
1.3 Definition of Soil Nails 1
1.4 Background 3
1.4.1 Historical Origins of Soil Nail Walls 3
1.4.2 Developments in the United States 3
1.4.3 Leading Historic Documents in the United States 4
1.5 Design Philosophy 5
1.6 Document Organization 5

Chapter 2: Applications and Feasibility Evaluations 9

2.1 Introduction 9
2.2 Elements of a Soil Nail Wall 9
2.2.1 Soil Nails 9
2.2.2 Facing 10

2.2.3 Other Components 11
2.3 Construction Sequence 13
2.4 Applications of Soil Nail Walls 16
2.4.1 Overview 16
2.4.2 Roadway Cuts 16
2.4.3 Road Widening Under Existing Bridge Abutments 17
2.4.4 Tunnel Portals 18
2.4.5 Repair and Reconstruction of Existing Retaining Structures 19
2.4.6 Hybrid Soil Nail Walls 19
2.4.7 Shored Mechanically Stabilized Earth (SMSE) Walls 20
2.5 Advantages and Limitations of Soil Nailing 21
2.5.1 Advantages 21
2.5.1a Construction 21
2.5.1b Performance 22
2.5.1c Cost 22
2.5.2 Limitations 23
2.6 Evaluation of Soil Nail Walls 23
2.6.1 Overview 23
2.6.2 Main Risks and Risk Management 24
2.6.3 Aesthetics 25
2.7 Favorable/Unfavorable Soil Conditions for Soil Nailing 25
2.7.1 Overview 25
2.7.2 Favorable Ground Conditions for Soil Nailing 26
2.7.3 Difficult Soil Conditions for Soil Nailing 28
2.7.4 Unfavorable Soil Conditions for Soil Nailing 28
2.8 Factors Affecting Costs and Construction Schedule 31

Chapter 3: Construction Materials and Methods 33

3.1 Introduction 33

3.2 Components of a Soil Nail Wall 33

3.2.1 Tendons 34

3.2.2 Connection Components 35

3.2.3 Grout 37

3.2.4 Centralizers 41

3.2.5 Corrosion Protection Elements 42

3.2.6 Wall Facing 43

3.2.7 Drainage 45

3.3 Construction Methods 46

3.3.1 Introduction 46

3.3.2 Excavation 47

3.3.3 Drilling of Nail Holes 50

3.3.4 Nail Installation and Grouting 52

3.3.5 Drainage Placement 53

3.3.6 Initial Facing Construction 54

3.3.6a General 54

3.3.6b Shotcrete Application 55

3.3.6c Shotcrete Reinforcement 56

3.3.7 Final Facing Construction 58

3.3.7a Reinforced Shotcrete 58

3.3.7b Cast-in-Place Reinforced Concrete 58

3.3.7c Precast Concrete Facing 59

3.4 Alternate Materials 60

3.4.1 Sculpted Wall Facing 60

3.4.2 Drilled Drainage with Hollow Bar Soil Nails 62

Chapter 4: Information Required for Design 63

4.1 Introduction 63

4.2 Project-Specific Requirements and Initial Information 63

4.2.1 Project-Specific Requirements 63

4.2.2 Initial Information 63

4.3 Subsurface Investigations 64

4.3.1 Introduction 64

4.3.2 Objectives 65

4.3.3 Requirements for Selecting Location and Number of Exploratory Borings 66

4.3.4 Field Procedures 68

4.3.4a Overview 68

4.3.4b Test Borings and Standard Penetration Testing (SPT) 68

4.3.4c Cone Penetration Test (CPT) 69

4.3.4d Other Field Testing Techniques 70

4.3.4e Test Pits 70

4.3.5 Groundwater 71

4.3.5a Overview 71

4.3.5b Groundwater Observation Methods 71

4.4 Laboratory Soil Testing 72

4.4.1 Overview 72

4.4.2 Soil Classification and Index Tests 72

4.4.3 Shear Strength Tests 74

4.4.4 Deformability and Modulus 74

4.4.5 Other Testing 75

4.5 Selection of Design Parameters 75

4.5.1 Overview 75

4.5.2 Soil Parameters for Design 75

4.5.3 Unit Weight 76

4.5.4 Shear Strength 77
4.5.4a Cohesionless Soils 77
4.5.4b Cohesive Soils – Drained Conditions 79
4.5.4c Cohesive Soils – Undrained Conditions 80
4.5.5 Compressibility and Modulus 81
4.5.6 Bond Strength 82
4.5.6a Drilled and Grouted Soil Nails 83
4.5.6b Pullout Resistance per Unit Length 85
4.6 Soil Creep 86
4.7 Soil Corrosion Potential 87
4.7.1 Overview 87
4.7.2 Field Corrosion Testing 88
4.7.3 Testing of Corrosion Potential in Soil 88
4.7.4 Corrosive Soil Types 89
4.8 Frost Potential 90
4.9 Seismic Data 91
Chapter 5: Resisting Mechanisms and Limit States 97
5.1 Introduction 97
5.2 Load Transfer in Soil Nail Walls 97
5.2.1 Soil-Nail Interaction and Tensile Force Distribution 100
5.2.1a Overview 100
5.2.1b Maximum Tensile Force Distribution 100
5.2.1c Tensile Forces at the Wall Facing 103
5.2.1d Considerations for Design 104
5.3 Introduction to Design Framework 106
5.3.1 Use of LRFD and ASD Platforms 107
5.3.2 Allowable Stress Design (ASD) 107
5.3.3 Load and Resistance Factor Design (LRFD) 109

5.3.4 Calibration of Resistance Factors 110
5.4 Load Combinations and Load Factors 111
5.4.1 Load Combinations 111
5.4.2 Load Factors 111
5.5 Limit States 114
5.5.1 Overall Stability 114
5.5.2 Strength Limit States 114
5.5.3 Extreme-Limit States 114
5.5.4 Service Limit States (Deformations) 114
5.6 Overall Stability of Soil Nail Walls 116
5.6.1 Overview 116
5.6.2 Internal Stability 116
5.6.3 Global Stability 118
5.6.4 Idealized Example 118
5.6.5 Resistance Factors for Overall Stability 120
5.6.6 Basal Heave 121
5.6.6a Methodology 121
5.6.6b Resistance Factors for Basal Heave 124
5.6.6c Lateral Squeeze 125
5.7 Geotechnical Strength Limit States 125
5.7.1 Overview 125
5.7.2 Nail Pullout Resistance 125
5.7.3 Lateral Sliding Stability 126
5.7.3a Methodology 126
5.7.3b Resistance Factors for Lateral Sliding 130
5.8 Structural Strength Limit States 131
5.8.1 Nail Tensile Resistance 131
5.8.2 Overview of Facing Limit States 131

5.8.3 Facing Flexural Strength Limit State 132
5.8.3a Introduction 132
5.8.3b Bending Mechanisms 132
5.8.3c Resistance Factors for Flexure at Facing 139
5.8.4 Punching Shear Strength Limit State 139
5.8.5 Headed Stud in Tension in Final Facing 139
5.9 Service Limit States (Deformations) 140
5.9.1 Introduction 140
5.9.2 Soil Nail Wall Displacements 140
Chapter 6: Design of Soil Nail Walls 143
6.1 Introduction 143
6.1.1 Overview 143
6.1.2 Initial Design Considerations 145
6.1.2a Step 1 Project Requirements 145
6.1.2b Step 2 Subsurface Exploration and Development of Parameters for Design 145
6.1.2c Step 3 Load Definition 146
6.2 Design Phase 146
6.2.1 Overview 146
6.3 Step 4 Soil Nail Configuration and Material Selection 147
6.3.1 Introduction 147
6.3.2 Step 4a Develop Wall Layout 147
6.3.3 Step 4b Develop Soil Nail Wall Cross Sections 147
6.3.3a Wall Batter 147
6.3.3b Soil Nail Spacing 148
6.3.3c Soil Nail Inclination 150
6.3.3d Soil Nail Length 151
6.3.3e Distribution of Soil Nail Lengths in Elevation 152

6.3.4 Step 4c Select Soil Nail Pattern on Wall Face 154
6.3.5 Step 4d Evaluate Soil Nail Horizontal Splaying 154
6.3.6 Step 4e Detail Corrosion Protection 154
6.3.7 Step 4f Select Soil Nail Type and Material Properties 154
6.4 Step 5 Select Resistance Factors 155
6.5 Step 6 Overall Stability 156
6.5.1 Step 6a Evaluate Internal Stability 156
6.5.2 Step 6b Evaluate Global Stability 158
6.5.3 Step 6c Evaluate Basal Heave (if applicable) 159
6.5.4 Step 6d Evaluate Sliding Stability (if applicable) 160
6.6 Step 7 Verify Geotechnical and Structural Resistances 160
6.6.1 Overview 160
6.6.2 Step 7a Verify Pullout Resistance 161
6.6.3 Step 7b Evaluate Sliding Stability (if applicable) 163
6.6.4 Step 7c Verify Nail Tensile Resistance 163
6.6.5 Step 7d Verify Facing Bending/Flexural Resistance 164
6.6.5a Initial Calculations 164
6.6.5b Estimation of Nominal Flexural Resistance 167
6.6.6 Step 7e Verify Facing Punching Shear Resistance 172
6.6.7 Step 7f Verify Facing Headed Stud Resistance 176
6.6.8 Step 7g Other Facing Design Considerations 178
6.7 Step 8 Service Limit States (Deformations) 179
6.7.1 Step 8a Evaluate Wall Lateral and Vertical Displacements 179
6.7.2 Step 8b Evaluate Lateral Squeeze (if applicable) 180
6.8 Step 9 Seismic Design 180
6.8.1 Overview 180
6.8.2 Step 9a Select Design Seismic Parameters 181
6.8.3 Step 9b Adjustment of Design Seismic Coefficients 181

6.8.4 Step 9c Evaluate overall stability with seismic loads 184
6.8.5 Alternative Evaluation of Seismic Effects on Overall Stability 184
6.9 Step 10 Drainage and Drainage Details 188
6.9.1 Introduction 188
6.9.2 Short-Term Surface Water and Groundwater Control 189
6.9.3 Long-Term Surface Water and Groundwater Control 190
6.9.3a Control for Low Groundwater or Low Volumes of Seepage 190
6.9.3b Control for High Groundwater 191
6.9.3c Permanent Surface Water Control 192
6.10 Specialty Items 194
6.10.1 Stepped Walls 194
6.10.2 Composite Walls 196
6.10.3 Strut-Nails 198
6.10.3a Overview 198
6.10.3b Applications 200
6.10.4 Considerations for Frost Protection 201
6.11 Step 11 Other Design Considerations 203
6.12 Step 12 Pre-Construction, Construction, and Post-Construction 203
6.12.1 Pre-Construction 203
6.12.2 Construction and Post-Construction 203
Chapter 7: Corrosion Protection 205
7.1 Introduction 205
7.2 Historical Background 205
7.3 Terminology 206
7.4 Corrosion Mechanisms 207
7.4.1 Overview 207
7.4.2 Types of Corrosion 207

7.4.2a General Corrosion 208
7.4.2b Localized Corrosion 209
7.4.2c Stray Current 209
7.5 Evaluation of Soil Corrosion Potential 210
7.5.1 Conditions Promoting Corrosion 210
7.5.2 Criteria for Assessing Ground Corrosion Potential 211
7.6 Corrosion Protection Systems 212
7.6.1 Introduction 212
7.6.2 Grout Cover 213
7.6.3 Epoxy Coating 213
7.6.4 Zinc Coating (Galvanization) 215
7.6.5 Encapsulation 215
7.6.6 Sacrificial Steel 216
7.6.7 Nail-Head Corrosion Protection 217
7.6.8 Protection against Stray Currents 217
7.7 Methodology for Selecting Corrosion Protection Levels 217
7.7.1 Introduction 217
7.7.2 Selection Procedure 218
7.7.2a Overview 218
7.7.2b Risk Tolerance 219
7.7.2c Corrosion Protection Levels 220
7.8 Design of Corrosion Protection 222
7.8.1 Design Steps for Corrosion Protection of Soil Nails 222
7.9 Unusual Conditions 223
Chapter 8: Contracting Approaches and Specifications 225
8.1 Introduction 225
8.2 Geotechnical Analyses Required by Specifications 225
8.3 Experience Requirements for Contractors 227

8.4 Procedural Specifications 228
8.5 Performance-Based Specifications 228
Chapter 9: Construction Inspection and Performance Monitoring 231
9.1 Introduction 231
9.2 Inspection of Construction Materials 232
9.2.1 Materials Inspection 232
9.2.2 Storage 232
9.2.3 Inspection of Corrosion Protection 233
9.3 Inspection of Construction Activities 233
9.3.1 Introduction 233
9.3.2 Excavation 234
9.3.3 Drilling 235
9.3.4 Tendon Installation 237
9.3.5 Grouting 237
9.3.6 Geocomposite Strip Drains 239
9.3.7 Wall Facing 239
9.4 Load Testing 241
9.4.1 Introduction 241
9.4.1a Verification Load Tests 242
9.4.1b Proof Tests 243
9.4.1c Creep Tests 243
9.4.2 Equipment for Testing 243
9.4.3 Details of Verification Testing 246
9.4.4 Details of Proof Tests 253
9.4.5 Details of Creep Tests 255
9.4.6 Strain Gauges 257

9.5 Instrumentation and Monitoring 258
9.5.1 Introduction 258
9.5.2 Monitored Parameters 258
9.5.3 Monitoring and Action Plan 259
9.5.4 Monitoring Instruments 261
9.5.4a Inclinometers 261
9.5.4b Survey Points 261
9.5.4c Strain Gauges 262
9.5.4d Load Cells at the Nail Head 263
Chapter 10: Considerations for Hollow Bar Soil Nails 265
10.1 Introduction 265
10.2 Installation 265
10.3 Bond Strength 266
10.4 Corrosion Protection of HBSNs 267
10.4.1 Introduction 267
10.4.2 Sacrificial Steel 268
10.4.3 Design Steps for Corrosion Protection of HBSNs 269
10.5 Corrosion Protection Monitoring 269
References 273
Glossary 281
Appendix A: Reinforcement Characteristics 285
Appendix B: Charts for Preliminary Design 295
Appendix C: Design Example 301
Appendix D: Soil Nail Wall – Reference Manual Example Plans 333
Appendix E: Soil Nail Wall Procedural Specifications 343
Appendix F: Soil Nail Wall Performance-Based Specifications 367
Appendix G: Similar Technologies 383

List of Figures

Figure 2.1: Typical cross-section of a soil nail wall. Modified after Porterfield et al. (1994). 12

Figure 2.2: Illustration. Typical soil nail wall construction sequence. Modified after Porterfield et al. (1994). 15

Figure 2.3: Photo. Construction of soil nail wall. Photograph courtesy of Schnabel Engineering. 16

Figure 2.4: Illustration. Roadway cut supported with soil nails. Modified after Porterfield et al. (1994). 17

Figure 2.5: Illustration. Road widening under existing bridge. Source: Porterfield et al. (1994). 18

Figure 2.6: Illustration. Hybrid soil nail/MSE wall. Source: Wood et al. (2009). 20

Figure 2.7: Illustration. SMSE wall for steep terrain. Modified after Morrison et al. (2006). 21

Figure 3.1: Illustration. Main components of a solid bar soil nail and associated wall facing. Modified after Porterfield et al. (1994). 34

Figure 3.2: Photo. Threaded solid bars. Photograph courtesy of Con-Tech Systems, Ltd. 35

Figure 3.3: Photo. Threaded solid tendon with steel plate, washer, and nut. Note headed studs for connection to final facing. Photograph courtesy of DYWIDAG-Systems International. 36

Figure 3.4: Photo. Typical grout plant setup. Photograph courtesy of Barry Siel, FHWA. 39

Figure 3.5: Photo. Baroid mud balance to measure specific gravity. Photograph courtesy of Schnabel Engineering. 40

Figure 3.6: Photo. Molding grout cubes for strength verification. Photograph courtesy of Schnabel Engineering. 40

Figure 3.7: Photo. Typical steel centralizers attached to HBSNs. Photograph courtesy of Schnabel Engineering. 41

Figure 3.8: Photo. Typical PVC centralizers attached to a solid bar. (The spacing between centralizers depicted is for illustration only; wider spacing is used in practice.) Photograph courtesy of Williams Form Engineering Corp. 41

Figure 3.9: Photo. Solid tendons with gray and purple epoxy coating (left) and partially encapsulated by corrugated sheathing (right). Photographs courtesy of DYWIDAG-Systems International. 43

Figure 3.10: Photo. Curing of a shotcrete test panel on site. Photograph courtesy of Schnabel Engineering. 44

Figure 3.11: Photo. Shotcrete test panel cored for compressive testing of recovered specimens. Photograph courtesy of Schnabel Engineering. 44

Figure 3.12: Photo. Strip drain prior to initial shotcrete placement. Photograph courtesy of Schnabel Engineering. 46

Figure 3.13: Photo. Excavation of soil lift during construction of soil nail wall. Photograph courtesy of Moretrench American Corporation. 48

Figure 3.14: Illustration. Soil nail support of excavation with stabilizing berm. Modified after Porterfield et al. (1994). 49

Figure 3.15: Photo. Drilling nail holes. Photograph courtesy of Barry Siel, FHWA. 51

Figure 3.16: Photo. Solid nail tendon installation. Photograph courtesy of Schnabel Engineering. 53

Figure 3.17: Photo. Initial shotcrete application. Photograph courtesy of Moretrench American Corporation. 54

Figure 3.18: Photo. Shotcrete reinforcement including WWM and waler bars. Photograph courtesy of Ryan R. Berg & Associates, Inc. 57

Figure 3.19: Photo. Cast-in-place facing. Photograph courtesy of DYWIDAG-Systems International. 59

Figure 3.20: Photo. Precast panel facing. Photograph courtesy of Texas DOT. 60

Figure 3.21: Photo. Sculpted wall facing. Photography courtesy of The Collin Group, Ltd. 61

Figure 4.1: Illustration. Preliminary layout of borings. Modified after Sabatini et al. (2002). 67

Figure 4.2: Graph. Estimation of soil unit weight. Modified after U.S. Navy (1982), Kulhawy and Mayne (1990). 76

Figure 4.3: Graph. Friction angle of cohesionless soils as a function of normalized overburden (Modified after Schmertmann 1975). 79

Figure 4.4: Graph. Relationship between friction angle and plasticity index (after Terzaghi, Peck, and Mesri 1996). 80

Figure 5.1: Illustration. Potential slip surfaces and soil nail tensile forces. 98

Figure 5.2: Illustration. Potential critical stability case during construction. 99

Figure 5.3: Illustration. Stress-transfer mechanism in soil nails: (a) basic soil layout; (b) schematic distribution of shear stresses at the grout-soil interface; (c) schematic distribution of resulting tensile forces.....100

Figure 5.4: Illustration. Location of maximum tensile forces in soil nails. Modified after Byrne et al. (1998). 101

Figure 5.5: Chart. Summary of maximum nail tensile forces measured in soil nail walls. From Byrne et al. (1998).....103

Figure 5.6: Chart. Summary of facing tensile forces measured in soil nail walls. From Byrne et al. (1998). 104

Figure 5.7: Illustration. Limitations to tensile forces in nails: (a) pullout resistance controls, (b) tensile resistance controls. 106

Figure 5.8: Illustration. Potential limit states in soil nail walls: (a) internal stability (slip surface intersecting soil and nails); (b) global stability (slip surface not intersecting nails); (c) global stability: basal heave; (d) geotechnical strength: lateral sliding; (e) geotechnical strength: pullout; (f) structural strength: nail in tension; (g) facing structural strength: bending; (h) facing structural strength: punching shear; (i) facing structural strength: headed stud in tension.....115

Figure 5.9: Illustration. Simple wedge for overall stability verification. 119

Figure 5.10: Illustration. Basal heave for: (a) deposit of soft-fine grained soils, and (b) deposit of soft fine-grained soil underlain by stiff layer. Modified after Terzaghi et al. (1996) and Sabatini et al. (1999).....123

Figure 5.11: Graph. Bearing capacity factor (N_c) for determining basal heave. Modified after Terzaghi et al. (1996) and Sabatini et al. (1999). 124

Figure 5.12: Illustration. Lateral sliding of a soil nail wall. 130

Figure 5.13: Illustration. Limit states in soil nail wall facings. (a) typical facing layout; (b) bending of a facing section; (c) punching shear in initial facing; (d) punching shear in final facing; (e) tension of headed studs. Modified after Lazarte (2011). 134

Figure 5.14: Illustration. Bending mechanism and nail force in facing. Modified after Lazarte (2011). 136

Figure 5.15: Illustration. Section showing bending mechanism in initial facing. 138

Figure 5.16: Illustration. Deformation of soil nail walls. Modified after Clouterre (1991) and Byrne et al. (1998).....142

Figure 6.1: Illustration. Soil nail patterns on wall face. 149

Figure 6.2: Illustration. Varying nail patterns: (a) example of nail arrangement for non-horizontal ground, (b) varying nail inclination and lengths around utilities, and (c) nail splaying at corners. 151

Figure 6.3: Illustration. Overall and compound slip surfaces in complex soil nail walls. Modified after AASHTO (2014). 159

Figure 6.4: Illustration. Single nail stress-transfer mode: (a) soil nail layout, (b) distribution of mobilized bond stresses, and (c) hypothetical distribution of loads along the nail. 162

Figure 6.5: Illustration. Reinforcement nomenclature for facing in bending. 167

Figure 6.6: Illustration. Soil pressure distribution behind facing. Modified after Byrne et al. (1998). 169

Figure 6.7: Illustration. Limit states for punching shear in facing – horizontal cross sections: (a) bearing plate connection, (b) headed-stud connection. Modified after Byrne et al. (1998). 174

Figure 6.8: Illustration. Geometry of headed studs. 175

Figure 6.9: Illustration. Forces acting behind a soil wedge (Mononobe-Okabe Method). ...185

Figure 6.10: Chart. Seismic active pressure coefficients: (a) horizontal backslope, and (b) correction for non-horizontal backslope. 187

Figure 6.11: Chart. Effect of cohesion on the seismic active earth pressure coefficient. Source: NCHRP (2008). 188

Figure 6.12: Illustration. Drainage of soil nail walls. 192

Figure 6.13: Illustration. Typical drain pipe details. Modified after Byrne et al. (1998). 193

Figure 6.14: Illustration. Example of stepped soil nail wall. Source: Byrne et al. (1988). 194

Figure 6.15: Illustration. Composite wall structures: (a) cross section, and (b) soil nail walls with concrete piles. Source: Byrne et al. (1988). 196

Figure 6.16: Illustration. Strut-nail concept. Source: Byrne et al. (1998). 199

Figure 6.17: Illustration. Examples of frost protection: (a) first example, and (b) second example. Modified after Byrne et al. (1998). 202

Figure 7.1: Illustration. General corrosion of a tendon. After Hamilton et al. (1995) and Samtani and Nowatzki (2010). 207

Figure 7.2: Illustration. Procedure for selecting soil nail corrosion protection level. 218

Figure 7.3: Illustration. Class A corrosion protection: encapsulation. Modified from Byrne et al. (1998). 221

Figure 7.4: Illustration. Class B corrosion protection: galvanizing or epoxy-coating. Modified from Byrne et al. (1998).....221

Figure 7.5: Illustration. Class C corrosion protection: bare steel tendon. Modified from Byrne et al. (1998). 222

Figure 9.1: Photo. Soil nail load testing setup. Source: Cadden et al. (2010). 244

Figure 9.2: Photo. Soil nail load testing setup against shotcrete facing. Photograph courtesy of Schnabel Engineering. 245

Figure 9.3: Illustration. Hydraulic jack used for soil nail load testing. Source: Porterfield et al. (1994). 246

Figure 9.4: Chart. Typical data sheet for soil nail load testing.....251

Figure 9.5: Chart. Example of data reduction from soil nail load testing. 252

Figure 9.6: Chart. Example of data reduction from a soil nail creep test. 257

Figure 9.7: Illustration. Typical comprehensive monitoring instrumentation. Source: Byrne et al. (1998). 260

Figure 9.8: Photo. Strain gauge attached to a sister bar. Photograph courtesy of Schnabel Engineering. 262

List of Tables

Table 3.1a: Hydraulic Rotary Auger Methods for Drilling Competent Soils or Weathered Rock (Modified from Elias and Juran 1991) 50

Table 3.1b: Hydraulic Air and Water Rotary Methods for Drilling Competent Soils, Rock, or Mixed Ground Conditions (Pneumatic Down the Hole Hammer Available) (Modified from Elias and Juran 1991) 51

Table 3.1c: Pneumatic Rotary (Top Hole Hammer) Methods for Drilling Non-Caving Competent Soils or Rock (Modified from Elias and Juran 1991) 51

Table 4.1a: Procedures and Laboratory Tests for Soils – Classification 72

Table 4.1b: Procedures and Laboratory Tests for Soils – Index Parameters 72

Table 4.1c: Procedures and Laboratory Tests for Soils – Shear Strength Tests 73

Table 4.2a: Correlations between SPT N-values, CPT, and Friction Angle of Cohesionless Soils – SPT 78

Table 4.2b: Correlations between SPT N-values, CPT, and Friction Angle of Cohesionless Soils – CPT 78

Table 4.3a: Elastic Properties of Soils Based on Soil Type (after AASHTO 2014) 82

Table 4.3b: Elastic Properties of Soils Based on SPT N-values (after AASHTO 2014) 82

Table 4.3c: Elastic Properties of Soils Based on CPT Cone Tip Resistance (q_c) (after AASHTO 2014) 82

Table 4.4a: Estimated Bond Strength for Soil Nails in Coarse-Grained Soils (Modified after Elias and Juran 1991) 84

Table 4.4b: Estimated Bond Strength for Soil Nails in Fine-Grained Soils (Modified after Elias and Juran 1991) 85

Table 4.5: Estimated Bond Strength for Soil Nails in Rock – Drilling Method: Rotary Drilled (Modified after Elias and Juran 1991) 85

Table 4.6: Presumptive Ultimate Values of Soil Nail Pullout Resistance per Unit Length (Modified after Sabatini et al. 1999) 86

Table 4.7: Laboratory Test Standards 88

Table 4.8: Definition of Site Classes (Kavazanjian et al. 2011) 94

Table 4.9: Site Coefficient F_{PGA} (Kavazanjian et al. 2011) 94

Table 4.10: Site Coefficient F_v (Kavazanjian 2011) 95

Table 5.1: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method....108
Table 5.2: Load Factors and Load Combinations Based on AASHTO (2014)....112
Table 5.3: Load Factors, γ_p, for Permanent Loads....113
Table 5.4: Resistance Factors for Overall Stability....121
Table 5.5: Resistance Factors for Basal Heave....125
Table 5.6: Resistance Factors for Pullout....126
Table 5.7: Resistance Factors for Lateral Sliding....131
Table 5.8: Resistance Factors for Tendon in Tension....131
Table 5.9: Resistance Factors for Flexure Resistance at Facing....139
Table 5.10: Resistance Factors for Punching Shear at Facing....139
Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings....140
Table 5.12: Values of $(\delta_h/H)_i$ and C as Functions of Soil Conditions....141
Table 6.1: Main Steps – Initial Design Considerations....143
Table 6.2: Main Steps – Design....144
Table 6.3: Resistance Factors for Soil Nail Wall Design....155
Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement....166
Table 6.5: Factor C_F....170
Table 6.6: Nominal Flexure Resistance, R_{FF} (Initial Facing)....171
Table 6.7a: Facing Resistance – Shear Punching, R_{FP} (Initial Facing)....176
Table 6.7b: Facing Resistance – Shear Punching, R_{FP} (Final Facing)....176
Table 6.8: Headed Stud Resistance in Tension, R_{FH} (Final Facing)....178
Table 6.9: Maximum Values of Apparent Cohesion to Use with Mononobe-Okabe Method (NCHRP 2008)....188
Table 7.1: Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)....211
Table 7.2: Criteria for Assessing Ground Corrosion Potential....211
Table 7.3: Corrosion Protection Levels for Soil Nails....219
Table 9.1: Load Schedule in Verification Tests....249

Table 9.2: Load Schedule in Proof Tests 255

Table 10.1: Nominal Bond Strength of Soil Nails in Granular Soils (after Cadden et al. 2010) 266

List of Equations

Equation 5.1: Maximum soil nail load at head. 104

Equation 5.2: Maximum load in a soil nail. 105

Equation 5.3: General design equation in ASD. 107

Equation 5.4: General design equation in LRFD. 109

Equation 5.5: Capacity-to-demand ratio. 110

Equation 5.6: Factored resistance. 119

Equation 5.7: Factored driving force. 119

Equation 5.8: Mobilized pullout resistance under equilibrium. 120

Equation 5.9: Mobilized pullout resistance using ASD approach. 120

Equation 5.10: Nominal soil resistance. 122

Equation 5.11: Nominal soil resistance. 122

Equation 5.12: Total pressure at the base of the soil block. 122

Equation 5.13: Basal heave stability. 123

Equation 5.14: Basal heave stability with external load. 124

Equation 5.15: Simplified expression for basal heave stability. 124

Equation 5.16: Factor of safety against basal heave. 124

Equation 5.17: Lateral sliding stability. 127

Equation 5.18: Nominal resistance of soil at the base of soil block. 127

Equation 5.19: Lateral earth thrust. 128

Equation 5.20: Effective height over which earth pressure acts. 128

Equation 5.21: Active earth pressure coefficient according to Coulomb theory. 128

Equation 5.22: Active earth pressure coefficient according to Rankine theory. 129

Equation 5.23: Active earth pressure coefficient (vertical wall, horizontal backfill, and no friction at the soil interface). 129

Equation 5.24: Nominal compressive resistance. 135

Equation 5.25: Nominal tensile resistance. 135

Equation 5.26: Bending force in initial facing. 135

Equation 5.27: Equivalent cross-sectional area of rebar. 137

Equation 5.28: Total equivalent area per unit length. 137
Equation 5.29: Bending moment. 137
Equation 5.30: Maximum long-term horizontal and vertical displacement at the top of wall. ..141
Equation 5.31: Distance of significant soil deformation behind the wall. 141
Equation 6.1: Nominal unit pullout resistance. 161
Equation 6.2: Nominal pullout resistance. 162
Equation 6.3: Capacity-to-demand ratio (CDR). 162
Equation 6.4: CDR definition. 163
Equation 6.5: Nominal tensile resistance of tendon. 163
Equation 6.6: Reinforcement ratio. 165
Equation 6.7: Definition of reinforcement cross-sectional area per unit width in the vertical direction at the nail head. 165
Equation 6.8. Definition of reinforcement cross-sectional area per unit width in the horizontal direction at the nail head. 165
Equation 6.9: Vertical bending resistance of initial facing at midspan. 168
Equation 6.10: Vertical bending resistance of initial facing at nail head. 168
Equation 6.11: Horizontal bending resistance of initial facing at midspan. 168
Equation 6.12: Horizontal bending moment of initial facing at nail head. 168
Equation 6.13: Nominal bending resistance for bending in the vertical direction. 168
Equation 6.14: Nominal resistance for bending in the horizontal direction. 168
Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete. 169
Equation 6.16: Definition of F 170
Equation 6.17: Nominal bending resistance. 170
Equation 6.18: Capacity-to-demand ratio for bending in facing. 171
Equation 6.19: Capacity-to-demand ratio for punching shear resistance. 172
Equation 6.20: Nominal punching shear resistance at facing. 172
Equation 6.21: Nominal punching shear resistance through facing. 173
Equation 6.22: Effective, equivalent diameter of conical slip surface through initial facing. ..173
Equation 6.23: Effective depth of the conical slip surface, initial facing. 173

Equation 6.24: Effective, equivalent diameter of conical slip surface, final facing. 175
Equation 6.25: Effective depth of conical slip surface, final facing. 175
Equation 6.26: Headed stud resistance at facing. 177
Equation 6.27: Capacity-to-demand ratio for a headed stud. 177
Equation 6.28: Verification for sufficient cross-sectional area of headed stud. 177
Equation 6.29: Verification of sufficient thickness of headed stud. 177
Equation 6.30: Effective cross-sectional area of threaded bolts. 178
Equation 6.31: Design, site-corrected peak ground acceleration (Site Classes C through E). 181
Equation 6.32: Design, site-corrected peak ground acceleration (Site Classes A and B). 181
Equation 6.33: Seismic coefficient corrected for wall height. 182
Equation 6.34: Wall height correction factor. 182
Equation 6.35: Estimated wall seismic displacement. 183
Equation 6.36: Seismic coefficient for rigid wall. 183
Equation 6.37: Seismic coefficient assuming wall can tolerate a lateral movement up to 2 in. 183
Equation 6.38: Seismic coefficient assuming wall can tolerate a lateral movement greater than 2 in. 183
Equation 6.39: Reduction factor. 183
Equation 6.40: Total active thrust per Mononobe-Okabe method. 185
Equation 6.41: Active earth pressure coefficient per Mononobe-Okabe method. 186
Equation 6.42: Relation between horizontal and vertical seismic coefficients. 186
Equation 6.43: Term D as used in Mononobe-Okabe equations. 186
Equation 7.1: Thickness of sacrificial steel. 216
Equation 7.2: Remaining bar diameter. 217
Equation 9.1: Maximum bond length in verification tests (Grades 60 and 75). 247
Equation 9.2: Maximum bond length in verification tests (Grade 150). 247
Equation 9.3: Maximum load in verification tests. 248
Equation 9.4: Theoretical elastic elongation of the unbonded length in verification tests 250
Equation 9.5: Maximum bond length in proof tests (Grades 60 and 75). 253

Equation 9.6: Maximum bond length in proof tests (Grade 150).....253
Equation 9.7: Maximum load in proof tests. 254
Equation 9.8: Theoretical elastic elongation of the unbonded length in proof tests. 254

List of Symbols

a	Extent of effective width of facing section that is in compression
a_{hm}	Cross-sectional area of horizontal reinforcement per unit width at midspan
a_{hn}	Cross-sectional area of horizontal reinforcement per unit width at nail head
a_{ij}	Cross-sectional area of reinforcement per unit width in the "i" direction and "j" location
a_{s_eq}	Equivalent cross-sectional area of reinforcement per unit width of facing
a_{s_W}	Cross-sectional area of WWM per unit width of facing
a_{vm}	Cross-sectional area of vertical reinforcement per unit width at midspan
a_{vn}	Cross-sectional area of vertical reinforcement per unit width at nail head
A_E	Effective cross-sectional area of bolts (used as connectors in wall facing)
A_{gt}	Ductility (or uniform elongation) of steel without necking
A_H	Cross-sectional area of the head of a stud
A'_{HN}	Equivalent cross-sectional area of added reinforcement at nail head in the horizontal direction
A_{s_R}	Cross-sectional area of rebar
A'_{s_R}	Equivalent cross-sectional area of rebar
A_S	Cross-sectional area of the shaft of a headed stud
A_t	Cross-sectional area of tendon
A'_{VN}	Equivalent cross-sectional area of added reinforcement at nail head in the vertical direction
b_{PL}	Size of bearing plate in a strut-nail
B_e	Excavation width
B'_e	Width of soil block with potential to cause heave
B_L	Length of horizontal slip surface (sliding stability)
c	Compressive force per unit width in facing
c_b	Cohesion of soil along base of soil block (sliding stability)
c'	Effective cohesion of soil
c^*	Normalized cohesion
C	Soil-dependent coefficient for estimating soil nail wall deflections
CDR	Capacity-to-demand ratio
C_F	Factor to consider non-uniform soil pressures behind facing
C_P	Correction factor to account for contribution of the soil support (punching shear)
C_{RTY}	Reduction coefficient for mild-grade steel
C_{RTU}	Reduction coefficient for high-grade steel
C_{ST}	Compressive force in the strut nail

C_{1L}	Correction coefficient of nail length for drill hole diameter (App. B)
C_{2L}	Correction coefficient of nail length for cohesion (App. B)
C_{3L}	Correction coefficient of nail length for factor of safety (App. B)
C_{1F}	Correction coefficient of nail load for drill hole diameter (App. B)
C_{2F}	Correction coefficient of nail load for cohesion (App. B)
d	Distance from the outer edge of a facing section in compression to the centroid of the reinforcement (bending in facing)
d_{bar}	Initial bar diameter including sacrificial steel
d_e	Tolerable wall movement under seismic loading
d_f	Distance "d" for the final facing
d_i	Distance "d" for the initial facing
d_i	Thickness of subsurface layer "i" to estimate Site Class
d_{loss}	Net bar diameter after sacrificial steel is depleted
dT	Differential of T
dx	Differential of x
D	Term used in the calculation of K_{AE}
D_B	Depth to top of deposit of stiff soil underlying excavation
DC	Dead load from structural/non-structural attachments
D'_c	Effective equivalent diameter of conical slip surface at soil nail head
D_{DEF}	Distance to consider ground deformation behind a soil nail wall
D_{DH}	Drill hole diameter
D_E	Effective diameter of bolt core
D_H	Diameter of the head of a stud
D_r	Relative density
D_S, D_{SC}	Diameter of the shaft of a headed stud
D_{SH}	Diameter of the head of a stud
DW	Dead load from wearing surfaces and utilities
E	Young's modulus of steel
E_s	Young's modulus of soil
EH	Horizontal earth pressure
EQ	Earthquake loading
ES	Horizontal component of permanent surcharge
EV	Vertical earth pressure load
f'_c	Compressive strength of concrete
f_u	Ultimate strength of steel
f_y	Yield strength of steel
$f_{y\text{-}f}$	Yield strength of reinforcement
$f_{y\text{-}hs}$	Yield strength of headed stud
$f_{y\text{-}R}$	Yield strength of rebar

F	Factor used to estimate R_{FF}
F_{PGA}	Adjustment factor on PGA for site conditions
FS	Factor of safety
FS_{BH}	Factor of safety for basal heave
FS_{FF}	Factor of safety for facing bending/flexure
FS_{FH}	Factor of safety for headed stud in tension
FS_{FP}	Factor of safety for punching shear
FS_{LS}	Factor of safety for lateral sliding
FS_{OS}	Factor of safety for overall stability
FS_{PO}	Factor of safety for pullout
FS_T	Factor of safety for tendon in tension
F_v	Adjustment factor on S_1 for site conditions
F_{y_W}	Yield strength of WWM
h	Thickness of facing (generic)
h_c	Effective depth of conical surface
h_f	Thickness of final facing
h_i	Thickness of initial facing
H	Height of wall
H	Total thickness of soil layer (Site Class estimation)
H_1	Effective height behind a wall over which the earth pressure acts
H_1, H_2	Heights of walls 1 and 2, respectively, of a stepped soil nail wall
i	Inclination of soil nail from the horizontal
i	Generic number in a summation
I_P	Plastic index (alternative symbol)
k_h	Horizontal seismic coefficient
k_{max}	Maximum seismic coefficient
k_v	Vertical seismic coefficient
k_y	Yield acceleration
K_a	Active earth pressure coefficient
K_{AE}	Active earth pressure coefficient under seismic loading
L	Length of a soil nail
L_{BP}	Size of a square bearing plate
$L_{B\ PT}$	Bonded length in proof load tests
$L_{B\ PT\ max}$	Maximum bonded length in proof load tests
$L_{B\ VT}$	Bonded length in verification load tests
$L_{B\ VT\ max}$	Maximum bonded length in verification load tests
L_e	Length of excavation
L_{fail}	Length between nail head and slip surface
LL	Live load

LL	Liquid limit
L_p	Pullout length or soil nail length behind slip surface
LS	Live load surcharge
L_S	Length of headed stud
L_{SN}	Length of a strut-nail
L_{UB}	Unbonded length
m_{hm}	Horizontal bending resistance of facing at midspan per unit of width
m_{hn}	Horizontal bending moment of facing at nail head per unit of width
m_H	Horizontal moment per unit of width
m_{vm}	Vertical bending resistance of facing at midspan per unit of width
m_{vn}	Vertical bending resistance of facing at nail head per unit of width
m_V	Vertical moment per unit of width
n	Number to describe slopes, as in n (vertical):1(horizontal)
n_t	Number of threads per unit length of a bolt (connector)
N	Normal force on slip surface
N	Uncorrected SPT blowcount
N	Last number in a summation
N	Total number of rows of soil nail in a generic wall
$N_{_bar}$	Average SPT N-value (Site Class estimation)
N_c	Bearing capacity factor for cohesion
N_H	Number of headed-studs
N_i	SPT N-value for layer "i" (Site Class estimation)
N_k	CPT factor for use with estimations of soil shear strength
N_{60}	N-value corrected for an energy efficiency of 60 percent
$(N_1)_{60}$	Normalized and corrected SPT N-value
P_a	Lateral earth thrust
P_a	Atmospheric pressure
P_{AE}	Total active thrust including seismic loading
PGA	Peak ground acceleration
PGA_D	Design peak ground acceleration
PGV	Peak ground velocity
PI	Plasticity index
PL	Plastic limit
P_L	Limit pressure in the pressuremeter test
PTL	Proof test load
q	Mobilized shear stress along grout-soil interface
q	Total pressure at the base of a soil-nail reinforced block (basal heave)
q_c	Cone tip or bearing resistance
q_U	Bond strength or nominal bond resistance of soil nails

$q(x)$	Mobilized shear stress along grout-soil interface as a function of the distance "x"
Q	External surcharge (schematic wedge analysis)
Q_i	Total load at the base of a soil block
Q_i	Load component "i" in LRFD
Q_{ES}	Permanent surcharge
Q_{LS}	Live load surcharge
r	Ductility reduction factor (k_{max} estimation)
r_{PO}	Ultimate pullout strength (or load transfer rate) per unit length
R	Factor to correct soil nail lengths in the lower portion of a wall
R_{ALL}	Allowable resistance
R_F	Nominal resistance (general) of facing
R_{FF}	Nominal bending (or flexure) resistance of facing
R_{Fi}	Nominal resistance of facing at nail "i"
R_{FP}	Nominal resistance of facing for punching shear
R_{FH}	Nominal tensile resistance of headed studs in final facing
R_n	Nominal resistance of a generic structural component
R_{PO}	Nominal pullout resistance
R_{POi}	Nominal pullout resistance of nail "i"
R_S	Nominal resistance of soil (schematic wedge analysis)
R_S	Resistance of soil of soil block (sliding stability)
R_T	Nominal tensile resistance of tendon
R_{Ti}	Nominal tensile resistance of nail "i"
S	Generic spacing of soil nails, in the horizontal or vertical direction
S_H	Horizontal spacing of soil nails
S_{max}	Maximum spacing of soil nails
S_S	Spectral response acceleration at T = 0.2 sec
S_{SH}	Spacing of headed studs
S_u	Undrained shear strength of fine-grained soils
S_{u_bar}	Average S_u (Site Class estimation)
S_{u1}	Undrained shear strength of fine-grained soils behind a soil block (basal heave)
S_V	Vertical spacing of soil nails
S_{VN}	Vertical distance between the deepest row of soil nails to the bottom of excavation
S_{V0}	Vertical distance between the top of wall to the first row of soil nails
S_1	Spectral response acceleration at T = 1.0 sec
S_{1D}	Design value of S_1
t	Tension force per unit width in facing section

t_f	Service life in years (sacrificial steel)
t_{MAX}	Normalized soil nail load (App. B)
t_P	Thickness of bearing plate
t_{SH}	Thickness of the head of a stud
T	Oscillatory period of structure
T	Tensile force of a single equivalent soil nail per unit length (schematic description)
T_{max}	Maximum nail force
T_{max_i}	Maximum load in nail "i"
T_o	Tensile force at the nail head
T_T	Axial force in soil nail for strut nail concept
T_x, $T_{(x)}$	Load along soil nail as a function of x
$T_{1\ldots N}$	Tensile force in soil nails 1…N
T_{1maxi}, …	Maximum load in soil nail at slip surface
$v_{_bar}$	Average shear-wave velocity
v_{si}	Shear-wave velocity of layer "i"
V_F	Punching-shear force acting through facing
VTL	Verification test load
w_n	In situ or natural moisture content
W	Weight of generic soil wedge
W	Weight of facing
x	Coordinate along nail
x	Horizontal coordinate in SNAP-2
X	Loss of steel
y	Vertical coordinate in SNAP-2
z	Moment arm between forces "a" and "t"
α	Batter angle (from vertical) of wall face
α'	Batter angle (from vertical) of internal wall face
α	Correction factor for wall height (k_{max} estimation)
β	Correction factor for site response (k_{max} estimation)
β	Backslope angle
β_{eq}	Equivalent backslope angle
γ_d	Dry unit weight of soil
γ_{EH}	Load factor for horizontal effect of earth loads
γ_{EQ}	Load factor for seismic force
γ_{ES}	Load factor for horizontal effect of permanent surcharge
γ_{EV}	Load factor for vertical effect of earth loads
γ_i	Load modification factor associated with load Q_i

γ_{LS}	Load factor for live load
γ_{moist}	Moist unit weight of soil
γ_p	Load factor for permanent loads
γ_s	Unit weight of soil
γ'_s	Buoyant unit weight of soil
γ_w	Unit weight of water
δ	Friction angle between soil and reinforced-soil block (lateral sliding)
δ_h	Horizontal displacement at the top of a soil nail wall
$(\delta_h/H)_i$	Ratio to estimate the horizontal displacement at the top of a soil nail wall
δ_v	Vertical displacement at the top of a soil nail wall
ΔH	Height difference in backslope grades existing behind wall
Δ_{PTL}	Total movement measured at PTL
Δ_{VTL}	Total movement measured at VTL
η_i	Load modification factor for load "i" per LRFD
θ	Inclination of wall face from horizontal
θ_1, θ_2	Inclinations of regular soil nail and strut-nail, respectively
μ	Normalized bond strength (App. B)
ξ	Angle of orientation of slip plane
ρ	Reinforcement ratio
ρ_{ij}	Reinforcement ratio in the "i" direction and "j" location
ρ_m	Reinforcement ratio at mid-span
ρ_n	Reinforcement ratio at the nail head
ρ_{tot}	Total reinforcement ratio
σ'_{vo}	In situ effective overburden pressure
ν	Poisson's ratio
ϕ	Resistance factor
ϕ_{BH}	Resistance factor for basal heave
ϕ_{FF}	Resistance factor for bending/flexure
ϕ_{FH}	Resistance factor for headed stud in tension
ϕ_{FP}	Resistance factor for punching shear in facing
ϕ_{LS}	Resistance factor for lateral sliding
ϕ_{OS}	Resistance factor for overall stability
ϕ_{PO}	Resistance factor for pullout resistance
ϕ_T	Resistance factor for tensile capacity
φ_s'	Effective angle of internal friction of soil
φ_f	Friction angle of retained soil
φ'_{fb}	Effective friction angle of soil at the base of a soil block (sliding stability)

φ_s	Nominal value of friction angle of soil
ω	Angle relating individual seismic coefficients

List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AC/DC	Alternating current/direct current
ACI	American Concrete Institute
ADSC	Also known as IAFD, International Association of Foundation Drilling
AFI	Air-Freezing Index
AL	Alignment load
ANSI	American National Standards Institute
API	American Petroleum Institute
ASA	American Shotcrete Association
ASCE	American Society of Civil Engineers
ASD	Allowable stress design
ASTM	American Society for Testing and Materials
AWS	American Welding Society
CALTRANS	California Department of Transportation
CD	Consolidated drained triaxial compression (test)
CIP	Cast-in-place (concrete)
CL	Lean clay
CPT	Cone penetration test
CU	Consolidated undrained triaxial compression (test)
D/B	Design/Build
D/B/B	Design/Bid/Build
DFI	Deep Foundations Institute
DMT	Dilatometer test
DOT	Department of Transportation
EDM	Electronic distance measuring (equipment)
EPS	Extruded polystyrene
FDM	Finite difference method
FEM	Finite element method
FHWA	Federal Highway Administration
FS	Factor of safety
GEC	Geotechnical engineering circular
GP	Poorly-graded gravel
GW	Well-graded gravel
HBSN	Hollow bar soil nails
HDPE	High-density polyethylene
LI	Liquidity index
LL	Liquid limit
LOI	Loss-on-ignition

LRFD	Load and resistance factor design
ML	Low-plasticity silt
MSE	Mechanically stabilized earth (wall)
NCDC	National Climatic Data Center
NCHRP	National Cooperative Highway Research Program
NHI	National Highway Institute
NOAA	National Oceanic and Atmospheric Administration
PCA	Portland Cement Association
PE	Polyethylene
pH	Potential of hydrogen
PMT	Pressuremeter test
PR	Polarization resistance
PTI	Post-Tensioning Institute
PVC	Polyvinyl Chloride
QC/QA	Quality Control/Quality Assurance
ROW	Right-of-Way
SHRP2	Strategic Highway Research Program
SI	International System of Units
SM	Silty sand
SMSE	Shored mechanically stabilized earth (wall)
SP	Poorly-graded sand
SPT	Standard penetration test
SW	Well-graded sand
TRB	Transportation Research Board
TYP	Typical (used in figures and drawings)
UC	Unconfined compressive strength (test)
USCS	Unified Soil Classification System
USGS	United States Geological Survey
UU	Unconsolidated Undrained Triaxial Compression (test)
VST	Field vane shear test
WRI	Wire Reinforcement Institute
WWM	Welded wire mesh

Chapter 1: Introduction

1.1 Overview

This manual provides guidance for the design and construction of soil nail walls. Some of the most significant additions to this manual with respect to the previous version of this document include:

- Implementation of the Load and Resistance Factor Design (LRFD) platform
- Design considerations for hollow bars used as soil nails (HBSNs)
- Incorporation of more thorough corrosion criteria for soil nail design
- Inclusion of design example based on the SNAP-2 computer program (Siel 2014)

The primary audience for this document is: Owners (e.g., typically a public Agency); engineers specializing in bridge, structure, geotechnical, and roadway design; and engineering geologists and consulting engineers providing technical reviews, or who are engaged in the design, procurement, and construction of permanent soil nail systems. This document is also intended for management, specification and contracting specialists, as well as for construction engineers dealing with soil nail systems. In this document, the term "soil nail" refers to inclusions in soils and soft and weathered rock. The term "soil nail" is used in this document regardless of the material supported by the soil nail wall. This document only addresses permanent walls and includes a design framework implemented in LRFD for permanent structures.

1.2 Purpose

The purpose of this manual is to provide updated, state-of-the-practice information for selecting, designing, and constructing soil nail walls for roadway projects. The information contained herein is aimed at producing safe and cost-effective soil nail designs for roadway projects, and to help Owners to identify and manage the risks associated with soil nail wall projects. This manual focuses solely on soil nail systems providing long-term support of excavation of a permanent structure. This manual does not specifically address the use of soil nails as temporary structures for providing temporary support of excavation or to stabilize landslides.

1.3 Definition of Soil Nails

This section presents a definition of soil nails to distinguish between elements that are addressed in this manual and other systems that may resemble, but are fundamentally different from soil nails in one or more aspects. The definition of a soil nail herein

incorporates the following fundamental elements: resisting mechanisms, materials, construction methods, and construction Quality Control (QC) and Quality Assurance (QA).

Soil nails are reinforcing, passive elements that are drilled and grouted sub-horizontally in the ground to support excavations in soil, or in soft and weathered rock that:

- Contribute to the stability of earth-resisting systems mainly through tension as a result of the deformation of the retained soil or weathered rock mass.
- Transfer tensile loads to the surrounding ground through shear stresses (i.e., bond stresses) along the grout-ground interface.
- Develop resistances that can be estimated with established design procedures.
- Have long-term, demonstrable corrosion protection to ensure adequate, long-term performance of the system.
- Interact structurally with the facing of the excavation.
- Are load-tested according to prescribed methods.
- Are routinely subject to construction QC/QA according to established procedures.

Reinforcing elements that are post-tensioned, even if installed adjacent to conventional soil nails, are referred to as ground anchors, which are not addressed in this manual. Information on ground anchors can be found in Article 11.9 of AASHTO (2014) and "Ground Anchors and Anchored Systems," Geotechnical Engineering Circular No. 4, Report No. FHWA-IF-99-015 (Sabatini et al. 1999).

As stated in the definition, load transfer to and from the surrounding ground develops through shear stresses acting along the grout interface of the soil nail. As the reinforced-soil block deforms, shear stresses develop at the grout-ground interface. Because the retained soil deforms toward the excavation, soil nails undergo extension resulting in axial tensile forces in the soil nail tendon. The axial tensile load in the tendon increases from the nail head to a maximum value; then decreases as the soil nail transfers load to the surrounding ground. The tensile resistance of the soil nail tendon and the pull-out resistance of the soil nail are the main resisting mechanisms.

Various soil-reinforcing techniques involving the insertion of bars or rods in soils may bear some similarity to the soil nails described in this manual. Some of these alternative techniques, which are proprietary, may also incorporate the term "nail" in the brand name or in a loosely used denomination of some construction systems. Some of these alternative techniques, which do not meet the definition of soil nails, are described in Appendix G. This section also includes brief discussions of their differences with soil nails.

1.4 Background

1.4.1 Historical Origins of Soil Nail Walls

The insertion and grouting of metallic reinforcement into the ground, in combination with the use of shotcrete facing to provide excavation support, derives from the system developed for rock-excavation support known as the "New Austrian Tunneling Method" (Rabcewicz 1964a, 1964b, and 1965). The passive steel bars are known as rock bolts in the tunnel industry. The concept of combining passive steel reinforcement and shotcrete extended subsequently to rock-slope stabilization projects (Lang 1961). The use of this method was later expanded to stabilize slopes and excavations in soil.

In 1972 soil nails were used to stabilize an approximately 60-ft high cut-slope in sand for a railroad-widening project near Versailles, France (Rabejac and Toudic 1974). The method proved to be cost-effective and the construction faster than other conventional support methods.

The use of soil nails became common in France and other European countries since the completion of the Versailles project. The first use of soil nails in earth-retaining systems in Germany took place in 1975 (Stocker et al. 1979). The University of Karlsruhe, Germany, and Bauer, a large German contractor, partnered from 1975 through 1981 in a research program that involved full-scale testing of experimental walls (Gässler and Gudehus 1981, Schlosser and Unterreiner 1991). The French Clouterre research program involving private and public participants started in 1986. That research involved full-scale testing, monitoring of in-service soil nail walls, field testing, and numerical simulations (Schlosser 1983, Clouterre 1993 and 2000).

1.4.2 Developments in the United States

One of the first applications of this technology in the U.S. was in 1976 when soil nails were used to provide support to a 45-ft deep excavation made in lacustrine, dense, silty sands for the expansion of the Good Samaritan Hospital in Portland, Oregon. It was estimated that this system was completed in nearly half the time and at about 85 percent of the cost of conventional excavation-support systems (Byrne et al. 1998). In 1984 FHWA funded a demonstration project for the installation of a prototype, 40-ft high soil nail wall near Cumberland Gap, Kentucky (Nicholson 1986).

Since its introduction in the U.S., the use of soil nail walls has increased greatly for roadway projects. This increase can be attributed to the technical feasibility and cost-competitiveness of soil nailing. For certain subsurface and project conditions, soil nailing is more advantageous than other top-down, earth retaining systems because the construction

equipment is smaller, and it provides greater structural redundancy (i.e., a larger number of reinforcing elements per unit of wall area than other systems). Easements tend to be smaller for soil nail projects because soil nails are shorter than ground anchors, for example, given the same wall height. Additionally, as the use of soil nailing has grown, the number of qualified, soil-nail specialty contractors has increased. The project experience gained among engineers and Owners, especially state transportation agencies, has also increased over the years.

1.4.3 Leading Historic Documents in the United States

FHWA's sponsorship of demonstration and research projects has contributed greatly to the expansion of soil nail use in roadway projects since the early 1990s. The first FHWA-published document for the design and construction of soil nails was authored by Elias and Juran (1991). In 1993, FHWA sponsored a technical scanning tour in European countries leading this technology and published the tour findings (FHWA 1993a). FHWA also commissioned the translation of the French national manual on soil nailing into English (FHWA 1993b). In 1994, FHWA initiated "Demonstration Project 103" to disseminate the use of soil nailing among state transportation agencies. FHWA published the document titled "Soil Nailing Field Inspectors Manual, Demonstration Project 103" to assist the QC/QA of soil nail construction (Porterfield et al. 1994). "Demonstration Project 103" led to the update of the 1991 FHWA design document for soil nails (Byrne et al. 1998). FHWA also published the previous version of this manual (Lazarte et al. 2003).

In 2007, FHWA developed a series of multimedia presentation modules to: (i) describe the use of ground anchors and soil nails in roadway construction; and (ii) introduce this technology to various users utilizing a modern digital medium (Smith 2007). In 2010, FHWA published a report containing a review of factors affecting the corrosion of HBSNs and a series of recommendations to mitigate these factors (Samtani and Nowatzki 2010). FHWA also published FHWA-CFL/TD-10-001 (Cadden et al. 2010), which contains the results of a pullout testing program for HBSNs. The objectives of the pullout testing program were to: (i) develop estimates of bond stress of HBSNs as measured at four test sites; (ii) assess whether correlations existed between published values of bond resistances and conventional pre-drilled soil nails with solid bar tendons; and (iii) provide recommendations for practical, standard procedures for performing pullout tests of HBSNs.

The Strategic Highway Research Program (SHRP 2) launched the GeoTechTools web-based information and guidance system (SHRP 2013) to provide organized, detailed information on geo-construction technologies including soil nailing. FHWA is currently implementing the use of GeoTechTools with state transportation agencies, in cooperation with the American

Association of State Highway and Transportation Officials (AASHTO) and the Transportation Research Board (TRB).

1.5 Design Philosophy

This document includes a framework that allows the manual users to:

- Verify structural capacities in the conventional LRFD equation format, using load factor values from the AASHTO "LRFD Bridge Design Specifications" (2014).
- Utilize conventional Allowable Stress Design (ASD)-based, limit-equilibrium, computer programs developed specifically for designing soil nail walls to determine nominal loads in soil nail wall components.
- Incorporate resistance factors correlated to load factors included in AASHTO (2014) that would result in designs that are equal to or slightly more conservative than designs developed through ASD-based factors of safety (FS) as defined in previous versions of this manual.
- Incorporate correlated resistance factors that are rounded to the closest 0.05 and are reasonable and compatible with, but not necessarily equal to, those presented in AASHTO (2014) for other earth-retention structures and corresponding limit states.
- Incorporate correlated resistance factors that are consistent with a minimum required frequency of verification testing.

Therefore, the soil nail wall design presented herein relies on ASD-based stability calculations to quantify soil nail loads and slip surface geometries, and then uses these results to perform LRFD checks. The LRFD framework contained in this manual considers service, strength, and extreme-event limit states, consistent with those of AASHTO (2014).

1.6 Document Organization

The remaining chapters of this manual present the following information:

- Chapter 2 – "Applications and Feasibility Evaluations" provides an overview of the use of soil nails in roadway projects and an introduction to the main components and construction sequence of soil nail walls. The chapter also presents a description of favorable and unfavorable ground conditions for soil nailing, guidelines for conducting feasibility evaluations of soil nailing, and the risks for Owners using this technology. Finally, it lists factors affecting costs and construction schedule.
- Chapter 3 – "Construction Materials and Methods" contains detailed descriptions of the main components of soil nail walls and of the equipment and methods used in construction.

- Chapter 4 – "Information Required for Design" describes the information that engineers should obtain to perform a complete soil nail wall design. This information encompasses field and laboratory testing data and soil and rock parameters (including bond resistance). The chapter also lists the information needed for assessing corrosion potential for soil nails, for designing soil nail walls under seismic loading, and for assessing frost action.
- Chapter 5 – "Resisting Mechanisms and Limit States" provides descriptions of the resisting and load transfer mechanisms in soil nail walls. The chapter also includes a detailed discussion on the process followed for calibration of the various resistance factors used in soil nail wall design.
- Chapter 6 – "Design of Soil Nail Walls" presents the main steps to complete a full design. The chapter presents guidance for the verification of various strength and service limit states. Additionally, the chapter presents supplementary, special considerations in design, including: (i) special wall geometries; (ii) drainage; (iii) effects of frost; and (iv) effects of creeping soil. The step-by-step design procedure of this chapter is illustrated with a design example presented in Appendix C.
- Chapter 7 – "Corrosion Protection" presents a description of corrosion phenomena and their effects on metallic elements present in soil nail walls. It also contains criteria for selecting adequate levels of corrosion protection as a function of corrosion potential. The chapter introduces different systems to provide corrosion protection, including epoxy-coated bars, grout and other elements.
- Chapter 8 – "Contracting Approaches and Specifications" introduces guidance for the procurement of soil nail wall projects. The chapter also provides the key considerations for developing high-quality construction specifications.
- Chapter 9 – "Construction Inspection and Performance Monitoring" presents procedures for conducting construction QC/QA, including inspection of materials and construction activities. The chapter introduces requirements for performing soil nail load testing (both proof and verification tests), and instrumentation and monitoring during construction, as well as for long-term monitoring.
- Chapter 10 – "Considerations for Hollow Bar Soil Nails" includes discussions of the special items that owners, engineers and contractors need to consider for the design and construction of walls reinforced with HBSNs. While many design aspects of HBSN design are similar to those of solid bar soil nails, special considerations related to corrosion, soil nail testing, and QC/QA construction must be taken into account for HBSNs.

The manual also includes a list of the references used and the following appendices:

- APPENDIX A – Reinforcement Data Reference
- APPENDIX B – Charts for Preliminary Designs
- APPENDIX C – Design Example
- APPENDIX D – Construction Details for Soil Nail Walls
- APPENDIX E – Procedural Construction Specifications
- APPENDIX F – Performance-Based Construction Specifications
- APPENDIX G – Similar Technologies

Chapter 2: Applications and Feasibility Evaluations

2.1 Introduction

This chapter describes common applications of soil nails in roadway projects and presents typical components and the construction sequence of soil nail walls. It also contains a discussion about favorable and unfavorable subsurface conditions for cost-effective construction of soil nail walls to aid in evaluating their feasibility, and the main factors affecting the construction costs of these systems.

Soil nail walls are constructed using a "top-down" construction sequence, where the ground is excavated in lifts of limited height. Soil nails and an initial shotcrete facing are installed at each excavation lift to provide support. Subsequently, a final shotcrete or cast-in-place-concrete (CIP) facing is installed. Nails are most often installed at a vertical spacing of 4 to 6 ft. The nail vertical spacing is comparable to the typical height of a stable, excavation lift, which is commonly 3 to 5 ft and could be more in some soils. The horizontal spacing of nails is often also in the range of 4 to 6 ft. Figure 2.1 depicts a soil nail wall installed to permit excavation in the right-of-way for a road widening project. Both the soil nails and the initial and final facing contribute to the stability of the excavation. The soil nails support the soil and transfer loads to the soil mass behind the wall. The facing supports the soil between nails and immediately behind the face, provides structural continuity, and enables the soil nail wall to act as a unit.

Soil nail walls can be more advantageous than other top-down retaining systems where the ground can temporarily sustain short, vertical or sub-vertical unsupported cuts. Section 2.7 contains a thorough discussion on favorable and unfavorable soils for soil nailing.

Soil nail walls are permanent earth-retaining structures in most roadway projects in the U.S. However, soil nail walls are also constructed as temporary structures (i.e., typical service life of a few months) in roadway work when used as shoring of temporary excavations.

2.2 Elements of a Soil Nail Wall

The components of soil nail walls used in the U.S. practice are identified in Figure 2.1 and are briefly described below. More detailed descriptions of these components are included in Chapter 3.

2.2.1 Soil Nails

- **Tendons** – Tendons are the ground reinforcing elements behind a soil nail wall and equivalent to (steel) bars. These terms will be used interchangeably in this manual.

Tensile stress in each tendon mobilizes in response to lateral movement and deformation of the retained soil. Soil movement can occur during excavation, after excavation in the absence of external loads (as a result of time-dependent deformations), or after excavation when external loads such as surcharge or traffic loads are applied. The tendons can be solid or hollow bars. Solid bars are placed in stable drill holes and grouted in place. Hollow bars are fitted with a sacrificial drill bit and are used to drill the hole to then remain in place as the permanent soil nail reinforcement; they are described in detail in Chapter 10. Both solid and hollow bars are typically fully threaded.

- **Grout** – Grout used for soil nails usually consists of Portland cement and water. The grout functions to: (i) transfer shear stresses between the deforming ground and the tendons; (ii) transfer tensile stresses from the tendons to the surrounding stable soil; and (iii) provide some level of corrosion protection to the tendons. Grout is placed in the drill holes under gravity using the tremie method.
- **Corrosion Protection** – Soil nails used in permanent applications require chemical and/or physical protection against corrosion. The required level of corrosion protection is greater for soils with higher corrosion potential and for projects with lower risk tolerance.

  The lowest level of corrosion protection in U.S. practice is provided by the grout alone. Encapsulation of the bar provides the highest level of corrosion protection and is achieved by adding a protective sheath and grouting the bars in a phased process. Corrosion protection of the soil nail tendon can also be provided by application of a fusion-bonded, epoxy coating, galvanization, or sacrificial steel. Chapter 7 contains detailed descriptions of the corrosion protection techniques available for soil nails.

2.2.2 Facing

- **Shotcrete** – Facing consists of an initial and a final component. Soon after excavation, the initial facing is applied on the exposed soil at each excavation lift before or after nail installation to provide temporary stability and protection. The initial facing also receives the bearing plate of the soil nail. The final facing is constructed over the initial facing and provides structural continuity throughout the design life. The final facing may also include an aesthetic finish. The initial facing most commonly consists of reinforced shotcrete. The final facing generally consists of CIP-reinforced concrete, reinforced shotcrete, or precast concrete panels.

  Reinforcement used in the shotcrete of the initial facing includes the following items: (i) welded-wire mesh (WWM) installed over the entire excavation lift, and effectively over the entire wall using appropriate lap splices; (ii) horizontal bars (referred to as

waler bars) placed around nail heads to add bending resistance in the horizontal direction; and (iii) vertical bearing bars placed at nail heads to add bending resistance in the vertical direction (see Figure 2.1). Other reinforcement options include the use of steel or synthetic fiber particularly for temporary facing in soft or weathered rock. If the final facing consists of shotcrete, the reinforcement in the final facing is similar to that described for shotcrete in the initial facing. If the final facing consists of CIP or precast concrete, rebar mesh is typical.

2.2.3 Other Components

- **Connection Components** – The soil nail is connected to the facing through a number of components including: nuts, washers, bearing plates, and headed-studs. The headed studs are attached to the bearing plate and become embedded within the final facing as depicted in Figure 2.1.
- **Drainage System** – A drainage system is installed behind soil nail walls to: (i) collect perched groundwater or infiltrated surface water that is present behind the facing; and (ii) direct the collected groundwater away from the wall. The drainage system commonly consists of composite, geosynthetic drainage strips, also referred to as geocomposite strip drains. The drainage system does not provide full coverage of the wall area, but rather covers commonly 10-20%, or more, of the excavation face, depending on the selected strip drain spacing and commercial widths that are available.

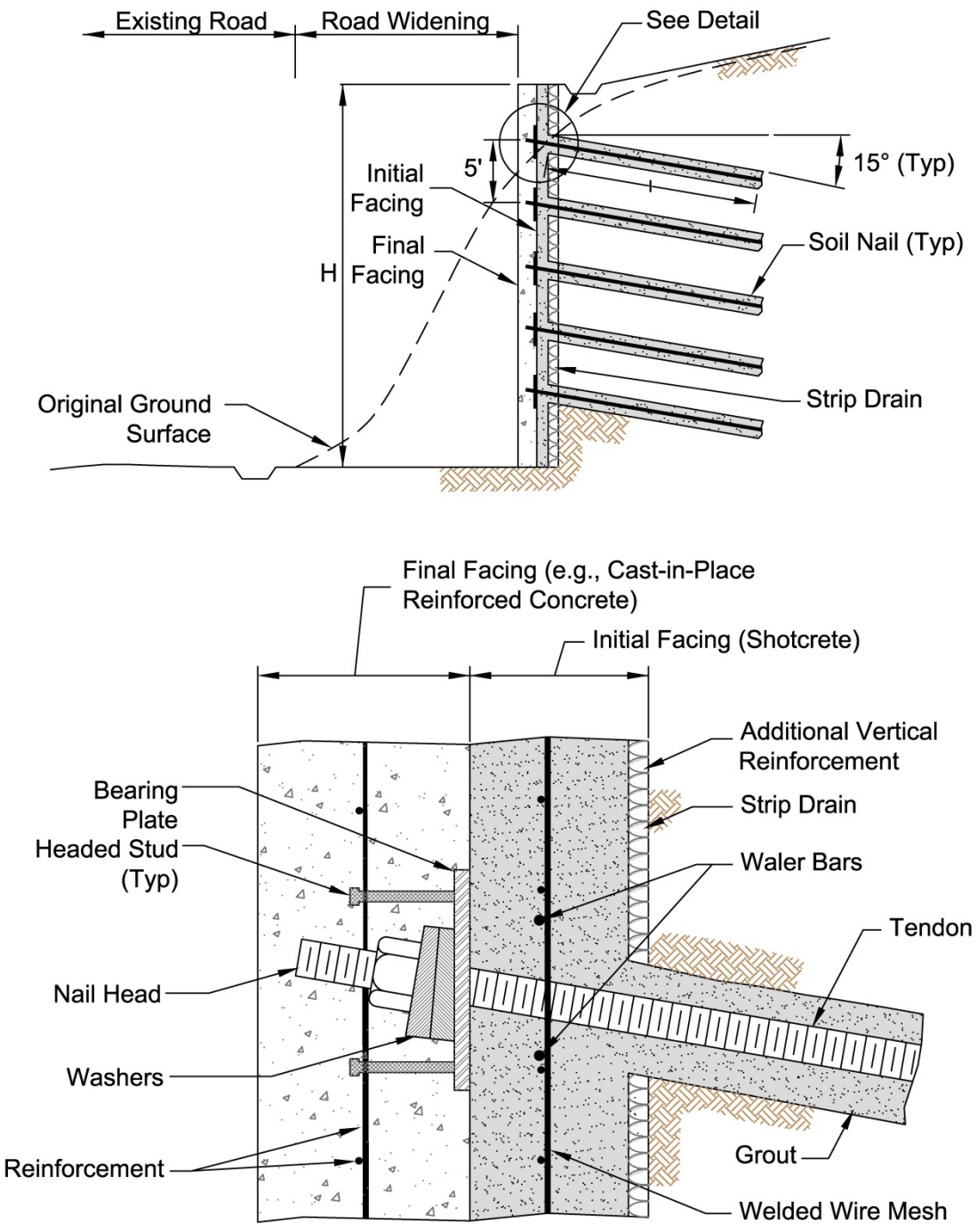


Figure 2.1: Typical cross-section of a soil nail wall. Modified after Porterfield et al. (1994).

2.3 Construction Sequence

The typical sequence of construction of a soil nail wall is described below and shown schematically in Figure 2.2.

Step 1. *Excavation*. The depth of the initial excavation lift (unsupported cut) may range between 2.5 and 7 ft, but is typically 3 to 5 ft and reaches slightly below the elevation where the first row of nails will be installed. The feasibility of this step is critical because the excavation face must have the ability to remain unsupported, until the nails and initial face are installed, typically one to two days. The type of soil that is excavated may limit the depth of the excavation lift. The excavated platform must be of sufficient width to provide safe access for the soil nail installation equipment.

Step 2. *Drilling of Nail Holes.* Drill holes are advanced using specialized drilling equipment operated from the excavated platform. The drill holes typically remain unsupported.

Step 3. *A) Nail Installation and Grouting.* Tendons are placed in the drilled hole. A tremie grout pipe is inserted in the drill hole along with the tendon; and the hole is filled with grout, placed under gravity or a nominal, low pressure (less than 5 to 10 psi). If hollow bars are used, the drilling and grouting take place in one operation.

B) Installation of Strip Drains. Strip drains are installed on the excavation face, continuously from the top of the excavation to slightly below the bottom of the excavation. The strip drains are placed between adjacent nails and are unrolled down to the next excavation lift.

Step 4. *Construction of Initial Shotcrete Facing*. Before the next lift of soil is excavated, an initial facing is applied to the unsupported cut. The initial facing typically consists of a lightly reinforced 4-in. thick shotcrete layer. The reinforcement includes welded-wire mesh (WWM), which is placed in the middle of the facing thickness (Figure 2.1). Horizontal and vertical bars are also placed around the nail heads for bending resistance. As the shotcrete starts to cure, a steel bearing plate is placed over the tendon that is protruding from the drill hole. The bearing plate is lightly pressed into the fresh shotcrete. Hex nuts and washers are then installed to engage the nail head against the bearing plate. The hex nut is wrench-tightened within 24 hours of the placement of the initial shotcrete. Testing of some of the installed nails to proof-load their capacity or to verify the load-specified criterion may be performed before proceeding with the next

excavation lift. The shotcrete should attain its minimum specified 3-day compressive strength before proceeding with subsequent excavation lifts. For planning purposes, the curing period of the shotcrete should be considered 72 hours.

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

Step 6. *Construction of Final Facing*. After the bottom of the excavation is reached and nails are installed and tested, the final facing is constructed. Final facing may consist of CIP reinforced concrete, reinforced shotcrete, or prefabricated panels. Weepholes, a foot drain, and drainage ditches are then installed to discharge water that may collect in the continuous strip drain.

Variations of the steps described above may be necessary to accommodate specific project conditions. For example, shotcrete may be applied at each lift immediately after excavation and before drilling of the holes and nail installation, particularly where stability of the excavation face is a concern. Another variation may be grouting the drill hole before placement of the tendon in the wet grout.

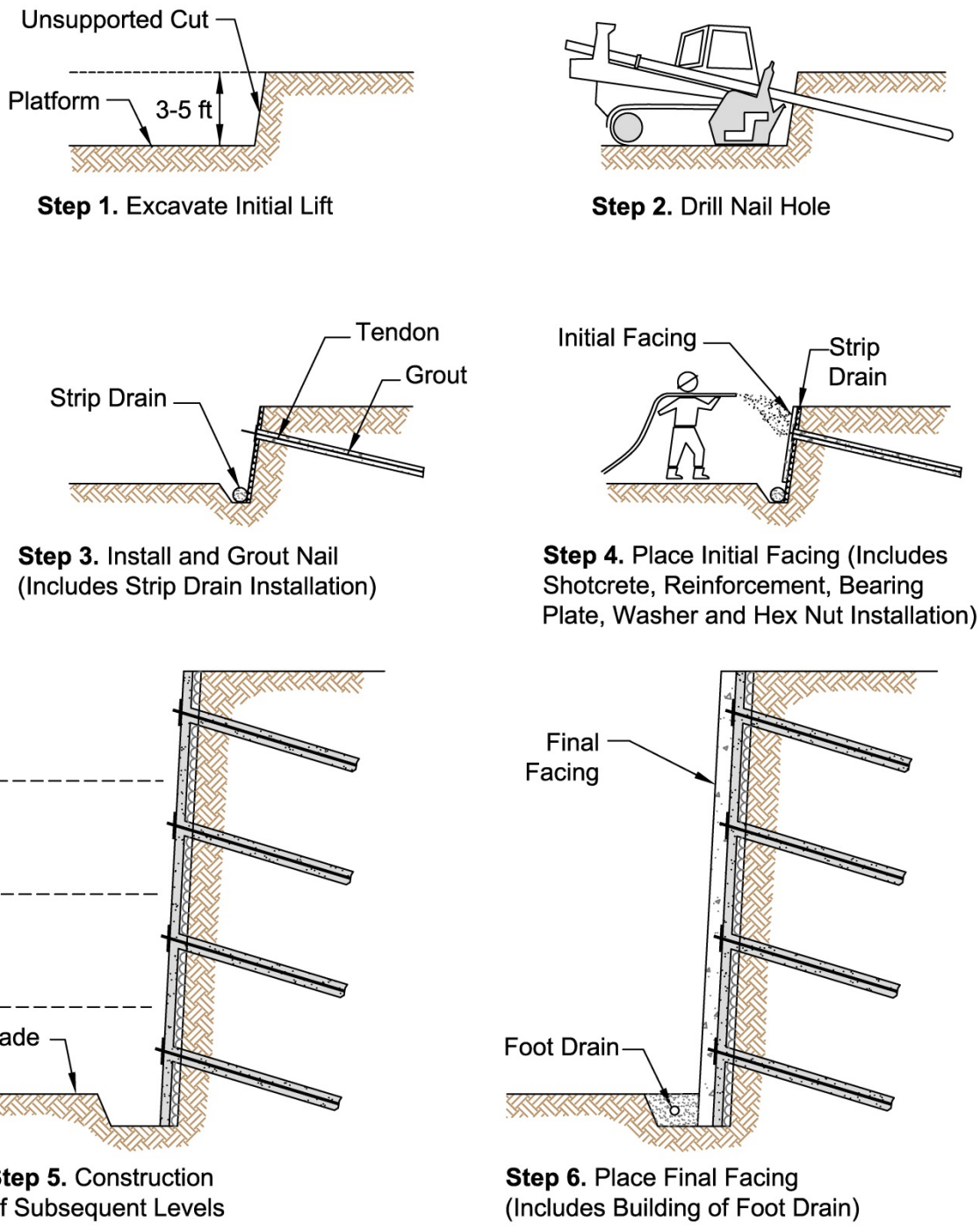


Figure 2.2: Illustration. Typical soil nail wall construction sequence. Modified after Porterfield et al. (1994).

2.4 Applications of Soil Nail Walls

2.4.1 Overview

Soil nail walls can be used in the following roadway applications:

- Roadway cuts
- Road widening under existing bridge abutments
- Tunnel portals
- Repair and reconstruction of existing retaining structures
- Hybrid soil nail systems
- Shored Mechanically Stabilized Earth (SMSE) walls

Figure 2.3 shows the construction of a soil nail wall for a large-cut face.



Figure 2.3: Photo. Construction of soil nail wall. Photograph courtesy of Schnabel Engineering.

2.4.2 Roadway Cuts

Soil nailing is attractive in roadway cuts because a limited excavation and reasonable right-of-way (ROW) and clearing limits are required (See Figure 2.4). These factors help to reduce the environmental impacts along the transportation corridor. The impact to traffic may also be reduced because the equipment for installing soil nails is relatively small.

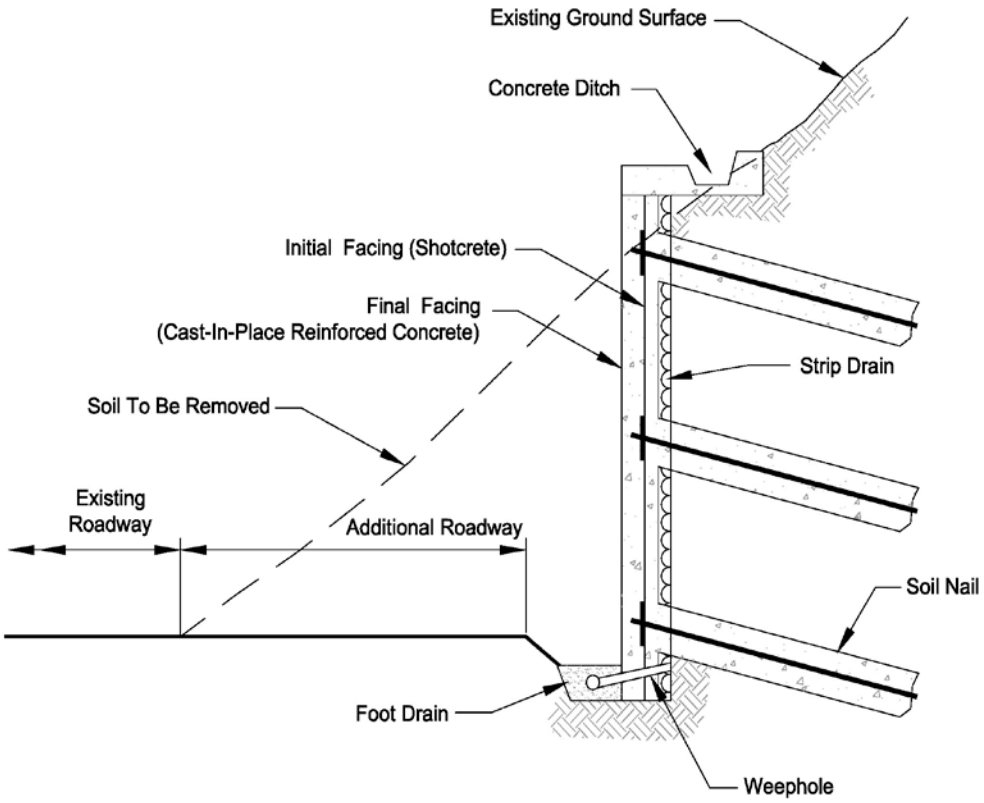


Figure 2.4: Illustration. Roadway cut supported with soil nails. Modified after Porterfield et al. (1994).

Variations in the details shown schematically in Figure 2.4 may exist, particularly to those details related to runoff control. Some of these variations will be discussed in Section 6.9 "Drainage and Details."

2.4.3 Road Widening Under Existing Bridge Abutments

Soil nail walls can be advantageous for underpass widening when the removal of an existing bridge abutment slope is necessary. While the cost of installing a soil nail wall under a bridge abutment may be comparable to that of other applicable systems, the advantage of soil nailing is that the size of the soil nail drill rig is relatively small. Soil nailing equipment can operate within limited overhead, and traffic flow along the underpass road may not need to be totally interrupted during the widening. The location, length, and inclination of soil nails need to be carefully planned so that the nails do not interfere with the existing bridge girders and do not intersect the existing abutment foundation. The upper soil nails must be positioned within the clear space between bridge girders and must be parallel to them. The remaining, lower rows of soil nails must be positioned and oriented to avoid hitting the

foundation elements below. Figure 2.5 depicts an example of road widening under an existing bridge.

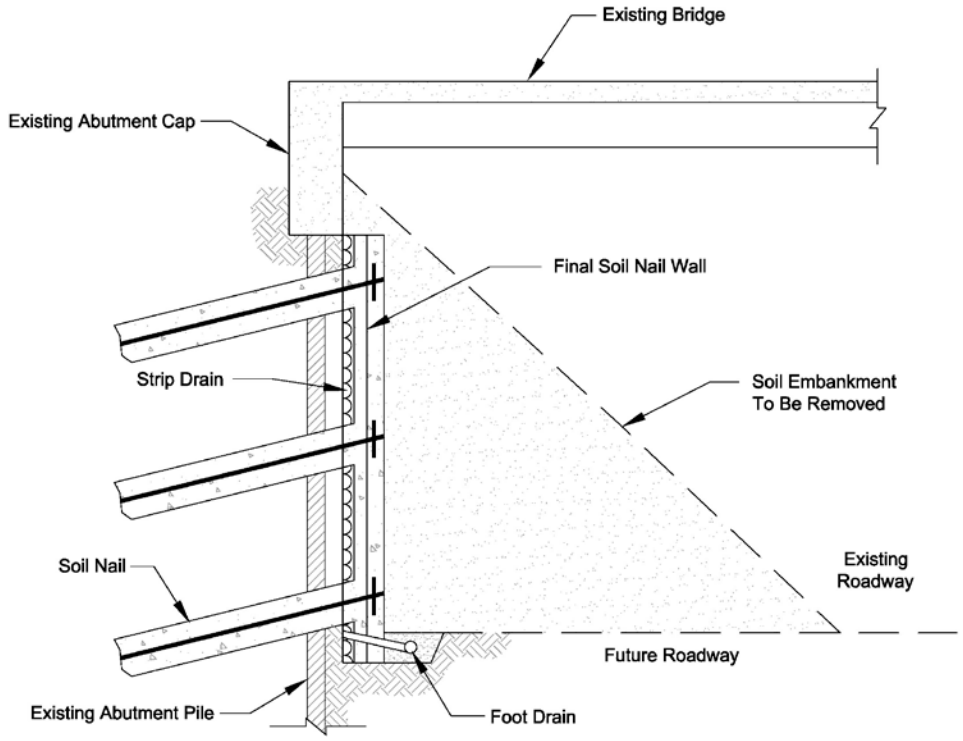


Figure 2.5: Illustration. Road widening under existing bridge. Source: Porterfield et al. (1994).

Vertical micropiles and soil nails have also been used in modified abutments for road widening projects where an existing bridge was originally supported on shallow foundations constructed on top of an existing embankment. The micropiles support the modified abutment and prevent settlements, as they transfer bridge loads below the new road grades. At the same time, the added soil nail wall retains the excavated soil. Because overhead clearances are small in most roadway widening projects, the combined use of soil nails and micropiles is attractive in these applications as they can help expedite construction by allowing the bridge to remain operational during widening of the underpass lanes.

2.4.4 Tunnel Portals

Tunnel portals can also be stabilized using soil nails. Although the principle behind the use of soil nails in tunnel portals is similar to that for road cuts, other aspects must be considered in the design and construction of this application. First, the vertical stability of the shotcrete facing above the tunnel must be considered. The potential transfer of soil nail loads to the

tunnel structure at the portal must also be taken into account. The interaction between soil nails and the initial shotcrete support and lining of the tunnel near the portal need to be fully evaluated. In addition, the layout of soil nails may be different than that in the conventional use in roadway applications. Soil nails must be installed with an appropriate horizontal splay and a suitable vertical orientation to avoid interfering with the tunnel support components.

2.4.5 Repair and Reconstruction of Existing Retaining Structures

Soil nails can be used to stabilize and/or strengthen failing or distressed retaining structures. For example, some mechanically stabilized earth (MSE) walls may exhibit excessive deformation due to poor design, poor construction, or both. Soil nails can be installed directly through the face of an MSE wall if the existing face is sufficiently stable to resist drilling. As the MSE wall continues to deform, the backfill of the MSE wall and its facing would transfer loads to the installed soil nails, and these would transfer loads to stable soils lying behind the MSE-reinforced block of soil.

Of particular concern in these cases is the use of appropriate drilling and grouting procedures. These procedures should not be detrimental to the stability of the existing MSE wall, and should create a stable drill hole through the wall backfill and through any drainage material placed at the back of the MSE wall. In some cases, it may be necessary to stabilize the MSE wall face before the start of drilling to prevent partial or total collapse. Similar considerations apply to the use of soil nails in the stabilization of crib walls, gabion, and masonry walls.

The selection of an appropriate bearing plate to support soil nails stabilizing MSE and masonry walls is very important. The bearing plate must be able to fully transfer loads without damage to the existing facing. In some cases, consideration of the maximum load at the bearing plate may control the design.

2.4.6 Hybrid Soil Nail Walls

Soil nail walls can be used with other types of wall systems such as ground anchor walls and MSE walls to combine the advantage of each method. This situation may arise for walls with a complex layout or when the costs associated with other earth-retaining systems are too high. The combination of MSE and soil nail walls may provide a more economical design in cut/fill situations than the traditionally used full-height MSE walls or drilled shaft retaining walls (Wood et al. 2009). Figure 2.6 shows an example of a hybrid soil nail/MSE wall.

Another example of a hybrid soil nail wall is the combination of soil nails with ground anchors. This application may be used in areas where utilities or other underground obstructions exist and do not allow soil nails to be installed in the upper sections. In such a

case, the wall facing can be designed to work in cantilever in the upper 8 to 10 ft, with one or two rows of ground anchors providing horizontal restraint at the bottom of the cantilever. Soil nails are used in the rows below.

Soil nails are not generally well suited to address instability along deep-seated slip surfaces. Where there is potential for deep-seated instability of a proposed roadway cut, ground anchors can be introduced in combination with soil nails. Soil nails may also be used for tiered walls to support the top of the slope, where one or two rows of relatively long ground anchors would intersect the slip plane and provide global stability.

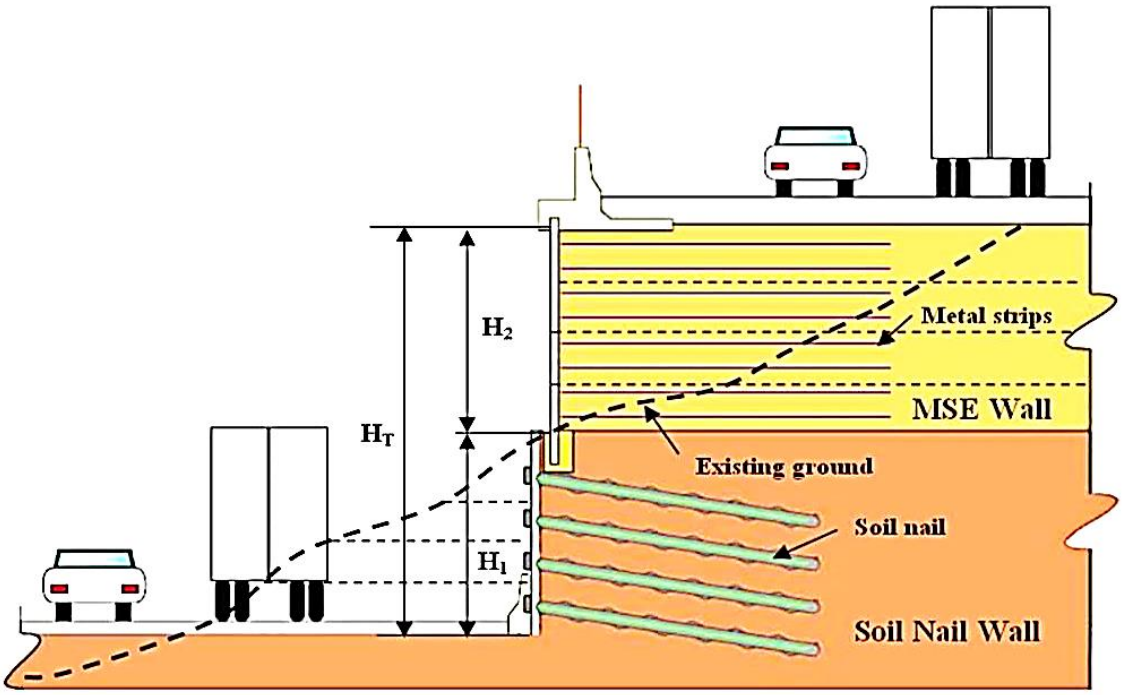


Figure 2.6: Illustration. Hybrid soil nail/MSE wall. Source: Wood et al. (2009).

2.4.7 Shored Mechanically Stabilized Earth (SMSE) Walls

Soil nail walls have been increasingly used in combination with MSE walls for widening low-volume roads by fill placement in steep terrain. MSE wall construction in steep terrain requires excavation to establish a flat bench to place the soil reinforcement. The required depths of embedment increase with the steepness of the slope below the wall toe. If the slope is too steep, the excavation for the MSE wall becomes impractical, particularly in situations where traffic must be maintained during construction of the widening. Soil nail walls can be used as shoring to stabilize the backslope (or back-cut) first, and then allow the construction of a conventional MSE wall in front of the soil nail wall. Figure 2.7 shows a generic cross-section of this combination.

If the soil nail wall is designed as a permanent wall, nails can significantly reduce the long-term lateral pressures on the MSE wall. This configuration is known as an SMSE wall. Details of SMSE walls are presented in the ASD platform in FHWA-CFL/TD-06-001 (Morrison et al. 2006), and are updated in the LRFD platform in FHWA-NHI-10-024/FHWA GEC 11 (Berg et al. 2009).

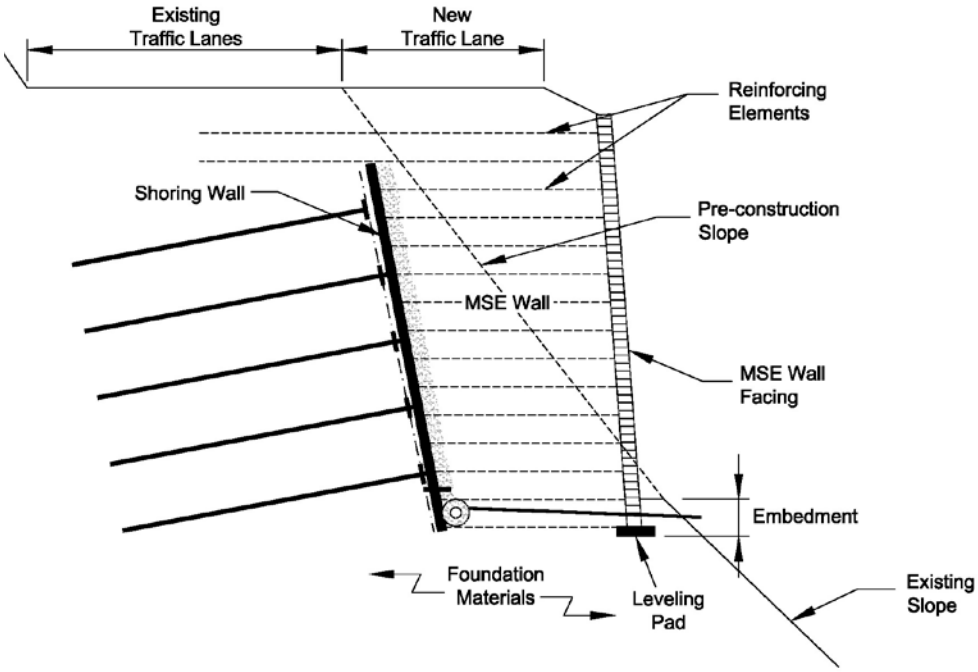


Figure 2.7: Illustration. SMSE wall for steep terrain. Modified after Morrison et al. (2006).

2.5 Advantages and Limitations of Soil Nailing

2.5.1 Advantages

Advantages associated with soil nailing fall into three main categories: Construction, Performance, and Cost.

2.5.1a Construction

- Soil nail walls require smaller ROWs than most other competing systems. This is also true for ground anchors as soil nails are typically shorter.
- Soil nail walls are less disruptive to traffic and cause less environmental impact compared to other construction techniques such as drilled shafts or soldier pile walls, which require relatively large equipment.

- Soil nailing causes less congestion in the excavation when compared to braced excavations.
- The installation of soil nail walls is relatively fast.
- Easy adjustments to nail inclination and location can be made when obstructions are encountered, such as boulders, piles or underground utilities. As a comparison, adjustments in the horizontal position and orientation of ground anchors often require changes to the soldier pile layout or the addition of waler beams, making adjustments in the field costly.
- Soil nail wall installation is not as restricted by overhead limitation as in the case of soldier pile installation. This advantage is particularly important when construction occurs under a bridge.
- Soil nailing may be more cost-effective at sites with remote access because the smaller equipment is more readily mobilized.
- Soil nails are installed using equipment that is multipurpose and can be used for other substructure elements such as underpinning or protection of adjacent, movement-sensitive structures.
- A relatively large number of qualified soil nail contractors exists.
- A widespread knowledge about soil nailing exists among engineers.
- Soil nail walls can accommodate curves and "bends" more easily than other top-down construction wall systems, which would otherwise require straight wall segments.

2.5.1b Performance

- Soil nail walls are relatively flexible and can accommodate comparatively large total and differential movements.
- The measured deflections of soil nail walls are usually within tolerable limits in roadway projects when the construction is properly controlled.
- Soil nail walls have performed well during seismic events.
- Soil nail walls have more redundancy than anchored walls because the number of reinforcing elements per unit area of wall is larger than for anchored walls.
- Sculpted facings, which can be applied to soil nail walls, give a more natural appearance than other finishes, to fit in with the surrounding environment.

2.5.1c Cost

- Conventional soil nail walls tend to be more economical than conventional concrete gravity walls taller than approximately 12 to 15 ft.
- Soil nail walls are typically equivalent in cost or more cost-effective than ground anchor walls when conventional soil nailing construction procedures are used.

2.5.2 Limitations

The main limitations associated with soil nailing are:

- In projects where strict wall movement criteria exist, additional measures to limit deflections may be required. These requirements would add cost. If very strict movement criteria exist, soil nails may not be a feasible option for the project.
- The existence of utilities behind the wall will likely create restrictions to the location, inclination, and length of soil nails, particularly in the upper rows.
- Soil nail walls are not well-suited where large amounts of groundwater seep into the excavation. Soil nail walls require maintaining a temporary unsupported excavation face during construction.
- Permanent soil nail walls require permanent underground easements.
- Soil nail tendons may interfere with certain types of communication lines (e.g., optic fiber) running immediately adjacent to soil nail walls.

2.6 Evaluation of Soil Nail Walls

2.6.1 Overview

The feasibility evaluation of a soil nail wall should encompass technical and economic considerations and include:

- An evaluation of the prevailing ground conditions
- An assessment of the advantages and disadvantages of soil nail walls, as well as the risks associated with these systems for the particular application being considered
- A comparison of soil nail walls with alternative systems such as ground anchor or braced wall systems
- A preliminary evaluation of costs

The evaluation of prevailing ground conditions is discussed in Chapter 4. The evaluation of existing ground conditions should include assessing the feasibility of a stable test cut section. While stable cuts can be between 3 and 5 ft high, as described earlier, taller cuts, in the range between 4 and 6 ft, would be more indicative of favorable subsurface conditions. Unfavorable stability conditions observed during test sections may also be related to other undesirable conditions, such as potential caving of drill holes if these are uncased.

The following sections present a discussion of the main risks associated with soil nails and the best practices that need to be considered to manage those risks. Besides the brief comparison with other systems presented above, in-depth comparisons of soil nail walls with alternative systems will not be presented herein.

2.6.2 Main Risks and Risk Management

Risks associated with the construction of soil nail walls include those common to all earth-supporting systems, such as the lack of:

- A thorough and adequate subsurface investigation
- An adequate selection of material parameters for design
- A proper analysis model of the system
- Thorough construction QC and QA procedures
- Verification of both short-term and long-term performance through geotechnical monitoring (generally only needed in cases where sensitive structures exist behind the soil wall, or when other conditions of long-term risk arise)

Risks specifically associated with the construction of soil nail walls include the following:

- Corrosion may impact the long-term performance of steel bars and other metallic components if not properly considered in design.
- Large amounts of groundwater seeping into an excavation may complicate the construction of soil nail walls, particularly of the lower, exposed soil excavation lifts. High groundwater may also affect the long-term performance of the system.
- Design procedures for soil nail walls require an effective confirmation of the design criteria used for the pullout resistance of soil nails.
- Significant risks may be created if some key elements of design are not given adequate consideration in specifications and construction schedule. For example, key elements of soil nail wall design, such as global stability or deformation analyses, may not be properly considered if the Owner delegates the entire design to the Contractor, but the Contractor is not able to provide these due to lack of resources and/or constrained schedule.

Measures to control or manage risk in soil nail wall projects include:

- In general, the Owner must obtain sufficient site data for internal and external design. The Owner must ensure that corrosion potential of the ground is thoroughly evaluated by the Owner's engineer. Alternatively, the Owner could require the evaluation of corrosion potential to be the Contractor's responsibility. These evaluations will indicate which appropriate level of corrosion protection is necessary. The Owner should provide input about their tolerance to risk of corrosion and ability to cover costs of adequate corrosion protection methods.
- The Owner's engineer must ensure that reasonable movement criteria are selected for design. If strict deformation control is required, the Owner's engineer must evaluate ground movement during design, possibly using numerical analyses to support the

selection of appropriate means to control wall movement. In addition, the engineer may need to consider developing short-term, and possibly long-term, monitoring of wall movements to confirm adequate performance. As part of this effort, the engineer should develop adequate action plans for monitoring and remedial response during construction. These evaluations are important because this assessment might indicate that soil nail walls are not a cost effective system if strict deformation control is required.

- The Owner's engineer must include evaluations of groundwater and potential for seepage into the excavation in the geotechnical report and during the design stage. Drainage systems may need to be incorporated in design. These evaluations may also indicate that soil nail walls are not a cost-effective system at a site.
- To confirm values of soil nail pullout resistance used in design, a verification and proof load test program must be conducted in all soil nail projects.
- The Owner should engage their engineer in performing global stability and deformation analyses (if needed) before bids. Results of these evaluations should be made available to the Contractor. If these design elements are delegated to the Contractor, the Owner must develop contract documents that describe clearly the Contractor's design qualifications and submittals, the scope of these design elements, design criteria, and performance criteria (if applicable).

2.6.3 Aesthetics

The aesthetics of a soil nail wall can be an important decision-making factor for an Owner considering a variety of wall types. A designer should be sensitive to this concern as it is often the only aspect of wall construction that the Owner may provide input on. Shotcrete facing has a rough and irregular appearance. Shotcrete can be improved by trowels or brooms.

Some options are available to further improve appearance at a cost. Shotcrete and concrete can be dyed, sculpted, and/or molded to represent natural rock formations or stone or brickwork. In addition, the protruding nail head can be configured as a connection to support other types of facing as well, such as MSE wall panels. These aesthetic options can be expensive compared to shotcrete as a permanent facing. Further details about facing options are presented in Chapter 3.

2.7 Favorable/Unfavorable Soil Conditions for Soil Nailing

2.7.1 Overview

Soil nail walls can be used in a wide range of soil types and ground conditions. Project experience has shown that certain favorable ground conditions make soil nailing more cost-

effective than 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, stable materials. Conversely, certain unfavorable soil conditions can be considered marginal or difficult for soil nailing applications and may make the use of soil nails risky and/or more costly when compared with other techniques.

The following paragraphs present soil conditions that are considered favorable, difficult, and unfavorable for soil nail walls.

2.7.2 Favorable Ground Conditions for Soil Nailing

Soil nailing has proven economically attractive and technically feasible in the following conditions:

- The excavated soil can stand unsupported in a 4- to 6-ft high vertical or nearly vertical cut for one to two days.
- Soil nails, when installed in a relatively permeable formation, are located above the groundwater table.
- Ground conditions allow drill holes to remain stable without using casing until the tendons are installed and the drill hole is grouted.

As mentioned, soil conditions are presumed to be favorable for the construction of soil nail walls when field tests indicate that competent soils are present above the groundwater table. It is possible to construct a soil nail wall under the groundwater table, as long as nails are installed in fine-grained soils that exhibit low permeability and produce little seepage. However, this scenario might not be necessarily technically feasible for other reasons, or might not be cost effective. The design engineer must consider the long-term evolution of pore pressures, and effects on the internal and global stability of the wall. The design engineer must evaluate the potential for developing excessively large wall deflections in these conditions. Soil nail wall deflections tend to be larger in saturated, fine-grained soils with low permeability (and likely some plasticity) when compared to deflections in granular soils. The Contractor must use suitable drilling procedures in these soils that would not cause the ground to deteriorate.

The following ground types are examples of conditions generally considered well-suited for soil nailing applications:

- *Dense to very dense granular soils with apparent cohesion.* These soils include dense to very dense sand, gravel, or a combination thereof, with some fines (typically less than 10 to 15 percent) or with weak, natural cementation providing cohesion, and with SPT N_{60}-values (see definition in Chapter 4) greater than 30 blows per foot

(bpf). Capillary forces can also produce apparent cohesion in fine sands, as long as these soils remain unsaturated: neither dry nor saturated. Because there are currently no widely accepted criteria to assess the minimum amount of apparent cohesion required for soil nailing in granular soils, local experience will play a significant role when assessing the suitability of soil nail walls in these soils. Also, the standup ability of the ground can be established during the geotechnical site investigation through excavation of test pits or face cuts. A minimum apparent cohesion of 200 to 300 psf is necessary for stability of 4-ft tall, unsupported excavation faces. Some measures must be taken to avoid excessive breakage of capillary forces and thereby significant reduction of the apparent cohesion. These measures may include: (i) limiting the exposure of the cut excavation in dry weather to no more than a few hours to avoid soil desiccation; (ii) preventing surface water from infiltrating into the ground immediately behind the excavation; and (iii) preventing seepage toward the excavation face.

- *Weathered rock with adverse weakness planes.* Weathered rock may be a suitable material for installing soil nails as long as weakness planes, if present, occur with favorable orientations. It is also desirable that the degree of weathering of the rock be approximately uniform throughout so that only one drilling and installation method is required. If a highly variable degree of rock weathering exists at a site, the Contractor must be prepared to drill through a variety of ground conditions without retooling or changing equipment that may increase soil nail installation costs with respect to those in formations that are more consistent to drill.
- *Stiff to hard fine-grained soils*. These soils include stiff to hard clays, clayey silts, silty clays, sandy clays, and sandy silts. Fine-grained soils are classified as stiff if the measured SPT N_{60}-values are 9 bpf or greater. However, the characterization of the consistency of fine-grained soils should be performed with caution when SPT N or N_{60}-values are obtained with non-automatic hammers or other antiquated methods. The consistency characterization should be supplemented with other field and/or laboratory testing. The potential for excessive long-term, creep-like, lateral displacements of soil nail walls is low in fine-grained soils with a plasticity index (PI) of less than 15.
- *Engineered fill.* Soil nails can be installed in existing engineered, structural fill if this material is a mixture of well-graded granular material (approximately 90 percent of the mix or more) and fine-grained soil with Liquid Limit (LL) and PI values of less than 40 and 20, respectively; and if they were placed with acceptable compaction methods and acceptable levels of compaction energy (at least 90 percent of Standard Proctor, ASTM D698 or AASHTO T99). In addition, the age of engineered fills is critical for suitability and stability. Young embankment fills may be problematic.

- *Residual soils.* Some 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.
- *Glacial till.* Some glacial till soils are suitable for soil nailing applications because they are often dense, well-graded granular materials, and have a relatively small fines content. However, glacial soils may also present installation difficulties as discussed in Section 2.7.4.

2.7.3 Difficult Soil Conditions for Soil Nailing

Soil conditions that are less favorable than those described above can be considered difficult or marginal. Soil nail walls have been installed in such soils successfully, but not necessarily with a consideration of being the most cost-effective option. Examples of difficult or marginal soil conditions include:

- *Non-engineered fill.* Soil nails may be installed successfully in existing non-engineered fill if this material has characteristics that are similar to those of engineered fills (see above). Newly placed embankments of non-engineered fill or fills that were placed with inadequate compaction methods or unacceptable levels of compaction energy may be problematic due to the potential of irregular nail capacity and excessive deformations.
- *Residual soils with unsuitable conditions.* Some residual soils may contain materials (e.g., mica, shale) that may impart low strength or stiffness to these materials.

2.7.4 Unfavorable Soil Conditions for Soil Nailing

Soil nail walls are generally unsuitable, or are more difficult and expensive to design and construct in unfavorable soil conditions. Unfavorable soil types and ground conditions are described below:

- *Dry, poorly graded cohesionless soils.* When cohesion is not available in poorly graded soils, vertical or nearly vertical cuts are difficult to achieve.
- *Granular soils with high groundwater*. Groundwater occurring behind the proposed soil nail wall may require drainage to stabilize the mass of soil in this location. Additionally, large amounts of groundwater can cause drill holes to collapse easily, particularly in loose granular soils; thus requiring temporary casing and increasing the cost of the installation. Excessive groundwater seeping out to the excavation face may cause significant difficulties for shotcrete application.

- *Soils with cobbles and boulders*. A significant proportion of cobbles and boulders in the soil may cause excessive difficulties for drilling and may lead to significant construction costs and delays.
- *Soft to very soft fine-grained soils.* These soils have SPT N_{60}-values less than 4 bpf. Even if these soils had N_{60}-values slightly above 4 bpf, they may be highly plastic and tend to develop low bond resistances and creep. Low bond resistances would require unreasonably long nail lengths to provide adequate pullout resistance and stability. Long-term deformations such as creep may be a concern with highly plastic clays.
- *Collapsible soils.* These soils can appear competent but can experience large and sudden volume changes after they are saturated. These changes may occur in the absence of added loads. The collapse of the internal structure of these soils can cause problems during excavation or can deteriorate the long-term bond resistance at the grout-soil interface.
- *Organic soils*. Organic silt, organic clay, and peat often exhibit very low shear resistance and thereby tend to develop low bond resistances, which may lead to uneconomical nail lengths or, in extreme cases, to an unfeasible wall design. While some organic soils can exhibit acceptable and more or less uniform shear resistances, other organic soils like fibrous peat may be highly heterogeneous and highly anisotropic. In this case, while the soil shear resistance may be reasonable along some orientations, it may be significantly lower along others. 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 or highly corrosive groundwater.* These conditions are especially detrimental for permanent applications of soil nail walls. Examples of non-natural soil-like materials with high corrosion potential include cinder and slag (i.e., both residues derived from the smelting or refining of metals).
- *Weathered rock with unfavorable weakness planes*. Weathered rock with prevalent unfavorable weakness planes such as joints, fractures, shears, faults, bedding, schistosity, or cleavage may affect the drill hole stability and make grouting difficult. The presence of discontinuities may cause the formation of potentially unstable blocks in the mass of ground retained behind the wall during excavation. If the stability of blocks is marginal, it may rapidly deteriorate after excavation due to various factors, such as presence of 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 other conventional wall types. In addition, grouting in rock with very large open joints or voids will be very difficult and/or expensive due to excessive grout loss.

- *Karst formations*. Grouting in karstic formations is not appropriate due to the potential for excessive grout loss.
- *Loess.* When dry, loess may exhibit acceptable resistances that would allow economical installation of soil nails. However, when significant amounts of water collect 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 resistance may arise for the wetted condition. In these cases, unusually long soil nail lengths may result from using conventional methods of nail installation. Regrouting (an atypical and more costly step) has been used to increase bond strengths in loess.
- *Glacial till.* Despite the generally favorable characteristics of glacial till for soil-nail wall construction described in Section 2.7, glacial till may pose difficulties in some cases. If the glacial till contains boulders, cobbles, and coarse gravel, difficulties for drilling may arise. In predominantly granular glacial till with significant preconsolidation pressures, stress relief may cause instability of the excavation face during construction. The excavation face may need to be stabilized by the introduction of vertical drilled bars – to allow excavation. Stress relief and outward movement may also develop during excavation in predominantly fine-grained, overconsolidated, glacial till. The outward movement may continue to develop over an extended period of time.
- *Expansive soils.* These soils, even if stiff, may induce localized pressure on the facing and may tend to deteriorate the bond resistance.

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 localized pressures will be applied to the temporary and permanent facings.
- Repeated freeze-and-thaw cycles in the soil retained by a soil nail wall may reduce the bond strength at the grout-ground interface of soil nails near the ground surface, and the adhesion between the shotcrete and the soil. To minimize these detrimental effects, freeze-thaw cycles should be considered in the shotcrete design.
- Granular soils that are very loose ($N_{60} \leq 4$) and loose ($4 < N_{60} \leq 10$) may undergo excessive settlement due to vibrations caused by construction equipment.
- Loose and very loose saturated granular soil can be susceptible to liquefaction in seismically exposed regions. Several ground modification techniques (with

significant associated costs) may be utilized to densify granular soils and thereby minimize these damaging effects.

- The presence of zones of weak materials contained within the natural soils behind the proposed wall may promote instability. Continuous utility trenches extending parallel to and close to the wall may contain poorly-compacted granular backfill or unsuitable fine-grained backfill.

Despite the difficulties associated with the unfavorable soil conditions described above, soil nail walls might still be constructible in these situations. It should be recognized that walls in these conditions would be more expensive to design and construct when compared to conventional walls in suitable soil types. Stricter long-term performance requirements and performance monitoring may be necessary for soil nail walls constructed in unfavorable soil conditions. It is also important to note that such ground conditions will also affect many of the alternative excavation support systems that may be considered.

2.8 Factors Affecting Costs and Construction Schedule

The following list provides some important factors that must be considered in cost comparison for soil nail walls:

- Site accessibility and right-of-way
- Presence of unfavorable soil conditions for soil nail walls
- High permanent groundwater in granular soils
- Excessive seepage on the wall facing, regardless of the groundwater source or the type of soil being retained
- Excessive amount of groundwater accumulated in the excavation and need for temporary dewatering
- Material availability, including bar size and cement type
- Cost of mobilization to the site
- Availability of specialty contractors skilled in soil nailing and shotcrete application near the project site
- Sufficient space to operate equipment including ability to have a drill platform
- Need for temporary support in exposed soil between excavation lifts, including placement of an intermediate soil berm
- Difficult conditions for advancing drill holes as a result of obstructions (e.g., utilities, large aggregate or stone particles, deep foundations behind bridge abutments, etc.)
- Need to provide in-hole soil stability such as casing to avoid drill hole collapse during drilling
- Need for ground anchors or other means to prevent lateral wall deflections adjacent to large, existing infrastructure

- Need for long-term monitoring
- Requirements for high levels of corrosion protection
- Large variations of ground conditions over small distances leading to frequent changes of drilling equipment and procedures
- Changing ground conditions requiring additional numerous verification load tests
- Special requirements for facing aesthetics, including the use of precast concrete panels or sculpted facings
- Regional conditions including high seismicity and frost susceptibility

Logistics related to other systems should also be considered when completing this comparison, such as: the ability to drill or drive soldier piles, property easements for long anchor installation, critical path schedule issues for access in front of the wall, and existing and proposed utility locations.

The construction costs of permanent soil nail walls in public transportation projects typically range from approximately $70 to $100 per square foot of wall in 2014. However, the actual construction cost may be considerably lower or higher depending on the factors listed above. When compared to typical cost ranges for other commonly used retaining structures on U.S. highway projects, soil nail walls can provide a 10 to 30 percent cost savings.

Chapter 3: Construction Materials and Methods

3.1 Introduction

This chapter presents information on materials and methods used in the construction of soil nail walls. Standard installation techniques for soil nails can be divided into two methods as dictated by the type of reinforcing bar:

- **Solid bar soil nails**: Solid bar soil nails are placed in typically 4- to 8-in. diameter drill holes that are drilled and grouted in a two-step operation. In the first step, drill holes are drilled at a shallow angle (usually 15 degrees from horizontal) using cased or open-hole techniques. In the second step, the tendons are inserted and grouted in the drill holes. Solid bar soil nails are the most commonly used soil nails. They are readily available in many tendon sizes, thread types, and steel grades, and with a variety of corrosion protection schemes to suit a variety of applications and site conditions.
- **Hollow bar soil nails (HBSNs):** These soil nails consist of hollow, steel threaded tendons that are drilled and grouted in a single operation. Grout is injected through the tendon as the drilling proceeds. The grout exits through ports that are located in a sacrificial drill bit, flushes soil cuttings out of the drill hole, and fills the annular space between the tendon and the drill hole. Rotary and sometimes rotary-percussion drilling methods are used with HBSNs. In general, HBSNs allow for faster installation than solid bar soil nails in appropriate subsurface conditions. These bars are particularly advantageous in soils where open drill holes would otherwise collapse and would require using temporary casing to maintain hole stability until grout can be injected. Corrosion protection schemes for HBSNs are limited, and standards for quality assurance and testing techniques are not fully established. More details regarding special considerations for HBSNs are discussed in Chapter 10.

3.2 Components of a Soil Nail Wall

The main components of soil nails installed using the techniques listed above are described in this section and are depicted in Figure 3.1.

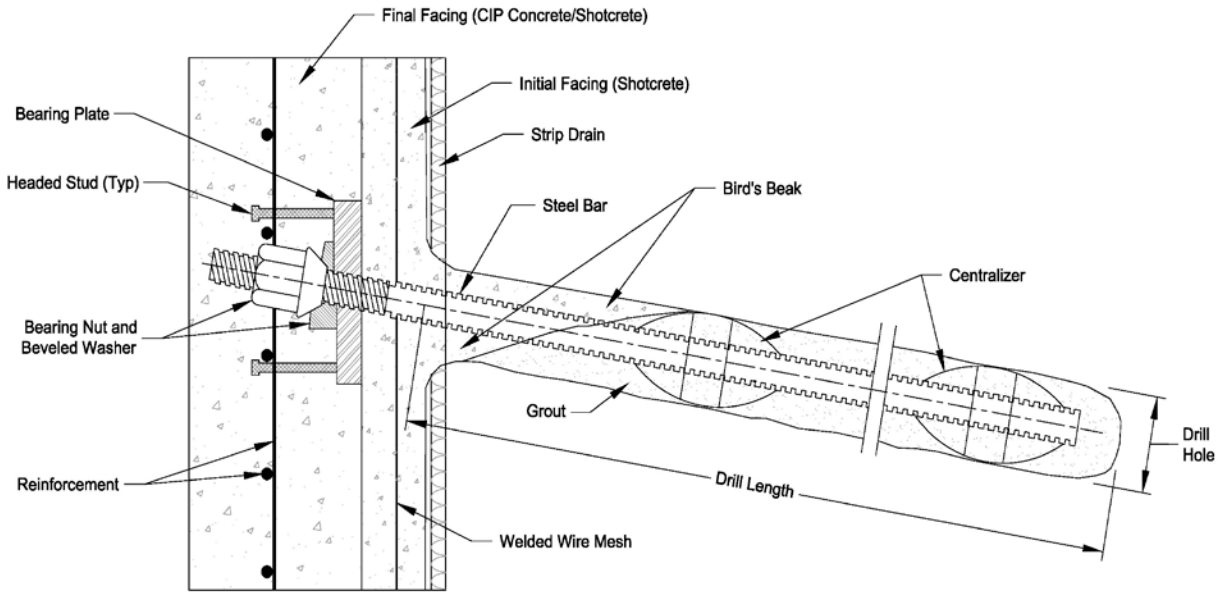


Figure 3.1: Illustration. Main components of a solid bar soil nail and associated wall facing. Modified after Porterfield et al. (1994).

3.2.1 Tendons

Tendons used for soil nails are threaded. The threads may be a continuous, spirally-deformed ribbing (continuous thread bars), or may be cut into a bare reinforcing bar. Bars must be threaded a minimum of 6 in. on the cut face to allow proper attachment of the bearing plate and nut. If threads are cut into a non-threaded bar, it is necessary to consider the reduction in steel area in the threaded portion of the bar during design. Tendons generally have a nominal tensile strength of 60 ksi (Grade 60) or 75 ksi (Grade 75). Grade 60 or 75 steel tendons should conform to ASTM A615.

Tendons with a tensile strength of 95 ksi (Grade 95), and as high as 150 ksi (Grade 150), may be considered for soil nailing. However, the use of high-grade steel is more typical with ground anchors (Sabatini et al. 1999) where the design loads are usually much greater than for soil nails and may not be an efficient choice for conventional soil nail applications. Bars with strength of 95 ksi (Grade 95) may be acceptable for soil nailing as long as the steel ductility is comparable to lower-grade steels. If Grade 150 steel bars are used, they should conform to ASTM A722.

Table A.1 of Appendix A presents section characteristics of solid bar tendons. Solid bar soil nails commonly include tendons with size designations Nos. 8, 9, 10, and 11. Bars in diameter smaller than No. 8 (up to Nos. 6 and 7) are used by some in soil nails but are often

avoided because small diameter bars may tend to bend excessively during handling and installation, and because they may result in too close, inefficient spacing of the nails. Bars No. 14 and larger may be inefficient as their maximum tensile load is limited by geotechnical pullout and strengths of the facing. However, these limitations to bar sizes do not apply in all cases.

The maximum manufactured length of threaded bars is 60 ft for No. 6 through No. 14. Tendons are generally continuous without splices or welds. Couplers can be used to extend the length of bars. They must develop the nominal tensile resistance of the bars as certified by the manufacturer. Figure 3.2 shows threaded solid bars.



Figure 3.2: Photo. Threaded solid bars. Photograph courtesy of Con-Tech Systems, Ltd.

Hollow bars for soil nailing applications range in yield tensile strength between 60 ksi and 90 ksi (410 MPa and 620 MPa in SI Units). Hollow bars range from about 1 to 3 in. (30 mm to 73 mm) in outer diameter, and 0.3 to 1 in. (7 to 25 mm) in wall thickness. Tendons are available in 10- and 20-ft lengths and can be cut and coupled to custom lengths. Tables A.2 through A.4 (Appendix A) present section properties of commonly available hollow bar tendons.

3.2.2 Connection Components

The steel components that connect soil nails to the facing consist of bearing plates, beveled washers, hexagonal nuts, washers and headed studs (Figure 3.3). The bearing plate, hex nuts, and washers provide connection between the nail and the initial facing, while the headed studs connect the nail end and the final facing. The purpose of the bearing plate is to distribute the force applied at the nail end onto the initial shotcrete facing and the soil behind the facing. The bearing plate is commonly Grade 36 (AASHTO M183/ASTM A36) or

Grade 50 steel, and is usually square and flat, with 8- to 10-in. side dimensions and typical thicknesses of 0.75 to 1 in. Some soil nail bar manufacturers supply forged bearing plates that are concave at the drill hole, thus allowing some articulation of the connection in any direction. Most Contractors place the bearing plate against the still-fresh shotcrete of the initial facing. In this case, the bearing plate must be well-seated but not excessively embedded into the grout (Figure 3.3).



Figure 3.3: Photo. Threaded solid tendon with steel plate, washer, and nut. Note headed studs for connection to final facing. Photograph courtesy of DYWIDAG-Systems International.

Beveled washers with an angle matching the inclination of the nail are placed between the bearing plate and the nut to provide uniform bearing. Alternatively, spherical seat nuts are often used to facilitate alignment of the connection components. Nuts are tightened slightly with a hand-wrench. After the shotcrete is set, care must be exercised to avoid over-tightening the nuts. Washers and nuts must meet the requirements of AASHTO M291/ASTM A563 Grade B.

The connection to the final facing typically consists of four headed studs that are welded to the bearing plate and embedded into the final facing. The welding must be performed according to the Structural Welding Code (ANSI/AWS 2000). In general, the plate should be

sized such that studs are welded no closer to a distance to any outer edge of the plate. This distance depends on the diameter of the stud shaft and is commonly between 1 and 1.5 in.

3.2.3 Grout

Grout, which fills the annular space between the tendon and the soil in the drill hole, is most often a neat cement grout. The minimum compressive strength for grout should be 1,500 pounds per square inch (psi) at 3 days, and 3,000 psi to 4,000 psi at 28 days, as tested in accordance with AASHTO T106/ASTM C109. If sand is used in the grout mixture, it must meet the requirements of AASHTO M6/ASTM C33. A typical grout plant setup is depicted in Figure 3.4.

Cement Types I, II, III, V, or Type I/II conforming to AASHTO M85/ASTM C150 can be used in the grout mix. Cement Type I (common or general purpose) is recommended for most applications. Cement Type II (moderate sulfate resistance) hardens at a slower rate, produces less heat, and is more resistant to the corrosive action of sulfates than Cement Type I. Cement Type III (high early strength) is ground finer, hardens faster, and can be used when a target grout strength is required to be achieved faster than for typical project conditions. Type V (high sulfate resistance) may be used in areas where ground conditions are corrosive as this material is more sulfate resistant than other cements.

The water/cement ratio for grout used in soil nailing applications ranges from 0.4 to 0.5 in neat cement mixes, which is equivalent to a specific gravity of 1.8 to 1.9. Specific gravity measurements of neat cement grout mixes are performed during grout preparation and produce immediate results (see Figure 3.5). They can be used as a primary quality control of the neat cement grout mix if there is proof that the materials and mix design consistently produce a grout of the minimum specified strength. This can be addressed by pre-production compression tests on grout cubes prepared with a mix of known specific gravity. Neat cement grout cubes may still be molded and tested periodically as needed as verification of grout compressive strength (see Figure 3.6).

The need for a stiffer grout may arise when it is desired to control leakage of grout into highly-permeable granular soils or highly fractured rock. In such cases, sand/cement grout with lower flowability may be used. However, using grout that is too stiff will make it unpumpable with conventional grouting equipment, and tendon installation will be difficult or impossible. Therefore, when a thicker grout is needed to control grout take in the soil, it may be necessary to pre-treat the ground before soil nail installation. Specific gravity measurements are not suitable for quality control of a sand/cement grout mix. In some granular soils with an open matrix with no cohesion (e.g., poorly graded gravel), the potential for drill hole collapse or grout leakage may be large, even if stiff grout is used. In this case, the use of a grout "sock" may be necessary to prevent the collapse of the drill hole and to

reduce grout flow into the highly-permeable soil. A grout sock consists of a geotextile sheath that is placed around the steel bar and the tremie pipe before the entire assemblage is placed in the drilled hole.

Admixtures are not typically required for most applications. However, admixtures that control bleed, improve flowability, reduce water content, retard set, and enhance workability may be used in the grout. Admixtures must meet the requirements of AASHTO M194/ ASTM C494, must be compatible with the grout, and mixed in accordance with the manufacturer's recommendations. Accelerators are generally not permitted. The use of accelerators containing calcium chloride must be avoided in grout because the corrosion potential around the tendon can be drastically increased. Expansive admixtures should not be permitted except where the grout is used as part of corrosion protection encapsulation as discussed in Chapter 7.

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. The improved workability of grout due to plasticizers can be extended commonly up to about 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 or physical corrosion protection provided by cement. Therefore, the use of air-entrainment agents should be approved only when corrosion protection methods other than grout cover are present, or when the thickness of the grout cover is larger than the minimum values included in Chapter 7. Some proprietary grouts contain chemicals that limit 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 are not adversely affected.

When solid bars are used, grout is pumped shortly after the tendon is placed in the drill hole to reduce the potential for squeezing or caving of the hole. For these applications, the grout is injected by tremie methods through a grout pipe inserted to the bottom of the drill hole until the grout fills the hole. Due to the fluid nature of the grout and the inclination of the drill hole, the fresh grout cannot fill the space above the bottom elevation of the drill hole opening. The space, called a "bird's beak" due to its shape, is filled up with additional grout after a temporary cover is placed in front of the drill hole, or, more commonly, filled with shotcrete either by hand-packing or during the shotcrete facing placement. Because this area is the most vulnerable to water infiltration and therefore corrosion, it is critical that it be subsequently filled to ensure complete coverage and protection from moisture.

Grout pipes typically consist of heavy-duty plastic tubing ranging from approximately 0.75- to 1.0-in. outer diameter. The grout tube is commonly removed after grout placement.

However, some contractors leave the grout tube embedded without producing any significant detrimental effect. If the grout tube is not removed, it should be plugged at its proximal end. If the tubes left in are not plugged, the effect the tube may have on the performance and durability of the soil nail must be considered. Grout injection must be conducted smoothly and continuously to ensure that the space between the drill hole and the tendon is filled completely, leaving no voids or gaps. The bottom of the grout tube must remain below the grout surface at all times while grout is being pumped into the drill hole. Given the fluidity of the grout, often the entire hole is filled prior to beginning to withdraw the grout tube. Details on grouting of HBSNs are provided in Chapter 10.



Figure 3.4: Photo. Typical grout plant setup.
Photograph courtesy of Barry Siel, FHWA.



Figure 3.5: Photo. Baroid mud balance to measure specific gravity. Photograph courtesy of Schnabel Engineering.



Figure 3.6: Photo. Molding grout cubes for strength verification. Photograph courtesy of Schnabel Engineering.

3.2.4 Centralizers

Centralizers are devices made of Schedule 20 or 40 polyvinyl chloride (PVC) or other synthetic materials not detrimental to the tendon (Figure 3.8). Steel centralizers are used with HBSNs (Figure 3.7). Centralizers are installed at various locations along the length of each solid bar or HBSN to ensure that a minimum thickness of grout completely covers the tendon. They are installed at regular intervals, not exceeding 10 ft along the length of the nail and at a distance of about 1.5 ft from each end of the nail. Centralizers must be securely attached to the tendons and must be sized to allow: (i) positioning the tendon within 1 in. of the center of the drill-hole; (ii) inserting the tremie pipe to the bottom of the drill hole; and (iii) grout to freely flow up the drill hole.



Figure 3.7: Photo. Typical steel centralizers attached to HBSNs. Photograph courtesy of Schnabel Engineering.



Figure 3.8: Photo. Typical PVC centralizers attached to a solid bar. (The spacing between centralizers depicted is for illustration only; wider spacing is used in practice.) Photograph courtesy of Williams Form Engineering Corp.

3.2.5 Corrosion Protection Elements

In addition to the cement grout, which provides both physical and chemical protection to the tendons, other devices are used to provide additional corrosion protection, as necessary. The solid bar tendon can be protected by encapsulation in a sheath of corrugated high-density polyethylene (HDPE) conforming to AASHTO M252, or of corrugated PVC tube conforming to ASTM D1784, Class 13464-B. After the bar is installed in the sheathing, the annulus between the sheathing and the tendon is filled with grout prior to leaving the manufacturer. Ultimately, once the encapsulated bar is placed in the drill hole, the annulus between the sheath and the drill hole is grouted using tremie methods as described previously.

Fusion-bonded epoxy coating can be applied to solid bars. The epoxy coating is a dielectric material that impedes the flow of electric currents that favor corrosion. Fusion-bonded epoxy coating must meet the requirements of ASTM A775. There are several different types and thicknesses of epoxy coating for bars. The color of the coatings is green, gray or purple. The green epoxy coating is flexible and suitable for steel reinforcement that will be bent in different shapes. The gray and purple epoxy coatings are less flexible than the green epoxy coating but have greater chemical resistance. The purple epoxy coating is better suited for marine or harsh environments.

Testing has been conducted on resistance to abrasion and performance of the coating by various manufacturers; however, there is not sufficient data available in the technical literature to provide a direct correlation between the numerical results of resistance to abrasion tests and the actual performance of the protective coating in the field.

Solid and hollow tendons and their hardware can also be hot-dip galvanized. In the case of HBSNs, however, because drilling is performed with the hollow bars themselves, it is likely that these coatings, as well as epoxy coatings, are damaged during installation (Cadden et al. 2010; Samtani and Nowatzki 2010).

The simplest form of corrosion protection is consideration of sacrificial steel. For non-aggressive soils, a nominal thickness, depending on the design life of the element, is subtracted from the bar diameter and used for bar area-based calculations of the steel structural resistance. A more detailed description of methods of corrosion protection as well as guidelines for their selection is presented in Chapter 7. Chapter 10 discusses HBSNs and corrosion protection.



Figure 3.9: Photo. Solid tendons with gray and purple epoxy coating (left) and partially encapsulated by corrugated sheathing (right). Photographs courtesy of DYWIDAG-Systems International.

3.2.6 Wall Facing

Tendons are connected to a facing system at the excavation face or slope surface. Facings most commonly consist of an initial facing of shotcrete and a final facing of shotcrete or CIP concrete. The purpose of the initial facing is to support the exposed soil between the nails during excavation and nail installation, provide initial connection among nails, and furnish protection against erosion and sloughing of the soil at the excavation face. The final facing performs the same functions as the initial facing and provides the opportunity to meet aesthetic requirements of the project. The final facing design must also consider long-term corrosion and climate variations as needed.

Initial facing most commonly consists of shotcrete, WWM and short reinforcement bars called waler bars and vertical bars around the nail heads. Final facing is commonly constructed of CIP-reinforced concrete or WWM-reinforced shotcrete. WWM is manufactured in Grades 60, 65, 70, 75, and Grade 80. The grade refers to the minimum yield strength (f_y in ksi) of the WWM wires. Both plain and deformed wires are available (see WRI 2006). WWM must meet the requirements of AASHTO M55/ASTM A1064 or ASTM 1064M. Reinforcing steel must meet the requirements of AASHTO M31/ASTM A615, Grade 420, deformed. Precast concrete panels may also be used to construct the final facing, especially for projects with special aesthetic requirements. Typical sizes and properties of WWM are presented in Table A.5 (Appendix A).

Shotcrete, which comes out of the feed hose at high velocity, is sprayed pneumatically onto the exposed soil surface. Shotcrete must be mortar or small-aggregate concrete. As the

material is applied, the impact energy consolidates the concrete. Shotcrete can be applied using a wet or dry process (PCA 2013). To verify the strength and consistency of shotcrete, test panels are prepared, cored, and tested for compressive strength (Figures 3.10 and 3.11) according to ASTM C1604. After shotcrete is applied, polyethylene film meeting the requirements of AASHTO M171 can be used to control moisture loss.



Figure 3.10: Photo. Curing of a shotcrete test panel on site. Photograph courtesy of Schnabel Engineering.



Figure 3.11: Photo. Shotcrete test panel cored for compressive testing of recovered

specimens. Photograph courtesy of Schnabel Engineering.

3.2.7 Drainage

To limit water pressure development behind the wall facing, vertical geocomposite strip drains (referred herein as strip drains) are installed behind the initial facing and adjacent to the excavation face (Figure 3.12). Geocomposite strip drains consist of a drainage core and a filtration geotextile attached to or encapsulating the core. The drainage core is manufactured from synthetic polymers composed of polypropylene, polyester, polyamine, PVC, polyolefin, or polystyrene. The core should have a minimum compressive strength of 40 psi when tested in accordance with ASTM D1621 Procedure A. In general, the drainage core with the geotextile fully encapsulating the core must have a minimum flow rate of 0.1 gallons per second per foot of strip width, when tested in accordance with ASTM D4716.

The strip drains are fitted with drainage elements that allow exit of the water from the strips to the outside of the wall, as depicted in Figure 3.12. Strip drains are also fitted to discharge into a pipe drain running along the base of the wall and/or through weepholes discharging through the facing and to the toe of the wall.

Most drainage elements are made of PVC and consist of a snap-on cap or grate, and an exit PVC pipe connected to the underdrain system. Generally the underdrain system consists of a drainage pipe embedded in an aggregate-filled trench, which runs along the bottom of the excavation. The underdrain system pipe, as well as the pipe fittings, should meet the requirements of Schedule 40 PVC solid and perforated wall. The drainage pipe is slotted PVC or polyethylene (PE) surrounded by clean, coarse-gravel sized aggregate that is free of fine particles.

The aggregate trench must be wrapped with geotextile filter fabric as the use of a geotextile sock directly over the slotted pipe must be avoided. The geotextile around the aggregate has the function of preventing the removal of fines by piping from the natural soil surrounding the aggregate trench. To prevent clogging, the geotextile should meet filter criteria (Holtz et al. 2008) and be selected from a range of available commercial products based on the particle size distribution of the soil being retained. The aggregate around the pipe has the function of collecting groundwater along the pipe and into the slots as needed, providing a redundant pathway to groundwater.

When the groundwater behind the proposed soil nail wall is abundant, horizontal pipe drains extending into the retained soils are necessary. However, if there are large volumes of groundwater or the groundwater is high, the applicability of soil nails versus other technologies should be carefully evaluated further during the feasibility study. Great care

should be exercised in the design and construction of a soil nail wall if significant seepage or high groundwater levels exist behind the planned wall. Additional aspects of the design of drainage systems are presented in Chapter 6.



Figure 3.12: Photo. Strip drain prior to initial shotcrete placement. Photograph courtesy of Schnabel Engineering.

3.3 Construction Methods

3.3.1 Introduction

The sequence of construction for soil nail walls was introduced in Chapter 2 and includes the following major steps:

1. Excavation

2. Drilling of nail holes
3. A) Nail installation and grouting

   B) Installation of strip drain
4. Construction of initial shotcrete facing
5. Construction of subsequent levels (Repeat Steps 1-4)
6. Construction of final facing

Detailed descriptions of these major construction activities are presented below.

3.3.2 Excavation

Prior to any excavation, surface water controls should be installed along the top of the wall to prevent surface water from flowing into the excavation, as this will adversely affect construction and promote instability of the excavated face. Collector trenches behind the limits of the excavation are used to intercept and divert surface water. Subsequently, soil excavation is performed using conventional mass excavation or earth-moving equipment from a platform, and final trimming of the excavation face can be carried out using a backhoe or excavator from this platform. The initial cut depth is often 3- to 5-ft high. 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 30-ft wide is required to accommodate the conventional drilling equipment used for nail installation. Small track drills can work on benches as narrow as 15 ft and with headroom clearance as low as 8 to 10 ft.

In many instances, the most critical situation related to wall stability occurs during construction. For the lower excavation lifts, the lowest margins of safety might occur when the open cut is temporarily unsupported; that is, before nails and shotcrete are placed at these levels. It is important to watch for early signs of instability during construction, such as bulging, sloughing, and excessive deformation of the excavated soil face. If early signs of wall instability are observed, it is critical to backfill the exposed face quickly with a temporary berm. The exposed length of an excavation should be limited to that which can be stabilized and covered with shotcrete during a single working shift. Figure 3.13 shows the excavation of soil below the already completed rows of soil nails.



Figure 3.13: Photo. Excavation of soil lift during construction of soil nail wall. Photograph courtesy of Moretrench American Corporation.

For cases where the excavated face cannot stand unsupported for the required period of time, a continuous berm may be left during excavation to provide support to the cut face (Figure 3.14). 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 shortly after.

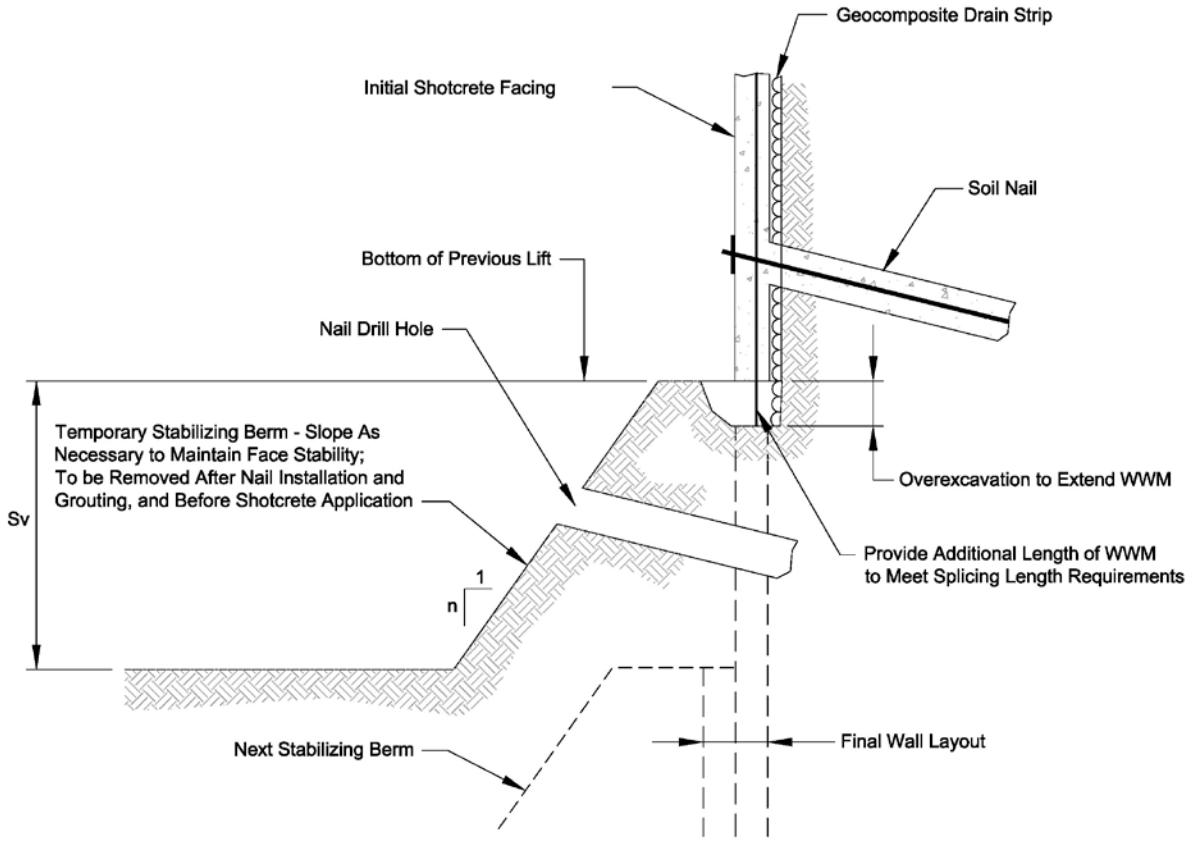


Figure 3.14: Illustration. Soil nail support of excavation with stabilizing berm. Modified after Porterfield et al. (1994).

Alternatively, the stabilizing berm can be removed in a staggered sequence in which alternating excavation slots are excavated and soil nails are installed in each open slot. 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 remaining berm sections are removed, the soil nails are installed, and shotcrete is applied between the previously installed nails.

The use of stabilizing berms will have a significant impact on construction costs and production schedule. If temporary excavation stand-up time is a major concern and represents a potential risk, alternative top-down construction methods such as soldier piles and ground anchors should be considered. In addition, ground improvement methods such as permeation grouting have been used and documented for improving stand-up time of unstable granular soils (LaRue 2010) and minimizing impacts on construction cost and schedule.

As an alternative to temporary berms, vertical grouted bars can be installed with a horizontal spacing of 1 to 3 ft along the base of the wall to provide temporary face stability. Soil nail spacing should be smaller for soils that are more prone to face instability. These grouted bars should only be installed where loose deposits are encountered. If the design incorporates a batter wall, the design can be modified to incorporate a stepped wall where loose deposits are encountered with the grouted bars being installed at this location.

While the overall cost of adding vertical grouted bars may be higher than the creation of temporary berms, this system may be advantageous regarding project schedule. Some contractors install vertical grout columns (without a bar) adjacent to the excavation face to limit overbreak, provide a neat excavation line, and prevent ground loss during excavation.

3.3.3 Drilling of Nail Holes

Nail holes are drilled using one of several available drilling methods, including rotary, percussion, auger, and rotary-percussion drilling. Table 3.1 (a, b, c) presents a summary of the most common 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. Figure 3.15 shows a drill hole being advanced using the external flush open hole method.

Table 3.1a: Hydraulic Rotary Auger Methods for Drilling Competent Soils or Weathered Rock (Modified from Elias and Juran 1991)

Drilling Method	Cased?	Cuttings Removal Method
Lead Flight Kelley-Bar Driven	No	Mechanical
Sectional Solid-Stem	No	Mechanical
Sectional Hollow-Stem	Yes	Mechanical (air support)
Continuous Flight Solid-Stem	No	Mechanical
Continuous Flight Hollow-Stem	Yes	Mechanical (air support)

Available drill hole sizes: 4-12 in.

Table 3.1b: Hydraulic Air and Water Rotary Methods for Drilling Competent Soils, Rock, or Mixed Ground Conditions (Pneumatic Down the Hole Hammer Available) (Modified from Elias and Juran 1991)

Drilling Method	Cased?	Cuttings Removal Method
External Flush Open Hole	No	Water
External Flush Cased	Yes	Water
Duplex	Yes	Compressed Air or Water

Available drill hole sizes: 4-8 in. (air rotary) and 4-12 in. (hollow stem augers).

Table 3.1c: Pneumatic Rotary (Top Hole Hammer) Methods for Drilling Non-Caving Competent Soils or Rock (Modified from Elias and Juran 1991)

Drilling Method	Cased?	Cuttings Removal Method
External Flush Open Hole	No	Compressed Air

Available drill hole sizes: 4-8 in., typically <5 in. Used for non-caving ground.



Figure 3.15: Photo. Drilling nail holes. Photograph courtesy of Barry Siel, FHWA.

The selection of the drilling method may also be controlled by the local availability of equipment and the specific ground conditions to be encountered. The selected drilling method should not cause excessive ground mining or 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 casings or HBSNs in unstable ground, or by limiting the air flush so as to ensure that the drill hole remains open without building pressure. Soil nail wall contract documents commonly 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.

Cased drilling includes the use of single tube and the duplex rotary methods. The single tube method involves drilling with the drill string and flushing the cuttings outside the tube by air, water, or a combination of air and water. 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 7 in. Casing the drill hole will increase the cost of soil nail walls, sometimes significantly.

3.3.4 Nail Installation and Grouting

After the tendon is inserted in the drill hole (Figure 3.16), the drill hole is filled with grout using a tremie pipe. The grout pipe is inserted to the bottom of the drill hole and the grout is injected until it fills the hole. As the grout sets, it bonds to the tendon and the surrounding ground.

The practice of grouting by gravity provides bond strengths that are often sufficient for soil nailing to be feasible and cost-effective. However, in cases where poor soil conditions are encountered, higher bond strengths may be required to keep soil nail lengths reasonable. High bond resistance may be achieved in granular soils and weak fissured rock by injecting grout under relatively low pressures through the casing. These increases in bond strength, when compared to strengths obtained with gravity grouting, are the result of an increase in confining pressures around the grout, the compaction of soils surrounding the grout, increased interlocking between the soil and the grout, and an increase in the grout effective diameter. Pressure grouting through the casing is seldom used for fine-grained soils because only minor increases in bond resistance are achieved, especially if the soils are saturated.



Figure 3.16: Photo. Solid nail tendon installation. Photograph courtesy of Schnabel Engineering.

There are instances when the design bond resistance cannot be achieved if appropriate installation procedures are not followed. For example, poor removal of cuttings from the drill hole may cause significant reduction of the bond resistance. In the case of clayey soils with moderate plasticity, excessive mechanical cleaning of the drill hole may remold the clay and decrease the bond resistance to values much lower than those expected for undisturbed conditions. More details on design bond strength of soil nails can be found in Chapter 4.

3.3.5 Drainage Placement

Strip drains are placed against the excavation face with the geotextile filter side against the soil. If the strip drain is packaged in rolls, the rolls must be protected while shotcrete is applied to each lift and then rolled down to be continuous. If the strip drains are panel-type, they must be spliced at the bottom of each excavation lift and generally should have a minimum 12-in. overlap of both the core and the geotextile filter such that the water flow and filtration are not impeded. The bottom drainage cap and exit pipe are connected to the strip drains and affixed to the reinforcing steel of the facing. During shotcrete placement, the exit pipe must be adequately covered to prevent penetration of shotcrete into the system. Once the shotcrete operation is complete, the exit pipe can be connected to the underdrain system, if applicable.

The underdrain system is constructed inside a trench excavated at the toe of the wall. The aggregate-filled trench is wrapped with geotextile filter fabric. The geotextile is placed in the excavated trench, then the clean aggregate and drainage pipes are placed, and the geotextile is wrapped across the top of the trench. The use of a geotextile-wrapped pipe is generally not recommended. Continuity of the drainage path is the key to proper construction of the drain system in the following locations: between the drainage core joints, between the drainage core and the weephole outlets, between the wall outlet and drain pipe, and between the drain pipe and the drain outlet.

3.3.6 Initial Facing Construction

3.3.6a General

The initial facing for soil nail wall applications is constructed using shotcrete, with a thickness most commonly between 3 in. and 4 in. However, thicker initial facings may be required in certain special applications. Shotcrete provides a continuous layer over the excavated face and fills irregularities. Shotcrete applications have been completed using WWM or fiber reinforcement and bars. The initial shotcrete covers the exposed soil with the exception of a narrow band at the bottom of the lift. This band remains uncovered to expose the lowest portion of the WWM and strip drains, and allows for the overlapping of these elements in the next excavation lift. A shotcrete facing for a wall under construction is shown in Figure 3.17.



Figure 3.17: Photo. Initial shotcrete application. Photograph courtesy of Moretrench American Corporation.

3.3.6b Shotcrete Application

Two opposing requirements exist for shotcrete mixtures: "shootability" and "pumpability." Shootability is the ability of a shotcrete 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. High flow-resistance and high viscosity are ideal for shooting. Low flow resistance and low viscosity are ideal for pumping. Once it is applied, a shotcrete mix with high flow-resistance and high viscosity will tend to stick, 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 up to a thickness of typically 6 to 8 in. is achieved without producing sloughing or sag cracks below the rebar. While thicker shotcrete sections are possible, these may be achieved without significant effort or without the use of specialized forms. However, because thinner sections are more common for initial facing, shootability is not expected to be an issue for initial facings. Methods for shotcrete mix design can be found in existing literature (ACI 2014).

Two types of shotcrete methods are 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 most common admixtures for shotcrete construction on a vertical surface are accelerators. 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 supplied with the wet method is applied by compressed air at higher velocities than that of the dry method.

Wet mix is often preferred for the construction of shotcrete facing walls because:

- Wet mix yields somewhat higher production of fresh shotcrete, typically 2.2 to 3.0 yd^3/hour versus 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 often 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.
- Supply of ready-mix shotcrete from commercial batch plants is readily available.

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. Water/cement ratios

less than 0.45 tend to result in relatively high strength, high durability, and low permeability as long as the shotcrete is properly compacted in situ by elimination of entrapped air.

Two of the most significant factors determining shotcrete quality and durability are cement content and in situ density. Shotcrete mixtures have relatively high contents of cement, from approximately 600 to 700 pounds/yd^3 of mixture. The amount of cement determines both the pumpability and shootability of the mixture. The in situ density of the mixture can be controlled by the amount of air that enters the mixture. Increasing the air content will decrease strength (a 5 percent reduction in shotcrete compressive strength for approximately 1 percent of air). Requirements of durability for initial facing may be relaxed in relation to those for final facing. However, in most cases, requirements are similar.

Steel fiber reinforcement is often added to shotcrete as part of a wet mix to increase ductility, toughness, and impact resistance. Fibers tend to reduce the brittleness of the shotcrete and thereby reduce crack propagation. They produce only a modest increase in flexural strength for conventional dosing rates.

Tolerances commonly used for formed CIP concrete wall facings cannot be easily achieved in shotcrete and, therefore, are not appropriate to specify. Suggested tolerances for initial shotcrete facings should be on the order of 1.5 to 2 times the tolerances normally specified for CIP concrete.

The initial facing shotcrete does not generally require vertical/horizontal contraction or construction joints. The practical limitation for the spacing of vertical construction joints in the initial facing is dictated by the length of exposed wall selected by the Contractor any given day of construction. The spacing of horizontal construction joints should be approximately the height of the exposed lift.

The Contractor must make provisions to avoid creating weak areas where shotcrete is stopped at the end of a shift. The shotcrete should be first tapered at the joint using shovels or other means, from the full thickness to an edge with a thickness up to approximately 1 in. Joints must be cleaned with air under pressure and then wetted before the next section of shotcrete is applied.

3.3.6c Shotcrete Reinforcement

WWM, which is sold in panels or sheets, is commonly used as reinforcement for the initial facing. Occasionally, it may also be used for final facing. The cross-sectional area and mesh opening of the WWM are selected to satisfy structural requirements including flexural and punching shear capacities and constructability constraints. Common properties for WWM are summarized in Appendix A.

The selected WWM panel must have a width that is consistent with the excavated lift height (equivalent to the vertical nail spacing), plus an overlap of at least one full mesh cell. WWM is available in the U.S. in 4- to 8-ft wide sheets and rolls. Sheets can be obtained in 12.5-, 15-, 20-, and 25-ft lengths. On request, WWM sheets can be manufactured up to 40 ft in length. In addition, the dimensions of the WWM panels must be selected to comply with ACI/AASHTO provisions for lap splice development.

The WWM wire diameter and mesh opening dimensions are evaluated as part of the soil nail wall design. For remote project locations, the design engineer should check the availability of different WWM widths near the site. In some cases when the availability of WWM widths is restricted locally, the design engineer may decide to modify the vertical nail spacing size as a function of the available commercial WWM widths.

Additional reinforcement, including walers and vertical bars, is placed around nail heads to provide additional flexural resistance at these locations. In general, two horizontal walers and two vertical bars are installed: one on each side of the nail. Characteristics of steel rebar are presented in Appendix A. Figure 3.18 shows WWM and waler bars surrounding an installed soil nail. Note that the PVC pipe shown in Figure 3.18 was used as a temporary measure to avoid shotcreting over the nail head.



Figure 3.18: Photo. Shotcrete reinforcement including WWM and waler bars. Photograph courtesy of Ryan R. Berg & Associates, Inc.

3.3.7 Final Facing Construction

3.3.7a Reinforced Shotcrete

The total thickness of a reinforced shotcrete final facing is often between 6 and 12 in., excluding the thickness of the initial facing. Headed studs welded to the bearing plates are used to connect the final facing to the soil nails. The thickness of the final facing is created by applying successive layers of shotcrete. The final facing is usually, but not always, applied after the deepest lift of the initial facing is completed, and is advanced in phases from the bottom up. The full final facing thickness can also be constructed as excavation progresses, but care must be taken to safely support the significant weight of the facing during subsequent excavation lifts.

The shotcrete facing is reinforced with WWM or rebar. In some situations, the available sections of WWM may not be sufficient to provide an efficient design. Final facings consisting of reinforced shotcrete usually do not require the use of waler bars.

Because the nail heads provide fixed points approximately every 5 ft that tend to resist lateral movements due to contraction/expansion, the requirement for spacing vertical contraction joints in final facings should be less restrictive than for other wall types. The maximum spacing of vertical contraction joints in permanent shotcrete facing is typically between 45 to 90 ft. The spacing of vertical joints must be adjusted to avoid placing joints over soil nails.

Some soil nail walls may have special aesthetic requirements. If the uneven finish and color of final shotcrete facing do not meet these requirements, other details can be specified. Wood, steel, or rubber tools can be used on the as-shot surface to smooth the shotcrete, and to add grooves to create a more aesthetic finish. Finished shotcrete surfaces can also be colored with pigmented sealers. In areas where wall aesthetics should match nearby rock outcroppings, the final shotcrete may be thickened, sculpted, and stained for a natural rock look. The final result is an irregular shotcrete surface that matches the surrounding landscape. See Section 3.5 for additional discussion and example photos of sculpted faces.

3.3.7b Cast-in-Place Reinforced Concrete

CIP concrete used as the final facing is routinely reinforced with vertical and horizontal rebar and supplementary, short bars placed around the nail head. A variety of finishes can be implemented by using commercially available form liners; an example is shown in Figure 3.19. It is generally acceptable to use one-sided formwork tied to the soil nails for support as long as this does not compromise the performance of the facing or the corrosion protection of the nail. When compared to a shotcrete facing, a CIP reinforced concrete finished facing

may have potential advantages, including the use and aesthetics of form liners, a uniform appearance, and homogeneous quality.



Figure 3.19: Photo. Cast-in-place facing. Photograph courtesy of DYWIDAG-Systems International.

The maximum spacing of vertical contraction joints in permanent CIP facing is typically 90 to 100 ft, similar to that of shotcrete final facings.

3.3.7c Precast Concrete Facing

Precast concrete facing has been used in permanent applications to meet aesthetic and durability criteria. A project using this method is shown in Figure 3.20. Precast facings also provide a means of integrating a continuous drainage blanket behind the final facing and/or a frost protection barrier in cold climates.

The panels are either small segmental (5 ft by 5 ft) elements or full-height tilt-up elements. Panels may be structural and carry the nail head load, or be non-structural. If structural, the nail plate may be provided with stud connectors or hoop-shaped rebar positioned at the back of the precast panels. The space between them is then filled with low-strength concrete to form the connection. Patents may exist over some connection details developed by various specialty contractors. If non-structural elements are used for the final facing, the initial facing requires adequate strengthening to resist the design loads.



Figure 3.20: Photo. Precast panel facing. Photograph courtesy of Texas DOT.

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

3.4 Alternate Materials

3.4.1 Sculpted Wall Facing

As an alternative to traditional final facing options, sculpted wall facing provides a highly aesthetic end product. This CIP concrete wall is hand-sculpted to resemble stone or brick and often stained to match the surrounding environment. The facing adds construction cost but offers Owners additional aesthetic options. Figure 3.21 shows an example of sculpted wall facing.

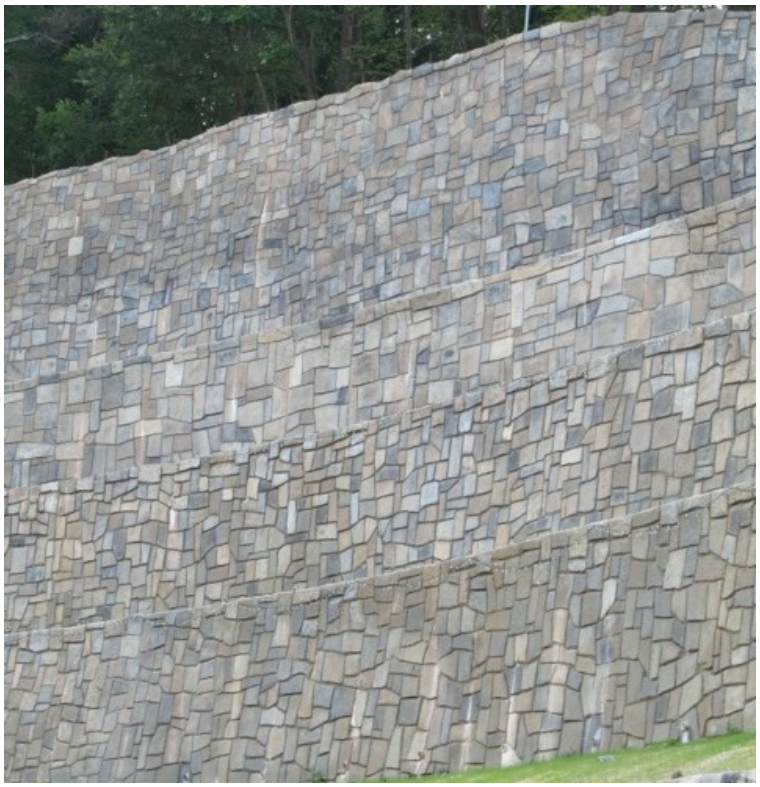


Figure 3.21: Photo. Sculpted wall facing. Photography courtesy of The Collin Group, Ltd.



Figure 3.22: Photo. Sculpted natural rock-like wall facing. Photograph courtesy of Con-Tech Systems, Ltd.

3.4.2 Drilled Drainage with Hollow Bar Soil Nails

A grout specifically designed to act as a filter material (i.e., consisting of a highly-permeable mix of cement and aggregate) can be installed as part of an HBSN. This special grout would allow an HBSN to also act as a drilled (sub) horizontal drain. This system can be used to relieve hydrostatic pressure from retaining walls and provide more efficient drainage than weepholes at the front of the wall. However, if the installation of deep drains behind the wall face needs to be considered, a soil nail wall should be carefully evaluated as it may not be the most appropriate alternative for permanent earth retention.

Drilled drains must intercept groundwater seepage well before it approaches the wall. In addition, the use of drilled drains helps to reduce pore water pressures and to avoid ice pressure near the facing during periods of freezing. Drilled drains are typically installed in a single operation, without casing, by means of rotary percussion drilling.

Chapter 4: Information Required for Design

4.1 Introduction

This chapter summarizes the information that is necessary for the design of soil nail walls. The information described in this chapter includes initial information, information from field investigations and laboratory testing, and engineering properties for design. In addition, this chapter provides guidance for obtaining data related to subsurface corrosion potential, frost effects, and site seismic characterization.

4.2 Project-Specific Requirements and Initial Information

4.2.1 Project-Specific Requirements

Engineers must become familiar with specific requirements of the work that would affect the design, construction, and performance of a soil nail wall. These project-specific requirements may include, but are not limited to, the following:

- Preliminary site development plans indicating height, length and location of the wall(s) and related to new infrastructure
- Physical constraints (wall near a bridge abutment, wall constructed as a cut in steep terrain, wall near a waterway or subject to scour)
- Potential effects from wall construction and use on existing and/or future, adjacent structures that may be sensitive to wall movement (bridge abutment, building, etc.)
- Accessibility to and ROW at project site
- Overhead and lateral limitations
- Presence of existing or new utilities in front, under, and behind the proposed walls
- Aesthetics of the wall finish
- Need for partial and/or full traffic closure during construction
- Availability of staging areas during construction

While the above requirements are not unique to soil nail walls, they must be carefully considered during the initial stage of their design. A field reconnaissance is highly recommended to help ascertain some of the above-listed conditions.

4.2.2 Initial Information

The engineer must also review key information related to the site, define design goals, and plan an effective site investigation. The review must focus on aspects such as site topography, historic land use, local geologic information (to be obtained from geologic maps and other sources), and other relevant geologic data (if applicable, including landslide maps

and fault maps). The review of air photographs from the United States Geological Survey (USGS) or other sources may be helpful where geologic features, such as landslides, may affect the planned structure. The review of information such as frost depth penetration or seismic hazard maps will also be needed in regions subject to freezing or exposed to seismic hazard.

Reviewing existing geotechnical reports and data, when available, is a key aspect of the evaluation of initial information. Historic geotechnical information is generally helpful; however, the engineer must evaluate the quality, completeness, and applicability of existing geotechnical information to a new project. The engineer must exercise caution when extrapolating subsurface conditions encountered in nearby areas to the project site because soil variability may be large in some cases.

The engineer must also review records of design and construction, as well as as-built documentation of adjacent structures that may be affected by soil nail construction and use. The designer is expected to assess the magnitude of allowable movement of adjacent structures based on the type of structure, and to evaluate geotechnical monitoring data, if available.

The engineer must also determine the magnitude of the temporary or permanent surcharge that may occur near the planned walls. Surcharge may include vehicular or construction traffic, loads from temporary stockpiles, loads from existing or future structures, utilities, etc. Chapter 5 contains a discussion about the loads that must be considered in soil nail wall design. Details about loads in bridges and roadways can be found in AASHTO (2012).

Conventional soil nail walls are not designed to resist scour. The designer must consider the potential for scour and, if this factor is of significance, an alternative wall type should be considered. Alternatively, appropriate measures will have to be considered for protection of the wall base if a soil nail wall is used.

4.3 Subsurface Investigations

4.3.1 Introduction

Soil nail walls should only be allowed for construction when site-specific geotechnical information, either existing and/or new, is available that fulfills the subsurface investigation objectives and requirements discussed in this chapter and any additional project needs described in Chapters 5 and 6. New geotechnical investigations would be necessary for most cases to: provide geotechnical information in the absence of historic information, close data gaps from past field investigations, or verify trends seen in historic investigations.

After completing a review of project-specific requirements and existing information, and ideally conducting a field reconnaissance, the engineer must develop a detailed work plan for conducting site geotechnical investigations.

Subsurface investigations should be performed in accordance with the AASHTO "Manual on Subsurface Investigations" (AASHTO 1988), Geotechnical Engineering Circular No. 5 "Evaluation of Soil and Rock Properties" (FHWA-IF-02-034; Sabatini et al. 2002), and "Manual on Subsurface Investigations" (FHWA-NHI-01-031; Mayne et al. 2001). Guidance for conducting field investigations can also be found in Section 10.4 of AASHTO (2012). Specific test procedures are presented in ASTM or AASHTO standards. Standards of the local agency, if available, may need to be consulted in some cases. The following sections highlight the main aspects of subsurface investigations used in soil nail wall projects.

4.3.2 Objectives

The objectives of a subsurface investigation are to:

- Determine the subsurface stratigraphy
- Obtain representative soil samples for laboratory testing
- Measure in situ soil properties through field testing
- Measure soil properties in the laboratory
- Assess soil and groundwater corrosion potential
- Evaluate the location and fluctuations of groundwater levels and gradients

The subsurface investigation must be conducted along the entire extent of the proposed wall. It must include a sufficient number of field tests and sampling locations to assess the variation of soil properties across the project site. Determining the location and nature of the groundwater is extremely important in soil nail wall projects because the presence of high groundwater (particularly in free-drainage soils) is an unfavorable condition.

The subsurface investigation may need to be conducted in a single phase or in multiple phases, depending on the complexity of the project and the availability of existing geotechnical data. If little or no geotechnical information is available for a site, conducting the investigation in two phases may be a sensible approach. The first phase would consist of a preliminary subsurface investigation to help identify the soil strata and their basic characteristics, and to determine if the ground is favorable, difficult, or unfavorable for soil nails. The second phase would consist of a final subsurface investigation focused on collecting the specific data required for design and construction of the soil nail wall.

The engineer must develop a subsurface exploration plan that allows developing a site characterization that is sufficient for design and construction of the soil nail wall. The lack

of a thorough site investigation will produce various undesirable outcomes, including uneconomical designs, re-designs, construction difficulties, additional construction costs and delays, and inadequate or unsafe performance.

4.3.3 Requirements for Selecting Location and Number of Exploratory Borings

The type, number, location, and depth of test borings and other field tests are a function of the wall dimensions, the presence of adjacent structures, and existing and future grades. Other factors include the stage of the project at which the field investigation is conducted (i.e., feasibility study, preliminary design, or final design), the availability of existing geotechnical data, and the variability of subsurface conditions.

Guidance for selecting the number of test borings and defining their depths is provided in Table 10.4.2-1 – "Minimum Number of Exploration Points and Depths of Exploration" of AASHTO (2012). Figure 4.1 provides preliminary guidance about the number and location of borings for earth-retaining systems. Borings must be drilled along the centerline of, in front of, and behind the proposed wall. Borings drilled along the proposed wall centerline should be spaced between 100 and 200 ft and be advanced at least to a depth H below the planned bottom of the excavation, where H is the height of the wall at that location.

Borings drilled behind the wall should be spaced up to 150 ft along the alignment, be located within H to 1.5H behind the wall, and be advanced at least to a depth 2H below final grades. If the ground behind the wall slopes up, borings should be drilled within a horizontal distance of 1.5H to 2H from the wall. These borings should be deep enough to allow assessing potentially larger sliding masses occurring up the slope.

Borings drilled in front of the wall should be spaced up to 200 ft along the alignment, be located between 0.75H to H in front of the wall, and be advanced at least to a depth H below the planned bottom of the excavation.

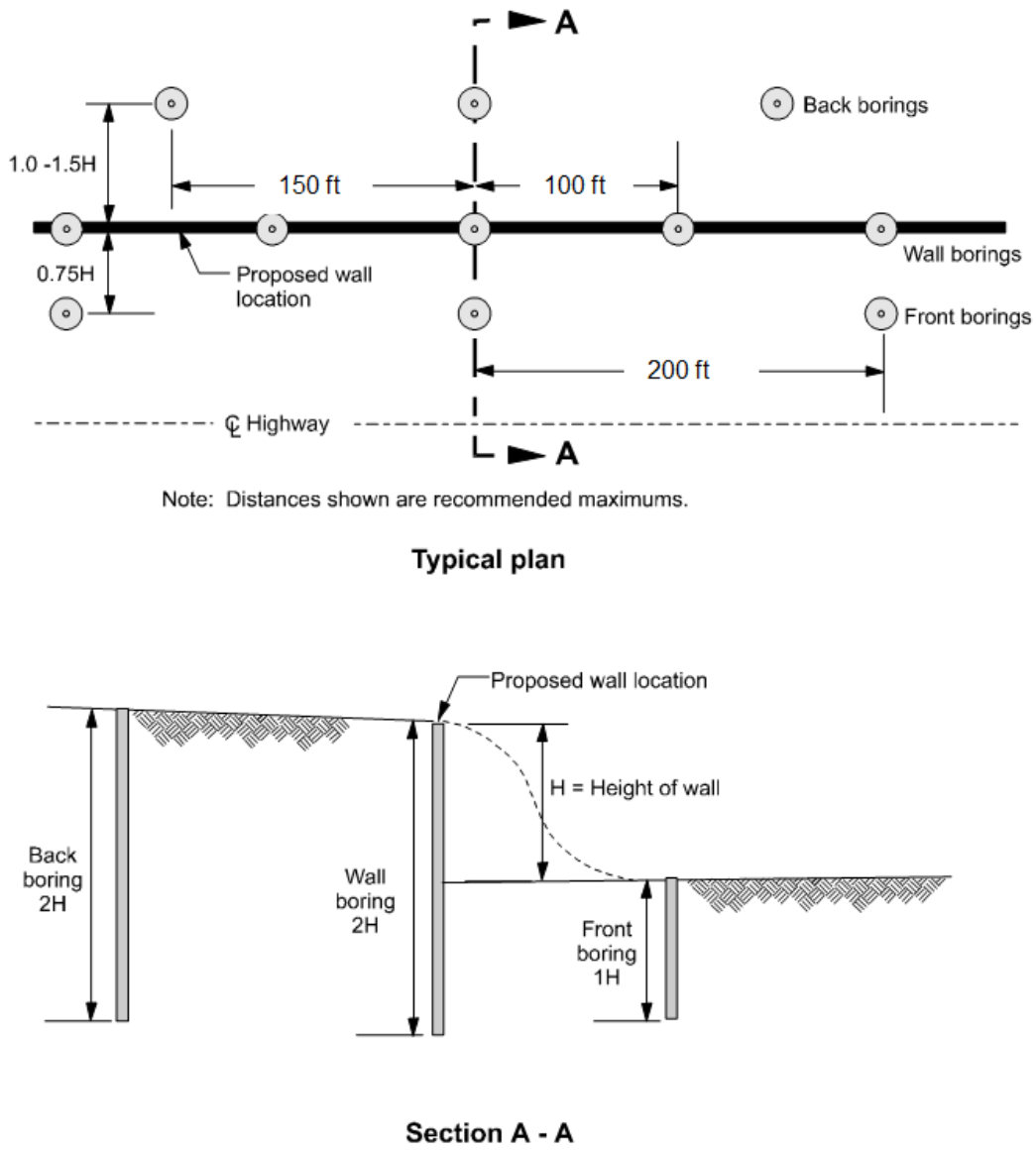


Figure 4.1: Illustration. Preliminary layout of borings. Modified after Sabatini et al. (2002).

When soft to very soft fine-grained soils, organic silt, or peat exist within the terminal boring depths shown in Figure 4.1, the depth should be increased at least by H for borings along and behind the wall, and by 0.5H for borings in front of the wall. For areas with significant seismic risk, borings should be extended to the base of deposits that are suspected to be liquefiable (i.e., deposits of loose to very loose, saturated, cohesionless soils), within a depth of about 2H below the planned bottom of the excavation. In most cases, the maximum depth of borings may not need to exceed approximately 50 ft.

If weathered rock is encountered within the drilling depths shown in Figure 4.1, borings should extend into the weathered rock so that a core sample, at least 10-ft long and in two 5-ft long runs, can be obtained to allow inspection of discontinuities. Finally, the designer must establish the depth of the borings in relation to any slip surfaces that likely may affect the stability and performance of the walls. Certain geologic conditions (joints, bedding, etc.) may require using oriented rock cores.

4.3.4 Field Procedures

4.3.4a Overview

This section presents general information regarding field procedures that are used for soil nailing projects. Sabatini et al. (2002) and Mayne et al. (2001) provide more detailed descriptions of site investigation and laboratory testing procedures. Corrosion field testing is covered in Section 4.7.

4.3.4b Test Borings and Standard Penetration Testing (SPT)

Subsurface investigations for soil nail projects are largely based on test borings. Test borings are advanced using rotary drilling methods, including hollow-stem auger and wash-borings. Both drilling methods allow retrieval of soil samples. Drilling with hollow-stem augers allows groundwater level observation, as discussed in Section 4.3.5.

Soil Standard Penetration Testing (SPT, ASTM D1586) and sampling of both disturbed and undisturbed soil samples are performed in test borings. SPT can be used in a wide range of soils that are favorable in soil nail projects. The SPT sampler allows obtaining disturbed samples; therefore, it is appropriate for sampling cohesionless soils (sands and elastic silt) for index geotechnical properties and for sampling soil for corrosion potential testing. The SPT sampler can retrieve residual soil to weathered rock, in which cases the SPT sampler is driven to refusal (defined when a blowcount of 50 blows has been reached for a penetration of one inch or less).

The SPT is not amenable to gravels, boulders and fill containing large particles. Even if sampling is successful, the retrieved samples of these soils would be unrepresentative because large particles cannot enter the sampler. Also, the blowcount is expected to be biased or meaningless.

Although the SPT is used in practice to sample soft to very soft, fine-grained soils (clays and plastic silts), thin-walled samplers (Shelby, Osterberg, Gregory Undisturbed Sampler, and other fixed-piston samplers) must be used to retrieve relatively undisturbed samples of these

soils that permit determination of mechanical properties through strength testing in the laboratory.

For most soil nail wall projects, SPT samples should be obtained at a depth interval of 5 ft. Sampling should be conducted at a smaller depth interval or continuously in thin layers containing soft to very soft, fine-grained soils, cavities, or soils with expected high corrosion potential.

Numerous correlations exist between the SPT N-value and soil engineering properties. The magnitude of overburden has a significant influence on the N-value and must be taken into account when using correlations with applicable soil properties. Also, corrections to N-values may be necessary to convert these values to N_{60}-values and account for different hammer efficiencies. An $(N_1)_{60}$-value, which is an N-value normalized to a 60-percent energy efficiency and to 1 tsf of overburden (ASTM D4633), should be used in some correlations. The engineer should exercise caution when using SPT-based correlations because N-values, even if corrected, tend to exhibit significant variability due to the numerous sources of uncertainty in this test.

In cases where deflection of the wall and settlement of the overlying ground surface are critical project constraints and require accurate estimates (e.g., using finite element methods or similar techniques), SPT-based correlations to estimate soil modulus values may not be suitable. Mechanical laboratory testing of undisturbed specimens such as oedometer or triaxial testing, or in situ tests such as pressuremeter testing, may be performed. The reader is referred to Sabatini et al. (2002) for additional information on this testing.

4.3.4c Cone Penetration Test (CPT)

The Cone Penetration Test (CPT, ASTM D5778) allows a rapid and cost-effective development of continuous subsurface profiles, and permits identifying thin soil layers, which would otherwise be difficult to detect using discontinuous SPT sampling. This capability may prove particularly useful when investigating thin layers of weak soil behind soil nail walls. However, CPT is more limited than SPT for soil nail wall design applications because the soils that are conducive to CPT are generally not ideal for application of soil nailing. CPT cannot be performed in hard soils, gravelly soils, weathered rock, and boulder deposits.

Correlations between CPT parameters (cone bearing resistance, friction ratio, etc.) and various soil engineering properties are available that may possibly carry less variability than SPT-based correlations, particularly for properties of soft fine-grained soils. For most soil nail projects, the soil shear strength can be estimated based on the CPT or the SPT data.

Likewise, the elastic modulus can be estimated from CPT data. However, there are no widely available correlations between CPT parameters and bond strength for soil nails.

Conventional CPT has a higher productivity (feet per day) than conventional drilled borings. However, CPT does not provide soil samples, which is a significant disadvantage compared with borings. However, some CPT contractors have the ability to collect small-diameter, disturbed, soil samples by pushing a tube sampler and using the same CPT equipment. The sampling is conducted after the CPT probing is complete, using a sampler that is similar to the CPT cone penetrometer but does not contain the internal electronics. While the samples are disturbed, they can be used for classification purposes, index tests, and soil corrosion potential. Some special CPT probes are equipped with sensors that provide a continuous profile of soil electric resistivity, which is one important parameter in the assessment of corrosion potential.

The combined use of CPTs (high productivity but no samples) and fewer-than-normal test borings with SPT (fewer but well selected samples) might be an attractive alternative in certain projects, particularly long wall alignments in relatively consistent geologic conditions.

4.3.4d Other Field Testing Techniques

Other field testing techniques such as the Pressuremeter Test (PMT, ASTM D4719) and the Field Vane Shear Test (VST, ASTM D2573) are, in general, not as widely used as SPT and CPT. The flat dilatometer test (DMT) is also available but its use is quite rare.

PMT can be employed in a wide range of soil types, including those favorable for soil nailing construction. The PMT can provide reasonable estimates of the soil modulus and is therefore an attractive testing method when the estimation of wall deflection and ground settlement is critical. Also, a correlation developed in France between the PMT limit pressure (p_L) and the bond strength has been developed for sand, clay, gravel, and weathered rock (Clouterre 2002). The VST has no routine application for design of soil nail walls as it is suitable for soft to medium fine-grained soils, which are unfavorable for soil nails. However, it may still be used in connection with a project that incorporates soil nails walls if underlying fine-grained strata are critical to the global stability of the excavation.

4.3.4e Test Pits

The excavation of test pits can provide valuable information on whether an excavation face can stand unsupported. Test pits should be at least 20- to 25-ft long and up to approximately 10 ft deep (when possible). They should be protected as required against collapse and meet safety standards. Test pits should preferably be excavated perpendicular to the wall to reduce

affecting the subsequent excavation face. If test pits are excavated parallel to the wall, they should be excavated in the cut zone, at least 15 ft from the wall face. To evaluate the stand-up time of the excavation, test pits should be left open for 1 to 2 days with adequate protection for workers and the public. Excavated test pits should be inspected daily, as a minimum.

Test pits allow observations of seeping groundwater, which may affect the stability of the open cut. Test pits also allow grab samples or undisturbed specimens of fine-grained soils to be collected for laboratory testing as needed.

4.3.5 Groundwater

4.3.5a Overview

The presence of groundwater, either stable (i.e., static) or perched, must be identified during subsurface investigations because groundwater affects various aspects of the design and long-term performance of soil nail walls. Groundwater, if present above the bottom of the excavation, will likely impact the stability of temporarily unsupported cuts during construction of a soil nail wall. Careful observations should be made in test pits to detect seeping groundwater, even in small quantities. Fluctuations in the groundwater table over time may promote corrosion, affect overall stability, and cause increased pressures on the wall facing. Groundwater may affect drill hole stability and influence the selection of grouting procedures and the methods needed for permanent drainage.

Grossly underestimating the elevation of groundwater or gradients during field investigations can have serious consequences for any earth retaining system, but particularly for soil nail walls. These systems are not suited for high, static groundwater levels, significant amounts of perched groundwater, or low to moderate seepage velocities, particularly in free-drainage soils.

4.3.5b Groundwater Observation Methods

The depth to groundwater should be obtained from borings during drilling, at the end of drilling, and at least 24 hours after drilling. It is advisable to terminate some borings as temporary stand-pipe piezometers of slotted PVC pipe. These piezometers need to be monitored for days, weeks, or even longer depending on the anticipated or detected groundwater fluctuations and ground conditions. Groundwater levels obtained during drilling in cohesive soils or other formations with low permeability do not generally represent stabilized groundwater levels and, if standpipe piezometers are installed, a substantial wait period may be necessary for groundwater readings to stabilize.

It is important to pay attention to the moisture content and index properties of fine-grained soils as measured in the laboratory. A fine-grained soil with its water content higher than the plastic limit (PL) may be saturated in the field and subject to positive pore pressures. Knowing the in situ unit weight, specific gravity, and water content can help to establish whether the soil is above or below the groundwater table. Other information about groundwater observation methods can be found in GEC 5 (Sabatini et al. 2002).

4.4 Laboratory Soil Testing

4.4.1 Overview

The design of soil nail walls requires that some of the engineering properties be determined by laboratory testing. The amount and scope of the laboratory testing to be performed depend on the particular characteristics of the project. For projects of less complexity, laboratory testing of index parameters may suffice if a thorough and adequate field investigation was performed previously. For some projects with challenging subsurface conditions, a wider range of laboratory testing must be considered even if some existing geotechnical information is available.

4.4.2 Soil Classification and Index Tests

Table 4.1 (a, b, c) lists laboratory tests used in soil nail projects to classify soils and determine index properties.

Table 4.1a: Procedures and Laboratory Tests for Soils – Classification

Test Name	ASTM Standard	AASHTO Standard
Visual Identification of Soils	D2488	NA
Classification of Soils According to USCS	D2487	M145
Particle-Size Analysis	D422-63	T88
Soil Fraction Passing No. 200 Sieve	D1140	T11

Table 4.1b: Procedures and Laboratory Tests for Soils – Index Parameters

Test Name	ASTM Standard	AASHTO Standard
Moisture Content	D2216	T265
Atterberg Limits for Fines Fraction	D4318	T89, T90

Note: Individual ASTM and AASHTO standards referenced herein can be found in ASTM (2012) and AASHTO (1988).

Table 4.1c: Procedures and Laboratory Tests for Soils – Shear Strength Tests

Test Name	ASTM Standard	AASHTO Standard
Unconfined Compressive Strength (UC)	D2166	T208
Unconsolidated Undrained Triaxial Compression (UU)	D2850	T296
Consolidated Undrained Triaxial Compression (CU)	D4767	T234
Consolidated Drained Triaxial Compression (CD)	D7181	NA

The tests included in Table 4.1c may be used on different soil types, but they are far more commonly used on fine-grained soils. Triaxial extension tests may also be considered for estimating soil shear strength parameters in unloading, which would be the predominant condition behind an earth retaining wall. However, the difference in results obtained from compression or extension triaxial tests is not expected to be large for soils considered favorable for soil nail walls.

Index tests should be performed for all soil nail projects for soil classification, according to the Unified Soil Classification System (USCS). These tests should include grain size distributions and Atterberg limits.

Sieve analyses help establish the grain-size distribution, fines content, and estimate the hydraulic conductivity of soils. Besides contributing to soil classification, Atterberg limits can be used to estimate compressibility and shear strength parameters of fine-grained soils. More specifically for soil nail projects, Atterberg limits are used to evaluate the potential for creep deformation of fine-grained soils that would be supported by a soil nail wall (see Section 4.6).

Testing for moisture content helps identify certain unfavorable conditions for soil nails. For example, in situ moisture contents significantly above the PL of the soil are indicative of low shear strength and high compressibility. If the in situ LL is greater than 50%, the potential for creep of the soil increases. In addition, the potential for creep increases if the Liquidity Index (LI) is equal or greater than 0.2 (Byrne et al. 1996). Moisture contents less than about 5% in granular soils with little fines may indicate that vertical cuts may not remain unsupported.

Organic content testing (i.e., loss-on-ignition, LOI D21974, T194) should be performed to determine whether soils are organic if field observations (e.g., odor, color, and/or texture of soil samples) or the review of existing information suggests the presence of organics. Also, organic soils with high natural moisture generally have a higher corrosion potential than inorganic soils. Organic soils also tend to have lower shear strength and be more compressible than inorganic soils, and have a tendency to creep over time.

Other testing, including specific gravity (D854, T100), might be required in some cases to support the estimation of other parameters.

Engineering properties, including strength and deformability, are presented in subsequent sections.

4.4.3 Shear Strength Tests

Most soils that are favorable for soil nailing are predominantly granular. The strength parameter values of these soils are estimated using correlations based on field testing methods (see Section 4.5.4) and not often measured using laboratory testing. If soil nails are installed in stiff to hard fine-grained soils or are used to stabilize the structural fill of an existing MSE wall, the shear strength of these materials should be estimated using laboratory tests, such as those listed in Table 4.1c.

The direct shear test (D3080, T236) may be used for fill when the sample is properly compacted in the lab to in situ conditions. Direct shear tests have a number of limitations. Because the failure surface is forced along a horizontal plane, rotation of principal stresses during the test and anisotropy of the specimen limit the applicability of the interpreted strength parameter values. In addition, the development of pore pressures is uncontrolled and their measurement is impractical.

In cases where soil nails are installed in stiff, overconsolidated clays that may mobilize fully softened or residual strengths upon deformation, the shear strength parameters may be estimated using triaxial testing (up to certain level of axial deformation) or the torsional ring shear apparatus device (D6467).

4.4.4 Deformability and Modulus

Where soft to medium fine-grained soils exist throughout the excavation or immediately below the bottom, it is important to evaluate the magnitude of wall deflections, ground surface settlement, and rebound. In these cases, the soil compressibility can be evaluated using conventional, one-dimensional consolidation tests (D2435, T216). Soil modulus values can also be estimated using triaxial testing.

It is important to note that the results of laboratory tests may be sensitive to the stress path followed and to the orientation of the specimen. Results may not be representative of field conditions unless special testing conditions are applied, or suitable interpretation of the results is performed, that account for differences with field loading or unloading.

4.4.5 Other Testing

Other tests, while not routinely used in the design of soil nails, can be useful to detect unfavorable conditions for soil nailing. For example, the presence of collapsible or swelling fine-grained soils can be established by performing the collapse potential test (D5333) and the swelling potential test (D4546, T258), respectively.

Special tests must be performed in areas exposed to soil freezing to establish the frost heave potential and susceptibility to thaw weakening (D5918) of the soils on the soil nails and the wall facing. These tests should be performed on samples obtained from soil layers that will be exposed to frost action in the long-term.

4.5 Selection of Design Parameters

4.5.1 Overview

Once sufficient information is available from borings, field tests, and the laboratory, the designer must develop the site stratigraphy and corresponding design parameters for each strata. The designer must also identify the variability of subsurface conditions across the site that may impact the design of the soil nail wall.

Developing the site stratigraphy involves locating the contact between soil, weathered rock, and possibly bedrock. The locations of these contacts are important not only for the global stability analyses, but also to establish whether the soil nails need to extend near or into the underlying harder materials. The contrasting conditions above and below this contact may affect the selected method and drilling equipment to advance the drill hole, construction procedures, and soil nail lengths. The designer must also identify the existence of moist, softer zones or water at the rock contact.

Developing the site stratigraphy is critical for design because the nature, extent, and distribution of the soil units affect the selection of the drilling methods, control the shape and dimensions of the potential sliding soil mass behind the soil nail wall, and affect the number and length of nails. The variability of the soils will dictate the number of nail verification tests that are necessary to confirm design criteria for each major soil deposit. The variability of subsurface conditions is more critical for design of long walls.

4.5.2 Soil Parameters for Design

The following properties, at a minimum, should be determined for soil nail projects from field investigation and laboratory testing:

- Depths of stratigraphy breaks
- Groundwater levels
- Unit weight of each major soil deposit
- Shear strength parameters of each major soil deposit
- Bond strength of each major soil deposit where nails will be installed

When accurate estimates of soil nail wall deflection or ground surface movement are necessary, the designer must also determine the values of soil modulus and compressibility based on results of field and/or laboratory testing, as described previously. The potential for soil creep must be assessed when dealing with fine-grained soils, as discussed in Section 4.6. The characterization of soil corrosion potential must be assessed for all types of soils where soil nails will be installed, as discussed in Section 4.7.

4.5.3 Unit Weight

The unit weight is used in stability analysis and calculations for soil nail wall design. The unit weight of granular soils and some fine-grained soils can be estimated from soil descriptions in conjunction with estimates of the relative density or through correlations (e.g., Kulhawy and Mayne 1990). Figure 4.2 shows the dry unit weight (γ_d) (expressed as a value normalized by the unit weight of water, γ_w) for various USCS-classified cohesionless soils without plastic fines, including some that are favorable for soil nailing, and as a function of the relative density (D_r). To account for the in situ moisture content, the moist unit weight, γ_{moist}, must be calculated as $\gamma_{moist} = \gamma_d (1+ w_n)$, where w_n is the in situ moisture content. Note that the use of Figure 4.2 requires knowing the relative density, which can be estimated using Table 4.2.a., or the friction angle, which is discussed below.

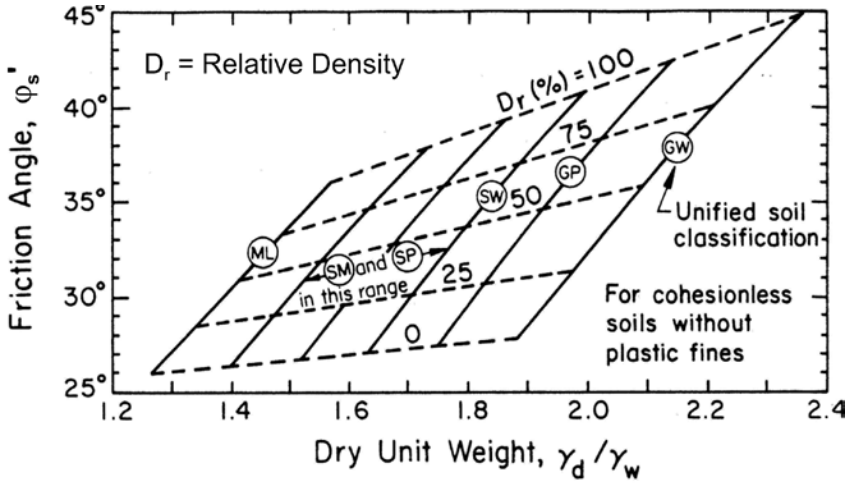


Figure 4.2: Graph. Estimation of soil unit weight. Modified after U.S. Navy (1982), Kulhawy and Mayne (1990).

The unit weight of fine-grained soils may be determined in the laboratory from undisturbed samples retrieved with a Shelby tube or similar sampler. Conversely, the in situ unit weight of granular soils should not be estimated through laboratory testing because samples are always disturbed and in situ conditions cannot be easily reproduced in the laboratory.

4.5.4 Shear Strength

4.5.4a Cohesionless Soils

For most projects, the drained, effective angle of internal friction (φ_s'), which represents the main component of the shear strength of cohesionless soils, can be estimated using correlations based on SPT and CPT values.

Effective stress analysis should be considered for cohesionless soils, which are frequently free-draining. A free-drainage condition requires a relatively high hydraulic conductivity of the soil, which can be estimated from the gradation of the soil. It is important to assess the fraction of material passing the #200 sieve (by weight), because the hydraulic conductivity of supposedly "free-drainage," granular materials can decrease significantly for a content of fines higher than about 15 to 25 percent.

The effective cohesion, c', is estimated separately. Cemented sands exhibit cohesion that should be considered for design of the soil nail wall; however, the designer should keep in mind that, depending on the project characteristics, there may be a potential for significant disturbance of the soil matrix during excavation and resulting loss of cohesion. Also, internal and surface erosion of the soil due to seepage and surface runoff may degrade the value of cohesion during the life of the project. Partially saturated soils exhibit apparent cohesion due to matric suction. Although it is customary to ignore this apparent cohesion, the designer may opt to consider cohesion for the analysis of temporary excavation stages if there is evidence that such apparent cohesion exists. In this case, short-term, apparent cohesion can be adopted to be between 50 psf and 150 psf.

Preliminary estimates of the effective friction angle can be obtained from Table 4.2a, using SPT N-values or from Table 4.2b using normalized CPT values. Figure 4.3 shows the increase of the effective friction angle based on SPT N_{60} as a function of increasing 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.

Table 4.2a: Correlations between SPT N-values, CPT, and Friction Angle of Cohesionless Soils – SPT

SPT N_{60} (1) (blows/ft)	Soil Density Description	Soil Relative Density	φ_s' (deg) (2)
0 to 4	Very Loose	< 20	< 30
4 to 10	Loose	20 – 40	30 to 35
10 to 30	Medium	40 – 60	35 to 40
30 to 50	Dense	60 – 80	40 to 45
> 50	Very Dense	> 80	> 45 (3)

Notes: (1) Uncorrected values are considered equivalent to N_{60}.
(2) From Kulhawy and Mayne (1990) based on Meyerhof (1956).
(3) Sands should be selected to have a friction angle ≤ 40 degrees. Gravelly soils should be selected to have a friction angle ≤ 45 degrees.

Table 4.2b: Correlations between SPT N-values, CPT, and Friction Angle of Cohesionless Soils – CPT

Normalized CPT Cone Bearing Resistance, q_c/P_a (1)	Soil Density Description	Soil Relative Density	φ_s' (deg) (2)
< 20	Very Loose	< 20	< 30
20 to 40	Loose	20 – 40	30 to 35
40 to 120	Medium	40 – 60	35 to 40
120 to 200	Dense	60 – 80	40 to 45
> 200	Very Dense	> 80	> 45 (3)

Notes: (1) P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.
(2) From Meyerhof (1956).
(3) Sands should be selected to have a friction angle ≤ 40 degrees. Gravelly soils should be selected to have a friction angle ≤ 45 degrees.

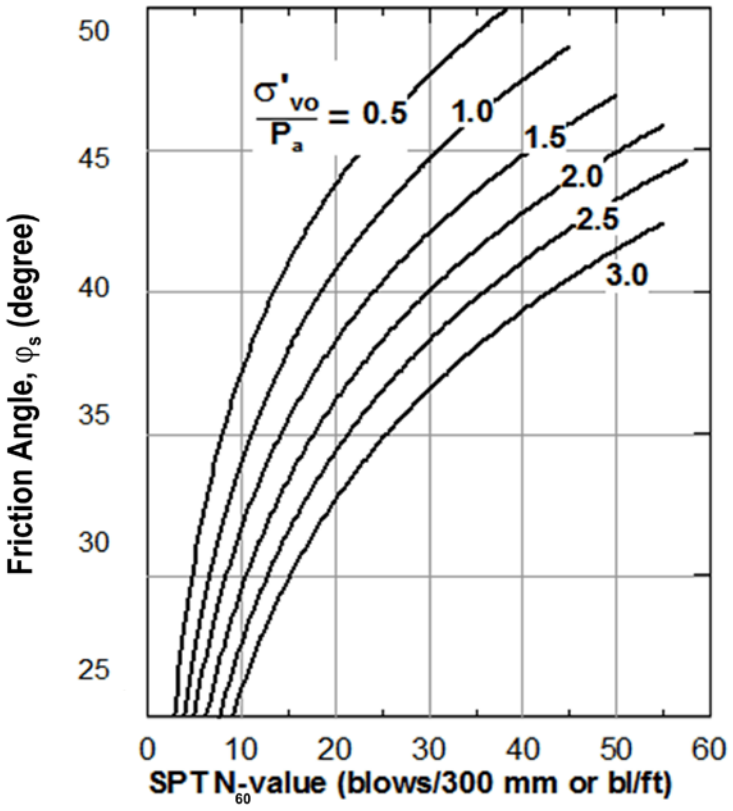


Figure 4.3: Graph. Friction angle of cohesionless soils as a function of normalized overburden (Modified after Schmertmann 1975).

4.5.4b Cohesive Soils – Drained Conditions

The shear strength of cohesive, fine-grained soils is generally a function of the rate of loading, as related to the ability of the soil to drain excess pore pressures. Cohesive soils are most often considered undrained in short-term excavation-support applications. The drained strength, however, must be considered when evaluating the long-term stability of a soil nail wall.

Overall stability evaluations of heavily overconsolidated cohesive soils would require that the long-term, drained conditions and associated drained friction angle of these soils be considered. In these cases, fully remolded (i.e., fully softened) or residual values of the friction angles must be used as opposed to peak values. Results from consolidated undrained triaxial (CU) tests with pore-water pressure measurements must be used to correctly assess the long-term drained strength of fine-grained soils.

When determination of strength parameters through testing is not possible, the effective friction angle of clays may be estimated as a function of PI using the relationship of

Figure 4.4. This figure does not apply to remolded or residual values. The designer must exercise caution when using correlations which may be applicable only to specific conditions.

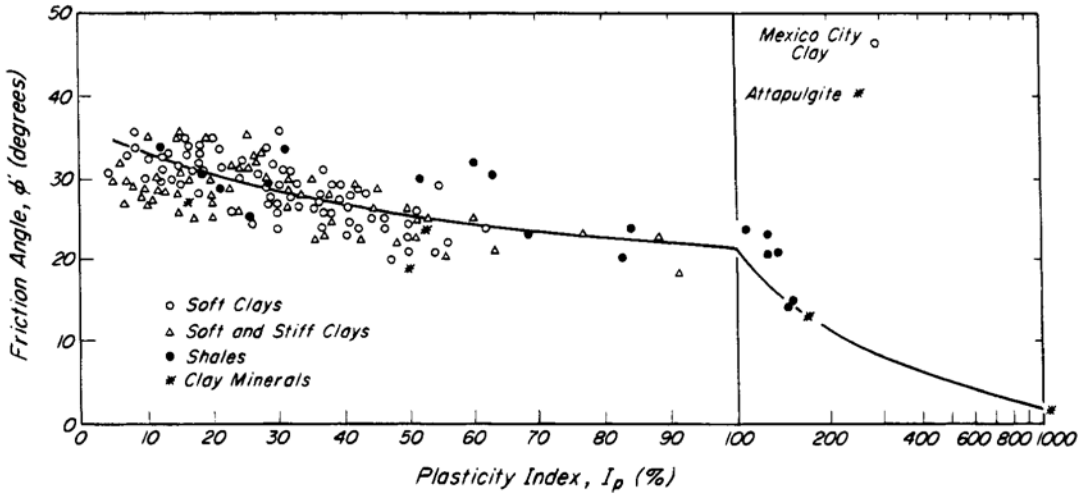


Figure 4.4: Graph. Relationship between friction angle and plasticity index (after Terzaghi, Peck, and Mesri 1996).

4.5.4c Cohesive Soils – Undrained Conditions

The undrained shear strength of fine-grained soils should be considered for short-term stability of a wall during construction, if these soils are affected by the excavation. For an excavation lift involving fine-grained soils, undrained stability analyses should be performed to model the wall stability immediately after excavation and before the nail installation for that lift.

The undrained shear strength (S_u) of fine-grained soils should be assessed using field techniques (preferably CPT or VST, although SPT may be used for preliminary estimates), and specific laboratory testing techniques that take into account the undrained response of these soils.

VST-based results obtained in the field should be modified using the Bjerrum correction (Kulhawy and Mayne 1990), which depends on the plasticity index. CPT-based estimates of S_u are a function of the cone tip resistance (q_c), the cone bearing factor (N_k) and the total overburden stress at the test depth. Various publications report N_k varying from about 10 to 20. The practice is to assume $N_k = 15$, which is appropriate for most situations. When possible, a site-specific calibration of N_k values should be performed by relating S_u values measured with adequate laboratory testing techniques (CU triaxial tests), carefully conducted VST tests, and CPT results.

In overconsolidated clays, triaxial testing on undisturbed specimens can be used to determine the value of S_u for design. However, the specimens for testing must be carefully selected so as to be representative of the shear strength throughout the stratum. It is especially important to consider that the value of S_u may vary significantly with stress history, desiccation, and fissuring. Fissuring may reduce the overall strength of the soil mass with respect to that determined from laboratory tests on specimens of limited size.

Evaluations of S_u should consider the stress history of the soils, in situ overburden, and in situ pore pressures. The preferred techniques for estimating S_u are: field VST and CU with pore-pressure measurements laboratory tests.

S_u values to be used for design should not be based on mini-torvanes, pocket penetrometers, or unconfined compression tests, because the results from these tests are greatly affected by local variations of the specimens and by the drying of the specimen surface. Furthermore, these test results do not reflect the shear strength along specific orientations within the soil mass.

When testing is not possible, S_u may be estimated as a function of various parameters. Numerous correlations exist between S_u and soil indices, including Atterberg limits (Kulhawy and Mayne 1990; Sabatini et al. 2002). As mentioned, the designer must exercise caution when using correlations, as they may only be applicable to specific conditions. Ideally, however, correlations of undrained shear strength with index property values are only used to supplement the values of shear strength measured in the field and the laboratory. A thorough review is recommended of other related parameters, such as moisture content, fines content and Atterberg limits, when evaluating S_u to understand possible site variability.

4.5.5 Compressibility and Modulus

Compressibility and soil modulus values may be estimated from correlations with SPT or CPT values. However, the value of these correlations is limited because of their potential inaccuracy. PMT can provide a direct measurement of the soil modulus over a range of soil strain; however, PMT is rarely used in practice of soil nails because accurate determination of modulus values is not often needed in soil nail projects.

Table 4.3a can be used to obtain initial estimates of the elastic modulus, based on the soil type. Tables 4.3b and 4.3c can be used to estimate the modulus from SPT and CPT results. Poisson's ratio (ν), which also is used in numerical analyses to estimate displacements, can be estimated from Table 4.3a. Poisson's ratio can also be evaluated from triaxial and oedometer tests; however, this level of accuracy in the estimation of this parameter is rarely needed in the design of soil nail walls.

Table 4.3a: Elastic Properties of Soils Based on Soil Type (after AASHTO 2014)

Soil Type	Range of Young's Modulus, E_s (ksf)	Poisson's Ratio, ν
Clay – soft sensitive	50 – 300	0.4 – 0.5 (undrained)
Clay – medium stiff to stiff	300 – 1,000	0.4 – 0.5 (undrained)
Clay – very stiff	1,000 – 2,000	0.4 – 0.5 (undrained)
Loess	300 – 1,200	0.1 – 0.3
Silt	40 – 400	0.3 – 0.35
Fine sand – loose	160 – 240	0.25
Fine sand – medium dense	240 – 400	0.25
Fine sand – dense	400 – 600	0.25
Sand – loose	200 – 600	0.20 – 0.36
Sand – medium dense	600 – 1,000	0.20 – 0.36
Sand – dense	1,000 – 1,600	0.30 – 0.40
Gravel – loose	600 – 1,600	0.20 – 0.35
Gravel – medium dense	1,600 – 2,000	0.20 – 0.35
Gravel – dense	2,000 – 4,000	0.30 – 0.40

Table 4.3b: Elastic Properties of Soils Based on SPT N-values (after AASHTO 2014)

Soil Type	E_s (ksf)
Silts, sandy silts, slightly cohesive mixtures	$8\ (N_1)_{60}$
Clean fine to medium sands and slightly silty sands	$14\ (N_1)_{60}$
Coarse sands and sands with little grave	$20\ (N_1)_{60}$
Sandy gravel and gravels	$24\ (N_1)_{60}$

Table 4.3c: Elastic Properties of Soils Based on CPT Cone Tip Resistance (q_c) (after AASHTO 2014)

Soil Type	Young's Modulus, E_s (ksf)
Sandy soils	$2\ q_c$ (q_c, in ksf)

4.5.6 Bond Strength

The pullout resistance of a soil nail is a quantity of fundamental importance in the design of soil nail walls. Calculation of the pullout resistance of each soil nail is based on the length of nail extending behind the potential slip surface under analysis, the diameter of the grout body

surrounding the tendon, and the bond strength of the soil-to-grout interface. The following sections provide information on the bond strength for solid bar soil nails, while the bond strength for HBSNs is discussed in Chapter 10.

4.5.6a Drilled and Grouted Soil Nails

The bond strength (q_u) is the (ultimate) shear strength of the soil-to-grout interface. This parameter is equivalent to a nominal bond resistance, as used in LRFD practice. In this manual, the term bond strength will be used for simplicity, regardless of the design platform being used.

Factors affecting the bond strength include the conditions of the ground around soil nails (e.g., soil type and soil overburden stress) and installation methods (drilling, drill hole cleaning, grouting procedure, and grout characteristics).

For preliminary design, the nominal bond strength of a soil nail can be estimated from published literature, correlations with parameters obtained from field tests, and soil nail load tests. Engineers may also estimate the bond strength based on local experience and construction techniques. The bond strength is not measured in the laboratory because the key aspects affecting the bond strength cannot be easily reproduced. Final design requires verification of the bond strengths with load tests (see Chapter 9).

Values of bond strength have been reported in the literature for drilled and gravity-grouted soil nails installed in various types of soils/rocks and for different drilling methods. Table 4.4 includes ranges of presumptive bond strength (i.e., nominal values) for various combinations of soil and drilling methods. The minimum and maximum bond strength values correspond approximately to the least favorable and most favorable conditions for each combination. The median of the range of values given for fine-grained soils is suitable for use as a preliminary value for design in many cases. Conversely, experience shows that the presumptive bond strength values for granular soils tend to be conservative and, thus, a value somewhat higher than the median can be used in most cases.

Typical ranges of the bond strength are included in Table 4.4 for gravity grouted soil nails. The bond strengths in Table 4.4 are provided for guidance. It is important that the design engineer estimates bond strengths based on soil descriptions and other factors, such as the soil shear strength and overburden, as described below. It is important that the bond strengths from Table 4.4 or any other source to be used in design must be confirmed in the field by soil nail load testing.

Pressure grouting and post-grouting have a beneficial effect on bond strength. However, these procedures are uncommon in soil nail practice due to their added cost, and because

other measures can be taken to increase the pullout capacity of soil nails. For example, the Contractor may simply increase the diameter of the drill hole or improve the cleaning method to obtain larger pullout resistances.

When nails are installed in ground 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 dry loess using the gravity grouting method. In this case, the bottom and lateral portions of the drill hole wall are weakened because the grout provides moisture to the dry loess, possibly causing the drill hole walls to collapse. Also, along portions of the nail, the grout may permeate and develop a void between the nails and the drill hole wall. Experience has shown that adequate bond strengths can be achieved in loess, using pressure grouting or re-grouting with a tremie tube.

Table 4.4a: Estimated Bond Strength for Soil Nails in Coarse-Grained Soils (Modified after Elias and Juran 1991)

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)
Rotary Drilled	Sand/gravel	15 - 26
Rotary Drilled	Silty sand	15 - 22
Rotary Drilled	Silt	9 - 11
Rotary Drilled	Piedmont residual	6 - 17
Rotary Drilled	Fine Colluvium	11 - 22
Driven Casing	Sand/gravel w/low overburden (1)	28 - 35
Driven Casing	Sand/gravel w/high overburden (1)	41 - 62
Driven Casing	Dense Moraine	55 - 70
Driven Casing	Colluvium	15 - 26
Augered	Silty sand fill	3 - 6
Augered	Silty fine sand	8 - 13
Augered	Silty clayey sand	9 - 20

Note: (1) Low and high overburden are defined as effective overburden pressure being, respectively, less than and greater than 1.5 tsf.

Table 4.4b: Estimated Bond Strength for Soil Nails in Fine-Grained Soils (Modified after Elias and Juran 1991)

Drill-Hole Drilling Method	Soil Type	Bond Strength, q_u (psi)
Rotary Drilled	Silty clay	5 - 7
Driven Casing	Clayey silt	13 - 20
Augered	Loess	4 - 11
Augered	Soft clay	3 - 4
Augered	Stiff clay	6 - 9
Augered	Stiff clayey silt	6 - 15
Augered	Calcareous sandy clay	13 - 20

Estimated values of bond strength in weathered rock and rock are presented in Table 4.5 as a reference. The Post-Tensioning Institute (PTI 2005) also presents presumptive values of the nominal bond strength of ground anchors grouted under gravity.

Table 4.5: Estimated Bond Strength for Soil Nails in Rock – Drilling Method: Rotary Drilled (Modified after Elias and Juran 1991)

Rock Type	Bond Strength, q_u (psi)
Marl/limestone	44 - 58
Phyllite	15 - 44
Chalk	73 - 87
Soft dolomite	58 - 87
Fissured dolomite	87 - 145
Weathered sandstone	29 - 44
Weathered shale	15 - 22
Weathered schist	15 - 25
Basalt	73 - 87
Slate/Hard shale	44 - 58

4.5.6b Pullout Resistance per Unit Length

The ultimate pullout resistance per unit length of soil nail (r_{PO}) is equal to the bond strength multiplied by the perimeter of the bond zone. This quantity has also been referred to as the ultimate load-transfer rate. A correlation has been developed between SPT $(N_1)_{60}$ and the ultimate pullout resistance per unit length for gravity-grouted ground anchors installed in

soils with a relatively small diameter of 4 to 6 in. (Sabatini et al. 1999). Table 4.6 presents presumptive values of the pullout resistance per unit length for sand/gravel, sand, sand and silt, and silt-clay mixtures of low plasticity, and sand/silt mixtures as a function of soil density/consistency and $(N_1)_{60}$ ranges.

The bond strength can be estimated by dividing the values presented in Table 4.6 by the perimeter of the drill hole, once its diameter has been selected.

Table 4.6: Presumptive Ultimate Values of Soil Nail Pullout Resistance per Unit Length [(1)] (Modified after Sabatini et al. 1999)

Soil Type	Relative Density/ Consistency	SPT $(N_1)_{60}$ Range	Ultimate Pullout Resistance per Unit Length, r_{PO} (kip/ft)
Sand and Gravel	Loose	4-10	10
Sand and Gravel	Medium dense	11-30	15
Sand and Gravel	Dense	31-50	20
Sand	Loose	4-10	7
Sand	Medium dense	11-30	10
Sand	Dense	31-50	13
Sand and Silt	Loose	4-10	5
Sand and Silt	Medium dense	11-30	7
Sand and Silt	Dense	31-50	9
Silt-clay mixture of low plasticity or fine micaceous sand or silt mixtures	Stiff	10-20	2
Silt-clay mixture of low plasticity or fine micaceous sand or silt mixtures	Hard	21-40	4

Notes: (1) Values are for small-diameter (4 and 6 in.), straight-shaft, gravity-grouted ground anchors installed in soil.

4.6 Soil Creep

All soils retained by a passive structure, even granular soils, tend to creep under sustained shear stress over time. While in granular soils the effect is quite small, creep may be much more significant in fine-grained soils and soils with significant organic content. Creep will tend to induce deformation of the wall and the surrounding areas and, in the case of fine-grained soils, may result in some reduction of the shear strength of the soil. Creep of the soil mass behind a soil nail wall can be addressed following appropriate soil mechanics

procedures during slope stability analyses, which may include the use of fully softened shear strength and other considerations.

Time-dependent displacement may also take place at the soil nail grout-soil interface. Creep takes place under constant effective stress and may lead to excessive, time-dependent lateral deflection of the wall. Although there are no definitive criteria to estimate before construction which soil nails would creep, it is considered that creep may occur in soils that meet one of the following:

- LL ≥ 50
- PI ≥ 20
- S_u ≤ 1,000 psf
- Liquidity index (LI) ≥ 0.2
- Soil is organic

In addition, the soil stresses should be kept well below strength peak values to reduce the creep potential. Creep potential can be evaluated during construction through load testing of individual test soil nails. Details of creep testing are provided in Chapter 9.

4.7 Soil Corrosion Potential

4.7.1 Overview

Corrosion potential is of primary concern in soil nail applications and must be evaluated in every soil nail project. Corrosion potential is assessed through laboratory soil testing of samples obtained from field investigations and through field testing. The following properties must be assessed:

- pH (potential of hydrogen)
- Electrical resistivity
- Chloride content
- Sulfate content
- Organic content

All the parameters listed above are measured in the laboratory. Electrical resistivity and pH can also be measured in the field (see below). It is recommended that each of the properties listed above be measured during the feasibility phase and/or final investigations of a soil nail project.

4.7.2 Field Corrosion Testing

It is desirable to perform field measurements of soil resistivity, a quantity associated with soil corrosion potential. Field measurements could be performed using different arrays. The FHWA document titled "Application of Geophysical Methods to Highway Related Problems" (Wightman et al. 2003) describes the Schlumberger Array and the Wenner tests. ASTM G57 (2012) "Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method" describes the Wenner Array test.

These field methods, which are minimally invasive, provide electrical resistivity profiles that may aid in identifying potentially problematic zones within the subsurface or geological profile. These tests can provide independent resistivity measurements from those obtained in the lab. If these tests are conducted before a geotechnical investigation, they may help to plan where to obtain soil samples for laboratory testing of electric resistivity.

Some special CPT probes allow obtaining a continuous profile of the soil electric resistivity. Several portable probes exist that can be used in the field to provide pH values. In general, field-measured pH values should be verified by laboratory testing.

4.7.3 Testing of Corrosion Potential in Soil

Investigation plans should include retrieval of soil samples to be tested for corrosion potential. Samples should be obtained from locations where soil nails and any metallic components of the wall may be in contact with the soils. The number of samples depends on the size of the project and the aggressiveness and variability of native materials. As a minimum, three samples from each major soil deposit are recommended. More samples may be specified when the on-site soils are known to be aggressive and/or variable.

The laboratory tests are conducted according to the following standards:

Table 4.7: Laboratory Test Standards

Laboratory Test	Standard
pH	ASTM G 51
Soil Resistivity	AASHTO T288
Water-Soluble Chloride Ion Content in Soil	AASHTO T291
Water-Soluble Sulfate Content in Soil (Method A gravimetric)	AASHTO T290
Organic Content	ASTM D2974, AASHTO T194

Stray currents may also contribute to the corrosion potential of soils. Stray currents may result from power sources, such as electric rail systems, electrical transmission systems, and welding operations, and are particularly damaging in a coastal environment. Stray currents may be DC or AC and exist where the resistance of the ground becomes lower than that of the conduits or paths. When the sources are located more than about 100 to 200 ft from the nail tendons, the potential of stray current corrosion is very small (Elias 2000). Stray currents and their potential for impacting corrosion of soil nail walls should be evaluated by experienced personnel using geophysical field methods.

4.7.4 Corrosive Soil Types

Besides the conditions listed above, certain additional environmental and/or chemical conditions inherent in some soils make them more aggressive. This section provides examples of aggressive soils and lists other factors that may increase soil corrosion potential (Elias et al. 2009). The list does not imply that soil nails would be appropriate for use in some of these materials. For example, permanent soil nails would not be installed in industrial refuse or peats, which contain large amounts of organic matter.

- Acidic soils – These soils exhibit a naturally low pH (less than 5) and include pyritic soils and soils with a high level of soluble iron, which in turn can contain acidic iron sulfides. Acidic (sulfate) soils are found in areas containing pyritic soils, such as in the Mid-Atlantic States, and more specifically Appalachia. These pyritic soils may have pH < 4.5, SO_4 (1000-9000 ppm), and CL (200-600 ppm). Residual soils of tropical regions (e.g., Hawaii and Guam) with high rainfall may be subject to long-term leaching of soluble salts that lower the pH and increase the corrosion potential.
- Sodic soils – These are encountered in the western U.S. and arid environments. They exhibit low hydraulic conductivity, and may be alkaline soils (pH>7 or even pH > 9). They also tend to have high solute content including dissolved salts with sodium cations, and chloride and sulfate anions, which are known to cause corrosion. The low precipitation and intense evaporation cause soluble salts (e.g., sodium, chloride, and sulfates) to be gradually transported from the underlying rock to shallow layers of soil by capillary forces.
- Calcareous soils – These soils are another type of alkaline soils (7<pH<9) that may contain large concentrations of sodium, calcium, and calcium-magnesium carbonates and sulfates. These are mildly corrosive. Examples of these soils include those derived from calcite, dolomite and gypsum. Calcareous soils occur in Florida, Texas, New Mexico, and in the western U.S.
- Organic soils – When soils contain organic materials, they can initiate the formation of anaerobic pockets that may become contaminated with sulfate-reducing bacteria, thereby initiating severe pitting (described in Chapter 7). Testing for organic content

is part of the AASHTO criteria for determining corrosivity (see Table 7.1). Total organic content should be limited to 1% by weight of the total soil fraction to preclude formation of anaerobic pockets. Potentially organic soils include peats, mucks, cinder, and bogs, all exhibiting unusually high water content, and those soils with humic acid.

- Material of Industrial Origin – Industrial fills may include slag, fly ash, or fills containing construction debris. Some of these fills derived from industrial waste may have characteristics that can raise their pH. Other types of refuse (e.g., acid mine tailings and acid mine refuse) can have a low pH.
- Coastal Environments – These are encountered in Eastern, Southern and Western Seaboard States and Utah. Atmospheric salts and salt laden soils in marine environments may contribute to corrosion.
- Road Deicing Salts – These are encountered in Northern States. Deicing liquid contains salts that can infiltrate into soils and contribute to corrosion potential.

Other environmental factors may affect the corrosion potential. For example, the corrosion potential of granular soils seems to increase when these soils have a degree of saturation in the range of 60 to 80%. In regions where deicing salts are used, the top 7 to 8 ft of soil behind a soil nail wall should be assumed to contain a higher concentration of chlorides.

4.8 Frost Potential

Soil nail walls constructed in cold regions are susceptible to experiencing increased nail loads due to frost formation. Ice lenses can form if: (i) temperatures fall below freezing; (ii) the ground contains soils susceptible to frost; and (iii) a source of water (i.e., perched lenses or groundwater) exists. Ice formation in the soil can cause large pressures on the wall facing and loads in soil nails. Damage to soil nail facing was reported (Byrne et al. 1998) and settlements of the soil behind the wall may also occur when the frozen soil thaws.

Elias and Juran (1991) conducted a survey of monitored soil nail walls subject to frost action. They reported that nail loads are larger with increases in: (i) frost penetration depth; (ii) intensity and duration of the freezing season (as defined below); and (iii) the proximity of groundwater or other water sources to the wall. Nail loads during the freezing season were reported to be approximately twice as much as the loads measured immediately after construction. Kingsbury et al. (2002) reported that the force measured in nail heads caused by frost can be 2.5 times the maximum seasonal nail forces arising without frost. Some increase in soil nail loads and facing pressures can be expected when frost forming in susceptible soils lasts more than one week. The largest force increases occur near the facing and soil nail head connections, generally within 5 ft from the face of the wall. The maximum soil nail forces do not generally occur at the same time as the frost develops.

Ice lenses can form and tend to wedge between soil particles (Hallin 2004), but this effect changes with soil types. Little to no frost action generally occurs in free-draining granular materials because the relatively large voids permit water to freeze in-place without segregation and then form ice lenses, and they can better accommodate the expansion of ice lenses. Conversely, silts are generally highly susceptible to frost because the voids are relatively small and the capillary potential is relatively high.

The frost penetration depth is a critical parameter to ascertain the frost effect in soil nail walls. It increases with latitude and climatic factors and depends on soil properties. Frost penetration depth in the continental U.S. varies from approximately 0 in the southernmost states to 6 ft in the north. Contours of frost penetration depths can be obtained from various sources (e.g., FHWA-HRT-08-057, Selezneva et al. 2008).

The direction of heat loss, and hence frost creation, also effects soil nail walls. The heat loss is vertical near the ground surface behind the wall and horizontal through the facing. The effect depends on how frost formation advances into the soil as two-dimensional (and occasionally three-dimensional) fronts of frost penetration. This contrasts with flat structures, such as pavements where the direction of heat loss is mainly vertical. Studies demonstrated that thermal effects behind walls are as intense at the top and behind the facing when no insulation is provided (Duchesne 2003). To counter these effects, insulation may be provided.

The design of insulation should be based on the frost penetration depth and a measure of the severity and duration of freezing. The quantity defined as the Air Freezing Index (AFI), which is used in pavement design, expresses jointly the intensity and duration of freezing over the cold season. AFI has units of degree F-days and is computed as the sum of daily, average air temperatures that are below 32 degrees over the cold season. AFI may be used to estimate frost penetration depths because the frost penetration depth and AFI are approximately proportional. AFI contours are obtained from the NOAA National Climatic Data Center (NCDC) web site. With this information, the design engineer would evaluate design options, including insulation, to provide frost protection as presented in Section 6.10.14 “Frost Protection and Design for Frost.”

4.9 Seismic Data

Seismic loading must be considered to safely design and construct soil nail walls located in earthquake-prone regions. While the peak ground acceleration (PGA) is the main parameter to characterize seismic loading, the peak ground velocity (PGV) is used in some aspects of the seismic design of earth-retaining systems. Peak ground accelerations are differentiated as horizontal (in two orthogonal directions) and vertical components. The vertical component is

routinely disregarded; therefore, only the geometric average of the two horizontal ground acceleration components is considered for design. This value is referred to as PGA herein, unless indicated otherwise.

Structural effects of ground motions are routinely represented by a spectrum of response accelerations to an applied ground motion, which are the collective response of elastic structures with varying periods, T, and with a 5-percent damping. Modern codes, including AASHTO (2014), draw from the National Seismic Hazard Maps developed by the USGS (Petersen et al. 2014) to provide acceleration spectra based on only three parameters for the seismic design of structures. The parameters are PGA (equivalent to the response acceleration at T = 0), S_s (response acceleration at a short period of T = 0.2 sec), and S_1 (response acceleration at T = 1.0 sec). PGA is always used in the seismic design of earth-retaining systems; S_1 is used when a wall seismic-deformation needs to be considered; S_s is not often needed.

Modifications to the AASHTO (2014) values need to be considered. First, the AASHTO (2014) PGA, S_s, and S_1 values used for the seismic design of bridges and substructures are based on a 1,000-year return period. However, Owners may adopt a more severe seismic intensity for their structures and select a longer return period than AASHTO specifies.

In addition, the USGS maps are developed for a reference (mild) rock site (described as a Site Class B). However, if the site is not underlain by mild rock, PGA and S_1 must be modified if the site is underlain by soils (these produce seismic-wave amplification) or hard rock (this produces de-amplification). PGA and S_1 must be modified using site adjustment factors F_{PGA} and F_v, respectively, which are selected considering site subsurface conditions and the amplitudes of PGA and S_1. Subsurface conditions are represented by a single parameter, which is chosen as one of the following: average shear-wave velocity ($v_{_bar}$), average N_{60} ($N_{_bar}$), or, in the case of cohesive soils, average S_{u_bar}, all of which are calculated in the upper 100 ft below the ground surface. Note that these averages are not arithmetic averages but averages that rely on wave-propagation theory. For example, $v_{_bar}$ is calculated as follows:

$$v_{_bar} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

Equation 4.1: Average shear wave velocity.

Where:

d_i = thickness of layer "i", with ranging from i = 1 to n. Also, $\Sigma d_i = 100$ ft.

v_{si} = shear-wave velocity (in ft/s) of layer "i"

The average $N_{_bar}$, which can be used for cohesionless soils, cohesive soils, and rock, is defined as:

$$N_{_bar} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Equation 4.2: Average SPT N value.

Where:

N_i = SPT blow count of layer "i" per ASTM D1586. This value must be as measured in the field without corrections. N_i must be limited to 100 bpf where refusal is encountered in very competent materials.

A similar equation can be developed for S_{u_bar}.

Once these averages are computed, a site class, from Site Class A (hard rock site) through Site Class F (extremely poor soil conditions) is assigned to the site, according to Table 4.8. Site Class F requires that site-specific dynamic site response analysis be performed to quantify ground motion amplifications with a greater accuracy than that provided by the simplified 3-parameter AASHTO procedure. Note that site-specific analysis requires subsurface material parameters in addition to those described in this chapter. The additional parameters include: shear-wave velocity (or alternatively, small-strain shear modulus), cyclic modulus degradation curves, damping curves, and possibly, cyclic residual shear strengths. Kavazanjian et al. (2011) presents additional information.

The site adjustment factors F_{PGA} and F_v can then be obtained from Tables 4.9 and 4.10. Design, corrected values PGA_D and S_{D1} are obtained as:

$$PGA_D = F_{PGA}\, PGA$$

Equation 4.3: Design, site-corrected peak ground acceleration.

$$S_{D1} = F_v\, S_1$$

Equation 4.4: Design, site-corrected response acceleration at 1 second.

PGA_D is input in stability analysis as a seismic coefficient, and S_{1D} is used to modify the seismic coefficient considering seismically-induced deformations. Detailed procedures for the evaluation of soil nail walls are presented in Chapter 6.

Table 4.8: Definition of Site Classes (Kavazanjian et al. 2011)

Site Class	Soil Profile Name	Average Shear-Wave Velocity v_s (ft/sec) (1)	Average SPT Blowcount N_{60} (bpf) (1)	Average Undrained Shear Strength S_u (psf) (1)
A	Hard rock	>5,000	NA	NA
B	Rock	2,500 to 5,000	NA	NA
C	Very dense soil and soft	1,200 to 2,500	>50	>2,000
D	Stiff soil	600 to 1,200	15 to 50	1,000 to 2,000
E	Soft clay soil	<600	< 15	<1,000
E	-	(2)	(2)	(2)
F	-	(3)	(3)	(3)

Notes: (1) Averaged in the upper 100 ft of the subsurface profile.
(2) Any profile with more than 10 ft of soil having the following characteristics:
- Plasticity index, PI > 20; or
- Moisture content, $w_n \geq 40\%$; or
- Undrained shear strength, $S_u < 500$ psf.

(3) Any profile containing soils having one or more of the following characteristics:
- Soil that is vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
- Peats and/or lightly organic clays having a total thickness (H) greater than 10 ft.
- Very-high plasticity clays with PI > 75 with H > 25 ft.
- Very thick soft/medium stiff clays with H > 120 ft.

Table 4.9: Site Coefficient F_{PGA} (Kavazanjian et al. 2011)

Site Class	PGA ≤ 0.25	PGA = 0.5	PGA = 0.75	PGA = 1.0	PGA ≥ 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Note: (1) A site-specific dynamic site response analysis must be performed in these cases.

Table 4.10: Site Coefficient F_v (Kavazanjian 2011)

Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	(1)	(1)	(1)	(1)	(1)

Note: (1) A site-specific dynamic site response analysis must be performed in these cases.

Chapter 5: Resisting Mechanisms and Limit States

5.1 Introduction

This chapter introduces the load and resisting mechanisms and the relevant limit states that should be considered in the design of soil nail walls. It also presents a design framework following LRFD principles combined with ASD aspects as used in soil nail design software, SNAP-2 (Siel 2014). Resistance factors utilized in an LRFD framework for each limit state are back-calculated from ASD factors of safety.

5.2 Load Transfer in Soil Nail Walls

The load-transfer mechanisms that are applicable in soil nail walls from construction to completion are described below:

- The initial lift height is selected so at the end of Excavation Phase 1 (Figure 5.1), the portion of the soil behind the excavation is stable and the soil can remain unsupported before any nails are installed. Only the soil resistance participates at this stage as it is mobilized along potential slip surfaces.
- The upper soil behind the excavation tends to deform outward throughout Excavation Phase 1, after the installation of the first row of nails (Nail 1) and after the placement of the initial facing on the excavated face. The soil deformation generates lateral earth pressure on the facing. This pressure is transferred to the soil nails, and in turn is transferred from the soil nail to the soil through shear stress mobilized along the grout soil interface.
- Axial tensile force (T_1) develops in Nail 1, with its magnitude varying along the nail. T_1 varies as a function of the distance (x) along Nail 1 (top portion of Figure 5.1). T_1 increases from the facing (x=0) to a maximum value (T_{max}) near the critical slip surface for the excavation stage being considered, and decreases toward the soil nail end (x=L, where L is the length of Nail 1).

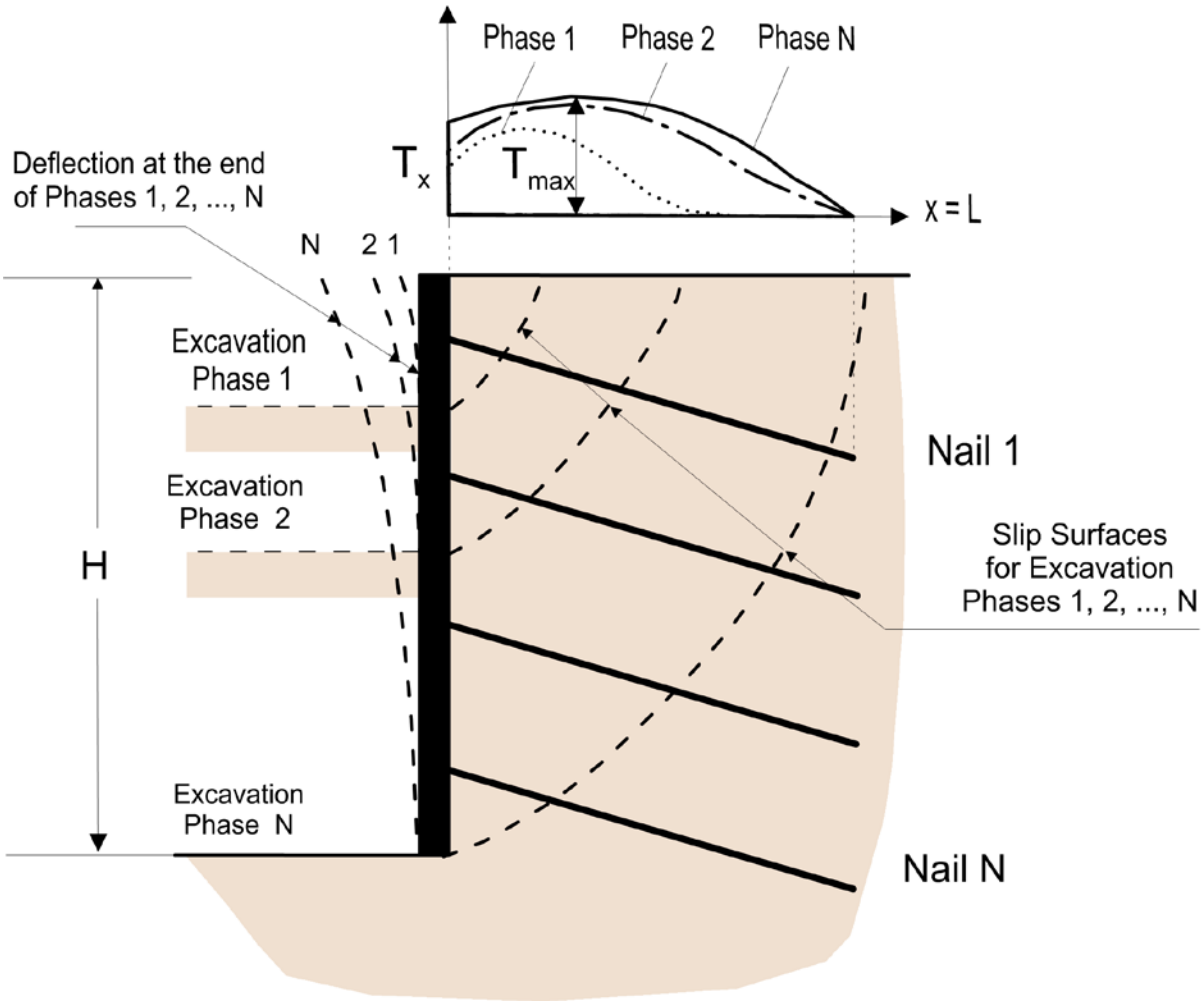


Figure 5.1: Illustration. Potential slip surfaces and soil nail tensile forces.

- In Excavation Phase 2, the partially completed wall and the upper portion of the soils deform farther. The critical slip surface at this excavation phase is larger than that at the previous excavation level, as the volume of deformed soil expands with a deeper excavation. The additional soil deformation causes additional pressures at the facing, a larger tensile force T_1, and larger shear stresses at the nail-ground interface. As the critical slip surface moves farther back from the excavation face, the location of T_{max} also moves farther back from the face.
- After Nail 2 is installed and facing is placed in front of Excavation Phase 2, subsequent soil movement above the excavation depth causes additional loads to be transferred to Nail 1 (note the increase in T_1 at this stage), and generates loads in the recently installed Nail 2.
- The increasing lateral deformation in subsequent excavation stages through the last stage (Excavation Phase N) causes the shear stress along the nail-ground interface and

the axial force to continue to increase in the lowest and intermediate nails. The maximum force in these nails is likely reached at the final excavation phase.

- The increasing lateral deformation with each excavation stage may cause the loads in upper nails to redistribute and possibly decrease. The contribution of the upper nails to the stabilization of a larger sliding mass decreases as the critical slip surfaces become deeper. The upper nails may in some cases appear to become entirely ineffective, as shown by stability analyses (see Section 5.6 "Overall Stability of Soil Nail Walls"), in tensile resistance for deep slip surfaces extending behind the terminal end of upper nails. However, the upper nails are still important because they provide local stability.

Nail forces tend to experience a moderate increase, up to approximately 15 percent, after the end of construction, over time and for long-term conditions (Plumelle et al. 1990; Holman and Tuozzolo 2009). These post-construction increases in nail forces occur mainly due to soil creep and stress relaxation. These increases in load are not explicit in soil nail design but should be considered by selecting suitable values of bond strengths that would account for soil creep.

The design of soil nail walls must consider both intermediate and final excavation stages to establish the most critical case at each soil nail level. The critical situation for most designs arises when a combination of loads (including dead load, live load and extreme loads such as earthquake) acts on the wall. However, a critical case may also occur after an excavation lift is completed but before the nails and shotcrete are installed at that level (Figure 5.2). The potential for instability of the unsupported cut at this stage may be exacerbated when seepage forces act on the open face.

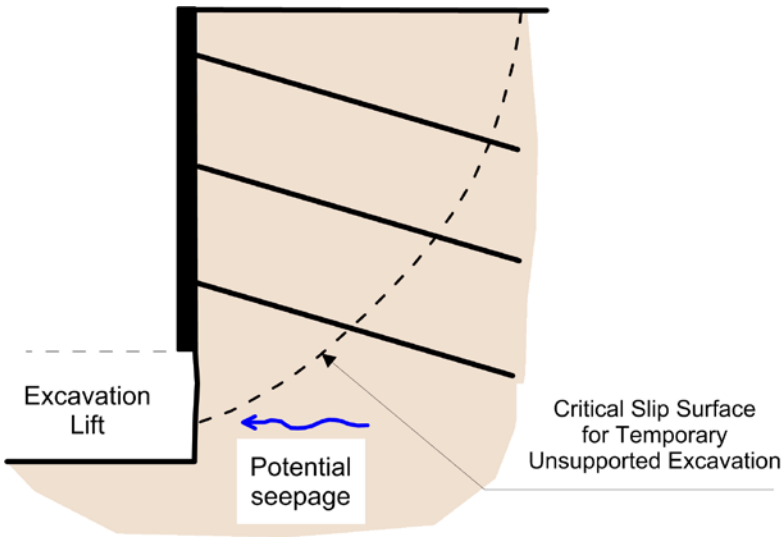


Figure 5.2: Illustration. Potential critical stability case during construction.

5.2.1 Soil-Nail Interaction and Tensile Force Distribution

5.2.1a Overview

The interaction between soil nails, the soil behind the wall, and the facing is complex and causes redistributions of tensile forces in the nails. The mobilized shear stress along the grout-soil interface, q, is in general not uniform and changes in direction along the nail length (Figures 5.3a and b). The maximum force occurs along the nail (Figure 5.3c).

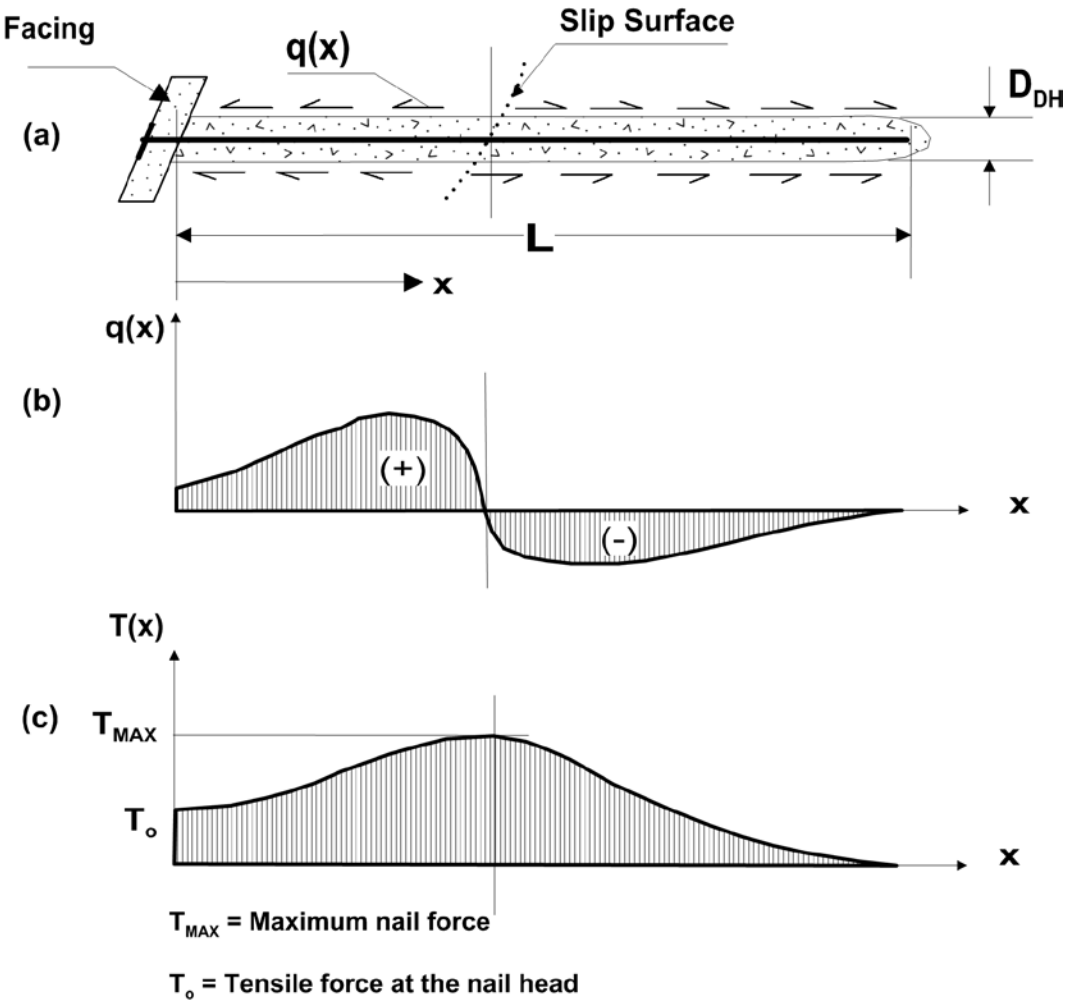


Figure 5.3: Illustration. Stress-transfer mechanism in soil nails: (a) basic soil layout; (b) schematic distribution of shear stresses at the grout-soil interface; (c) schematic distribution of resulting tensile forces.

5.2.1b Maximum Tensile Force Distribution

The tensile force that can develop in a tendon depends on the location where the nail crosses the slip surface. As mentioned, the location of maximum nail tensile forces is close to, but

generally does not coincide with, the critical slip surface established in stability analyses. Figure 5.4 shows the contribution of tensile forces to stability for a typical soil nail wall layout. As illustrated, loads change in magnitude from nail to nail. The location of the maximum load also changes from nail to nail. The intersection of a soil nail with the slip surface determines the length (L_p) of that soil nail that can develop pullout resistance. The stability contribution of the upper soil nail, T_1, is relatively small because L_p is relatively small for this nail; conversely, the contributions of nails T_2 and T_3 are greater because L_p is greater for them. The maximum load in Nail 1 at the slip surface is T_{1max} as shown in Figure 5.4.

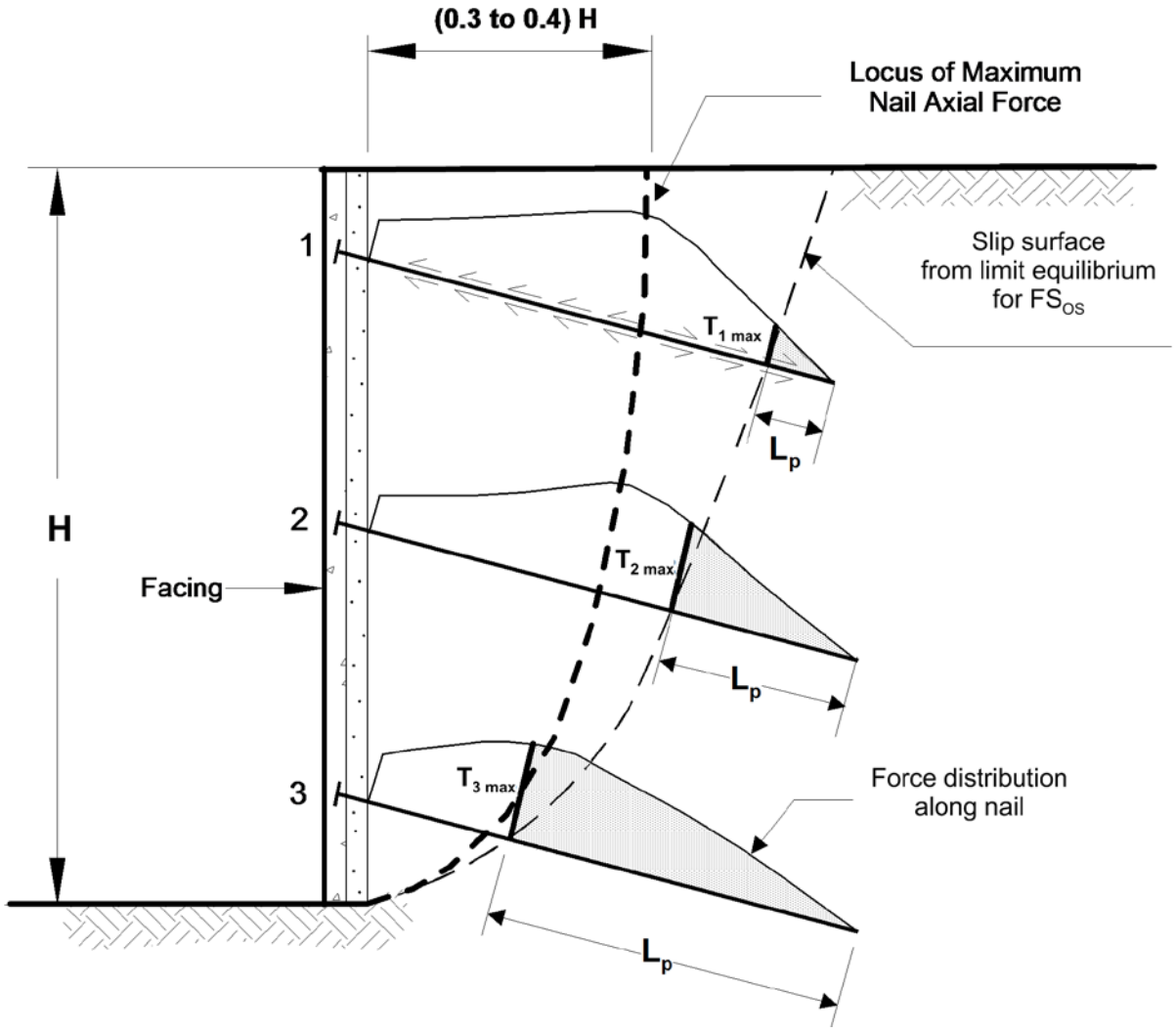


Figure 5.4: Illustration. Location of maximum tensile forces in soil nails. Modified after Byrne et al. (1998).

The maximum tensile force that can be generated in any nail is limited by not only the tensile strength of the tendon, but also by the various resistances of the facing. These conditions

will be discussed in more detail in Section 5.2.1c "Tensile Forces at the Wall Facing." Note that the maximum soil forces, T, must be equal to or less than the resistance, R, which will be presented subsequently.

Figure 5.5 presents the distribution of maximum nail tensile forces as obtained from full-scale instrumented soil nail walls. Strain measurements indicated that the maximum tensile force in the upper portion of these walls occurred approximately between 0.3H and 0.4H behind the wall facing. The maximum tensile force in the lower portion of the walls occurred approximately between 0.15H to 0.2H behind the wall facing. Measured nail forces were normalized with respect to the soil unit weight, γ_s; the soil nail vertical and horizontal spacing, S_V and S_H; the wall height, H; and the calculated active earth pressure coefficient, K_a. These normalized forces, which are shown as a function of the height of the wall, represent the long-term, service soil nail forces. These values exclude other effects on nail forces at the face, such as frost and earthquake loading.

The normalized nail loads shown in Figure 5.5 range from 0.4 to 1.1, averaging approximately 0.75 in the upper two thirds of the wall. The average normalized nail loads increase from approximately 0.5-0.6 near the top, to 0.75-0.85 in the central third, and decrease to about 0.4-0.5 at the lower third, and to 0 at the bottom. These observations are consistent with observations made on experimental walls (Plumelle et al. 1990; Holman and Tuozzolo 2009).

Tensile forces may be estimated using the normalized loads of Figure 5.5 and considering an influence area around individual nails ($S_V \times S_H$). The maximum nail tensile forces in the upper two thirds of the wall vary from approximately $T_{max} = 0.50\ K_a\ \gamma_s\ H\ S_V\ S_H$ to $T_{max} = 1.1\ K_a\ \gamma_s\ H\ S_V\ S_H$. Considering a normalized average of 0.75 in the upper two thirds of the wall, the average maximum tensile force can be estimated at $T_{max} = 0.75\ K_a\ \gamma_s\ H\ S_V\ S_H$. Maximum tensile forces in the lower one third of the wall are approximately 50 percent of the maximum force in the upper part. Briaud and Lim (1997) suggested that the average maximum tensile force of soil nails can be estimated as $T_{max} = 0.65\ K_a\ \gamma_s\ H\ S_V\ S_H$ for the top row of nails and half of this value for lower nail rows. Cohesion is not considered.

The foregoing information illustrates that, for the same contribution area, the average nail force is smaller than a force that would be estimated for the full active lateral earth pressure. This implication is important because the concept of apparent pressure applied over the tributary area $S_V \times S_H$ may be used to check or estimate soil nail forces, which are routinely calculated from limit-equilibrium analyses.

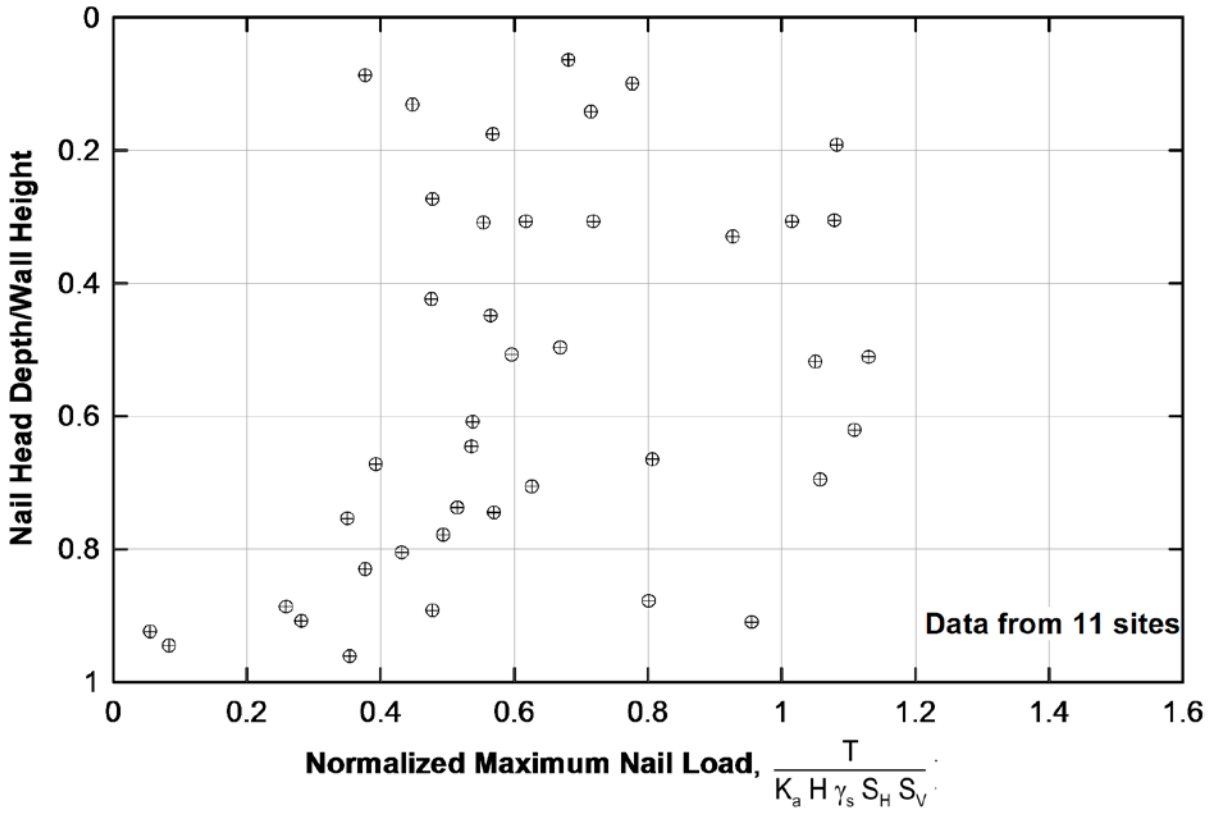


Figure 5.5: Chart. Summary of maximum nail tensile forces measured in soil nail walls. From Byrne et al. (1998).

5.2.1c Tensile Forces at the Wall Facing

The nail tensile force at the wall face, T_o (also referred to as the nail head force), is smaller than forces along the soil nail. Figure 5.6 presents the distribution of working (i.e., in-service, not ultimate) normalized tensile nail forces measured at the facing of instrumented soil nail walls. These forces exclude the effect of frost and other factors. The general distribution of the forces at the facing is comparable to that of the maximum nail forces shown in Figure 5.5. The normalized values of T_o vary from approximately 0.6 to 1.0. The normalized nail head tensile forces in the upper two thirds of the wall vary from approximately 0.40 to 0.70. The average tensile force in the nail head ranges from approximately $T_o = 0.50\ K_a\ \gamma_s\ H\ S_V\ S_H$ to $T_o = 0.60\ K_a\ \gamma_s\ H\ S_V\ S_H$.

These observations are consistent with observations made in experimental walls in Germany (Gässler and Gudehus 1981) and France (FHWA 1993a), and other instrumented walls (Holman and Tuozzolo 2009). These results showed that soil arching between nails contributes to load re-distribution among nails. Closer nail spacings tend to cause smaller nail forces at the facing than those estimated using simple tributary areas.

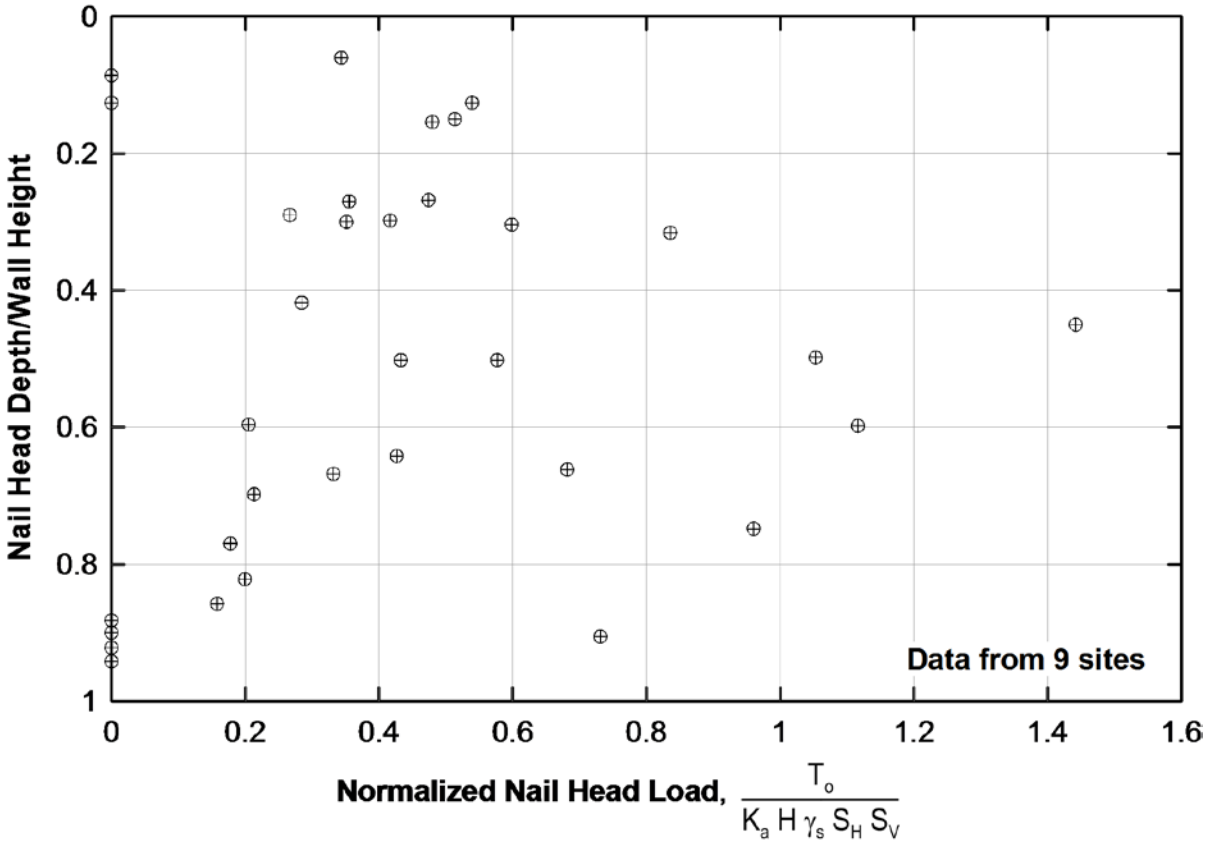


Figure 5.6: Chart. Summary of facing tensile forces measured in soil nail walls. From Byrne et al. (1998).

Clouterre (2002) provided design guidelines based on these results and suggested that the nail head force for working conditions be estimated as:

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

Equation 5.1: Maximum soil nail load at head.

Where S_{max} = maximum of S_V and S_H. For a typical nail head spacing of 5 ft, $T_o \sim 0.7\ T_{max}$ for normal conditions of loading and slope behind the wall. Note that T_o is determined or limited by the minimum of the available facing resistances, as discussed in the next section. The computer program SNAP-2 calculates the force at the facing for each nail based on the Rankine theory of lateral earth pressures, and checks whether this calculated load exceeds the calculated resistance (Siel 2014). However, Equation 5.1 will be preferred in the design procedures presented in Chapter 6.

5.2.1d Considerations for Design

The tensile load in a soil nail is determined or limited by the minimum of the soil nail pullout resistance, R_{PO}, and the tendon tensile resistance, R_T. In addition, at the facing, the nail head

load is determined by the minimum of the facing resistance(s), R_F (Figure 5.7). Nail loads are identified as T_1 ...T_N for nails 1…N, respectively, where N is the total number of rows of nails in a wall. The maximum load that can be achieved in each nail, T_{max_i}, is derived from limit-equilibrium stability analyses and is controlled by the conditions mentioned above. This can be represented as:

$$T_{max_i} = (T_{max} \text{ for nail "i"}) = \text{minimum of} \begin{cases} \text{tensile resistance of nail "i" } (R_{Ti}) \\ \text{facing resistance of nail "i" } (R_{Fi}) \\ \text{pullout resistance of nail "i" } (R_{POi}) \end{cases}$$

Equation 5.2: Maximum load in a soil nail.

If $R_{PO} < R_T$ and R_F, pullout controls T_{max_i} (Figure 5.7a). If $R_T < R_P$ and R_F, tensile resistance controls (Figure 5.7b). Finally, if $R_F < R_T$ and R_P, facing resistance controls T_o of soil nail "i."

Limit-equilibrium stability analyses, however, have limitations in estimating accurately the maximum loads in soil nails because those programs cannot reproduce the actual, complex wall deformation and load mobilization interaction existing in this top-down construction system. In addition, the loads estimated with limit-equilibrium stability analyses can vary significantly depending on the selected soil nail layout (i.e., soil nail length, vertical spacing, and inclination over the height of the wall). Two different soil nail layouts would appear to be equivalent when limit-equilibrium stability analyses of each provide the same overall factor of safety. However, the maximum soil nail loads estimated for each layout can vary greatly. Additional discussions and guidance will be provided in Chapter 6.

In most design situations, all nails have the same tensile resistance, and the same facing resistance. Therefore, $R_{Ti} = R_T$ and $R_{Fi} = R_F$. However, the developed pullout resistance is different in each nail because, even if the bond strength and drill hole diameter are the same for all nails, the length that can develop pullout resistance, L_p, is generally different. The location of Point A shown in Figure 5.7 defines the length L_p.

All nails are in practice designed based on T_{max}, which, as mentioned earlier, is the maximum, T_{max_i}, of all nails. Because the most critically loaded nail (or row of nails) is the basis for design, the other nails are over-designed, at least for the critical slip surface. However, these other soil nail rows are important for other slip surfaces and other excavation stages. Although a designer may consider optimizing the design by evaluation T_{max_i} of individual nails, this is rarely done in practice.

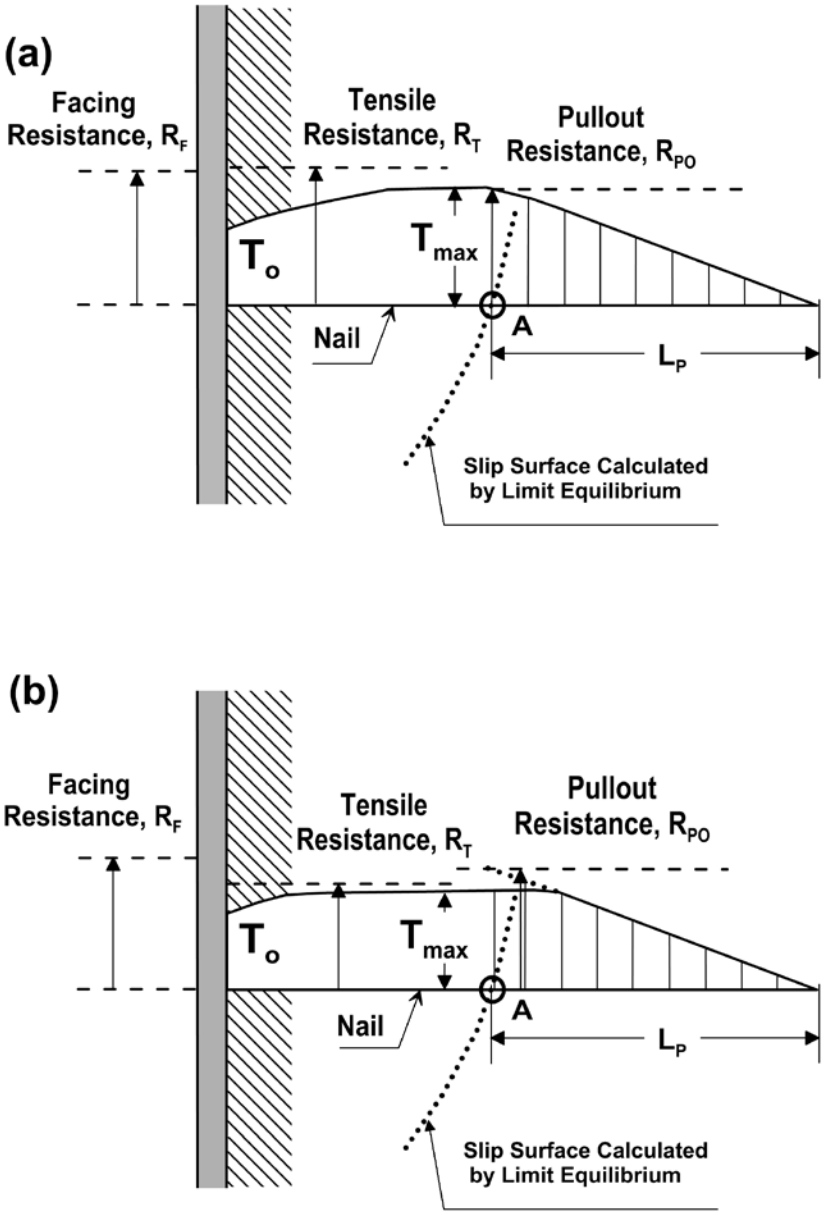


Figure 5.7: Illustration. Limitations to tensile forces in nails: (a) pullout resistance controls, (b) tensile resistance controls.

5.3 Introduction to Design Framework

This manual introduces a framework for the design of soil nail walls that: (i) takes into account factors of safety (FS) as previously used in allowable stress design (ASD); and (ii) integrates LRFD principles. Previously, soil nail design was completed with ASD, and with use of limit equilibrium slope stability analyses. Today, a designer is limited to ASD-based slope stability procedures because LRFD-based slope stability analysis procedures are not available. Therefore, a soil nail wall design must use ASD-based stability calculations to

quantify loads and slip plane geometries, and then take those values and perform limit state checks.

5.3.1 Use of LRFD and ASD Platforms

In this manual, a procedure is presented to allow the user to verify structural resistances in an LRFD format using load factors from AASHTO (2014) and results from ASD-based slope stability analysis. The resistance factors for LRFD presented in this manual were derived to be: (i) related to AASHTO (2014) load factors; (ii) compatible with (but not necessarily equal to) those already presented in AASHTO (2014) for comparable structures; (iii) related to FS included in the previous version of this manual, where applicable, to fit past practice until new research emerges; and (iv) consistent with the minimum required frequency of verification soil load testing, as discussed subsequently.

5.3.2 Allowable Stress Design (ASD)

The design condition in ASD is generally expressed as:

$$\Sigma Q_i \leq R_{all} = \frac{R_n}{FS}$$

Equation 5.3: General design equation in ASD.

Where ΣQ_i is the effect of all combined loads acting on the component under consideration (e.g., soil or structural element) in a given limit state; R_{all} is the allowable stress of the component; R_n, is the ultimate capacity (i.e., when an ultimate stress is considered to be acting over an entire element) of that component; and FS is the factor of safety. FS is the single factor that accounts for all the uncertainty from design, materials, and construction. Allowable values, which represent "working" conditions, are derived from the FS values. Minimum recommended factors of safety for soil nail design are presented in Table 5.1. These values coincide with those presented in the previous edition of this manual.

Table 5.1: Minimum Recommended Factors of Safety for the Design of Soil Nail Walls Using the ASD Method [(1)]

Limit State	Condition	Symbol	Minimum Recomm. Factors of Safety, Static Loads	Minimum Recomm. Factors of Safety, Seismic Loads
Overall	Overall Stability	FS_{OS}	1.5[(2)]	1.1[(6)]
Overall	Short Term Condition, Excavation	FS_{OS}	1.25-1.33[(3)]	NA
Overall	Basal Heave	FS_{BH}	2.0[(4)], 2.5[(5)]	2.3[(5)]
Strength – Geotechnical	Pullout Resistance	FS_{PO}	2.0	1.5
Strength – Geotechnical	Lateral Sliding	FS_{LS}	1.5	1.1
Strength – Structural	Tendon Tensile Strength (Grades 60 and 75)	FS_{T}	1.8	1.35
Strength – Structural	Tendon Tensile Strength (Grades 95 and 150)	FS_{T}	2.0	1.50
Strength – Structural	Facing Flexural	FS_{FF}	1.5	1.1
Strength – Structural	Facing Punching Shear	FS_{FP}	1.5	1.1
Strength – Structural	Headed Stud Tensile (A307 Bolt)	FS_{FH}	2.0	1.5
Strength – Structural	Headed Stud Tensile (A325 Bolt)	FS_{FH}	1.7	1.3

Notes: (1) The limit state and symbol nomenclature differ from that presented in the previous version of this manual. Many of these changes reflect the move toward using LRFD terminology as presented in AASHTO (2014).

(2) For non-critical, permanent structures, some Owners may accept a design for static loads and long-term conditions with $FS_{OS} = 1.35$ when uncertainty is considered to be limited due to the availability of sufficient geotechnical information and successful local experience on soil nailing.

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

(4) This factor of safety for basal heave is applicable to permanent walls for short-term conditions.

(5) This factor of safety for basal heave is applicable to permanent walls for long-term conditions.

(6) The minimum FS_{OS} for seismic overall stability should be 1.0, when horizontal seismic coefficients are used and these were derived from estimated, allowable seismic deformations.

5.3.3 Load and Resistance Factor Design (LRFD)

Transportation structures and substructures are analyzed and designed today based on an LRFD platform. ASD slope stability results must be checked with LRFD-based procedures, including verification that all limit states are not exceeded. The LRFD platform employs both load factors and resistance factors for each element to account for uncertainties. The LRFD condition is generally expressed as:

$$\phi R_n \geq \sum_{i=1}^{N} \gamma_i \eta_i Q_i$$

Equation 5.4: General design equation in LRFD.

Where:

R_n = nominal resistance of a structural component selected for a given limit state

ϕ = the resistance factor related to R_n

Q_i = generic load (or effect)

γ_i = load factor associated with Q_i

η_i = load-modification factor relating to ductility, redundancy, or operational classification (equal to 1.0 for soil nail walls)

i = 1…N and refers to the various loads/effects in that limit state

Nominal resistances include the yield strength of tendons, the pullout resistance of soil nails, the shear strength of soils, and various resistances of the facing. For consistency, the reader should note that the term strength (more typically used in ASD platforms) is equivalent to the term nominal resistance (more typically used in the LRFD platform). The nominal resistance of soils and other natural materials to be used in an LRFD platform are mean values. Soil resistances are obtained using the field/laboratory methods presented in Chapter 4. Load factors (γ_i) are defined in AASHTO (2014); however, resistance factors for soil nail walls are not contained in the current AASHTO (2014). Load factors and recommended resistance factors are presented in subsequent sections, which discuss each mechanism and limit state.

The objective of the LRFD methodology is to have a factored resistance greater than the factored load. The term capacity-to-demand ratio, CDR, which can be used to quantify the ratio of the factored resistance to the factored load, is defined as follows:

$$CDR = \frac{\phi \quad R_n}{\sum_{i=I}^{N} \gamma_i \, Q_i} \geq 1.0$$

Equation 5.5: Capacity-to-demand ratio.

5.3.4 Calibration of Resistance Factors

Various methods can be used to calibrate resistance factors. The most advanced calibration methods are those that use reliable statistical data, employ adequate statistical methods, and result in acceptable levels of reliability across the system. However, several limitations exist in obtaining consistent and complete databases for soil nails, which are noted in NCHRP Report 701 (Lazarte 2011). A calibration of the resistance factor for soil nail pullout was obtained in that research using a database of load testing for soil nails. Shortcomings of the load tests were identified, including data inconsistencies, incomplete or lack of information about soil nail installation procedures, and poor load test documentation. Most importantly, various load tests did not apply enough force to achieve the expected maximum pullout resistance of the test nails. With these limitations, only a preliminary reliability-based set of pullout resistance factors were obtained.

The resistance factors presented in this manual were back-calibrated from the safety factors provided in Table 5.1. The basis of calibration for individual resistance factors will be presented when each limit state is discussed in subsequent sections. This approach allows LRFD verifications, or designs, of soil nail walls based on existing ASD slope stability analysis procedures.

The resistance factors presented in this manual are valid for a minimum frequency of load testing as part of a performance monitoring program, as discussed in detail in Chapter 9. Specifically, resistance factors for nail pullout resistance are valid as long as verification and proof testing is conducted with the following minimum frequency. Proof load tests must be conducted during construction on a minimum of 5 percent of the total production nails that are installed. The number of verification load tests is project-specific, depending on various conditions. More verification tests should be conducted for: (i) a larger number of different soil types existing behind the wall; (ii) a higher variability of soil property even if there is a single soil type; and (iii) larger and longer walls, which would tend to encounter more variable conditions. Further details of soil nail load testing are presented in Chapter 9.

It is acknowledged that the approach of using ASD-based computer programs for overall stability and employing resistance factors calibrated from safety factors is an interim provision. One limitation in the current approach is that different load factors cannot be

applied to different load components in ASD-based computer programs. Satisfactory computational methodologies and the availability of complete databases are lacking, but will be available in the future to fully apply compatible LRFD methods to current limit-equilibrium analysis computations.

5.4 Load Combinations and Load Factors

5.4.1 Load Combinations

Structural design requires considering the simultaneous occurrence of various load types in load combinations. The load combinations to be considered for soil nail design in highway projects in an LRFD framework generally include:

- Service Limit State I
- Strength Limit State I
- Extreme-Event Limit State I (involving earthquake loads)

The above combinations are consistent with those used for earth-retaining structures in an LRFD framework, as presented in Articles 3.11.6.1 and 3.11.6.2 of AASHTO (2014).

5.4.2 Load Factors

The load combinations listed above include permanent, transient, and extreme-event loads. These loads are assigned load factors as presented in Table 3.4.1-1 "Load Combinations and Load Factors" and Table 3.4.1-2 "Load Factors for Permanent Loads" of Section 3 "Loads and Load Factors" of AASHTO (2014). These load factors are summarized below.

Permanent loads for earth-retaining structures may include:

- Dead loads from structural components/nonstructural attachments (DC)
- Dead load of wearing surfaces and utilities (DW), when supporting a bridge abutment or the roadway
- Horizontal earth pressure load (EH)
- Earth surcharge load (ES)
- Vertical pressures caused by the dead load from earth fill (EV)

Loads EH and EV are considered in this manual differently than they are dealt with in the design of other types of earth-retaining structures because of the computational limitations in software dedicated to soil nail design. These limitations are discussed in Section 5.6 "Overall Stability of Soil Nail Walls."

Transient loads may involve vehicular live loads (LL), live load surcharge (LS), construction equipment, and other live surcharge, when the walls support a bridge abutment or the roadway. They may also include construction equipment in the analysis of intermediate construction stages. In the case of a wall supporting a bridge abutment, transient loads originating in the bridge superstructure may be transferred to the abutment (see Article 3.11.6.4 in AASHTO 2014), and these loads may need to be considered in the design of the soil nail wall. Design of soil nail walls supporting a bridge is not within the scope of this manual. However, note that if the soil nail wall at an abutment settles, this effect may need to be considered in the bridge design. Extreme-event loads for soil nail walls comprise earthquake loading (EQ), as discussed in Section 6.8 "Step 9: Seismic Design."

Table 5.2 presents a summary of load factors (γ) to be used in LRFD load combinations for earth-retaining systems (AASHTO 2014). Only the factors for permanent loads and for extreme-event loads will be used in the LRFD verifications presented in this manual.

Table 5.2: Load Factors and Load Combinations Based on AASHTO (2014)

Limit State	Permanent Loads	Transient Loads	Extreme-Event Loads
Strength I	γ_p (1)	$\gamma = 1.75$	NA
Extreme Event I	γ_p (1)	NA	$\gamma = 1.00$
Service I	$\gamma_p = 1.00$	$\gamma = 1.00$	$\gamma = 1.00$

Note: (1) Load factors for permanent loads (γ_p) for these limit states are presented in Table 5.3.

Table 5.3 presents load factors for permanent loads (γ_p) in earth-supporting structures. In general, load factors for permanent loads for strength and extreme-event limit states must be selected as a function of the permanent load type being considered and whether the permanent load has a destabilizing or stabilizing effect (AASHTO 2014, Article 3.4.1). When the load effect tends to destabilize, $\gamma_p > 1.0$, maximum values are used; when the effect is stabilizing, $\gamma_p \leq 1.0$, minimum values are used (Table 5.3).

However, because of the shortcomings of ASD-based computer programs in dealing with load factors, only the load factor for the maximum effect of vertical earth loads (γ_{EV}) will be considered in this manual to conduct LRFD verifications for static loads. This selection is justified by results of stability analysis of model soil nail walls. These results suggest that the effect of the vertical earth loads is larger than that of horizontal earth loads. As a result, γ_{EV} =1.35 (per AASHTO 2014) is used for LRFD verification of various limit states in this

manual. This approach has limitations but is an interim provision to allow performing LRFD verifications.

Table 5.3: Load Factors, γ_p, for Permanent Loads

Load Type	Maximum Load Factor (1)	Minimum Load Factor (2)
DC: Dead loads	1.25	0.90
DW: Loads from wearing surfaces and utilities	1.50	0.65
EH: Horizontal earth pressure (active condition)	1.50	0.90
EH: Horizontal earth pressure (at-rest condition)	1.35	0.90
EV: Vertical earth pressure (overall stability)	1.00	N/A
EV: Vertical earth pressure (ret. walls and abutments)	1.35 (3)	1.00
ES: Earth surcharge	1.50	0.75

Notes: (1) When permanent load has a destabilizing effect.
(2) When permanent load has a stabilizing effect.
(3) This load factor is used in this manual primarily for static loads.

Load factors in service limit states are $\gamma = 1.0$ for all loads (AASHTO 2014). As discussed below, both overall stability using limit-equilibrium analyses (see Section 5.6) and deformation/settlement verifications (see Section 5.9) use a Service I Load Combination; therefore, unfactored loads must be used in these verifications.

Note that the need to treat overall stability as a service limit state stems from the present lack of analytical procedures that would allow using fully compatible LRFD methods in limit-equilibrium analysis computations. A simplistic manner to accommodate different load factors applied to loads other than soil self-weight (for example, surcharge or traffic loads) in an LRFD approach, using an ASD-based stability program, is to enter the values of these loads as corrected values in these programs. An adjusted load value would be the original load value times γ/γ_{EV}, where γ is the load factor associated to that load component.

5.5 Limit States

The following checks must be considered for service, strength, and extreme limit states in the design of soil nail walls:

5.5.1 Overall Stability

- Internal stability (with stability contribution from nails) (Figure 5.8a)
- General global stability (with no stability contribution from nails) (Figure 5.8b)
- Global stability – basal heave (Figure 5.8c)

5.5.2 Strength Limit States

- Geotechnical limit states:
    - Lateral sliding stability (Figure 5.8d)[1]
    - Nail pullout (Figure 5.8e)
- Structural limit states:
    - Nail in tension (Figure 5.8f)
    - Facing: bending (or flexure) (Figure 5.8g)
    - Facing: punching shear (Figure 5.8h)
    - Facing: headed stud in tension (Figure 5.8i)

5.5.3 Extreme-Limit States

- Earthquake loads

5.5.4 Service Limit States (Deformations)

- Lateral displacement
- Wall settlement

[1] Sliding stability is in general a special case of global stability limit state. It is considered a geotechnical limit state to illustrate the use of LRFD with load factors ≥ 1.0.

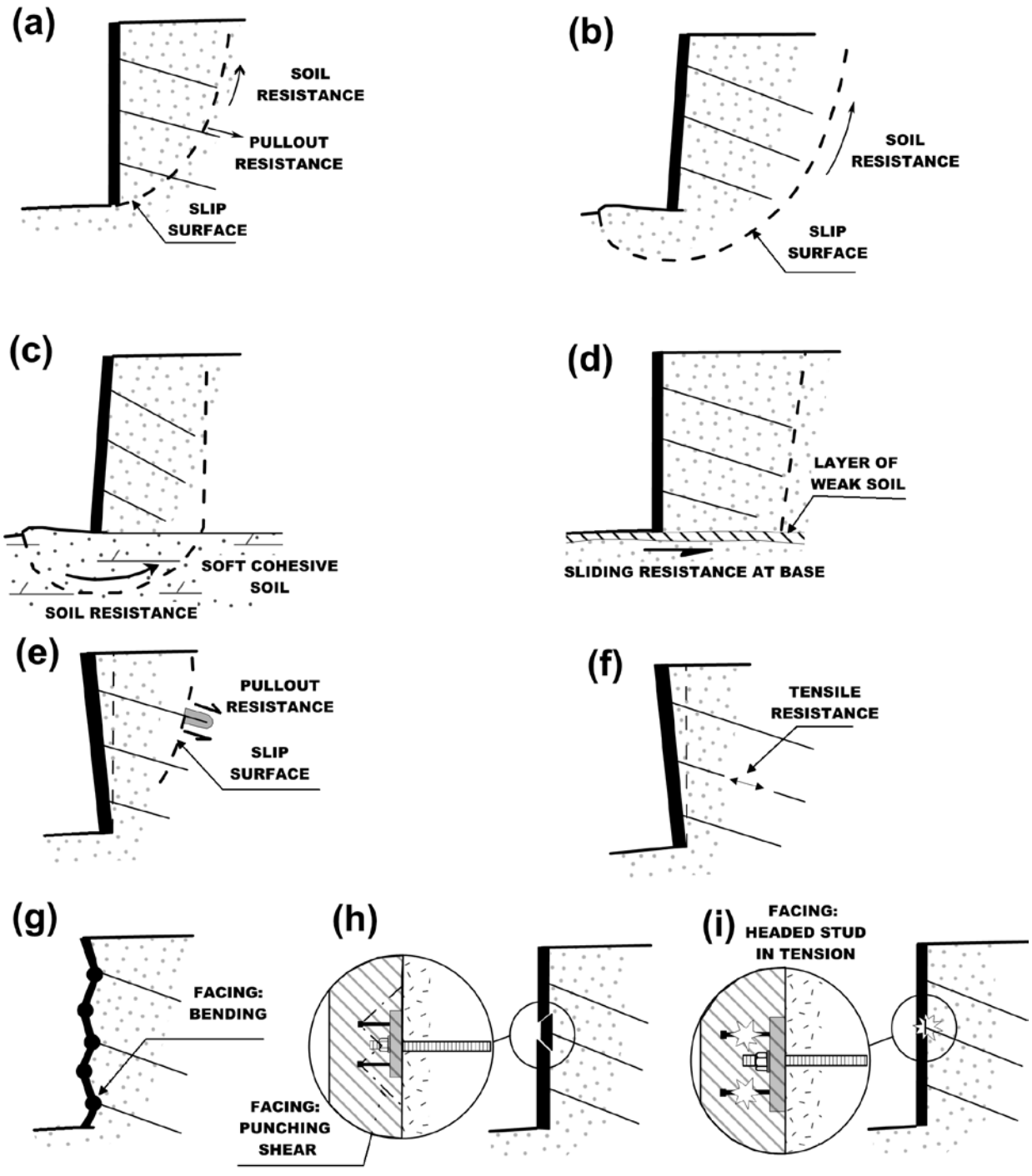


Figure 5.8: Illustration. Potential limit states in soil nail walls: (a) internal stability (slip surface intersecting soil and nails); (b) global stability (slip surface not intersecting nails); (c) global stability: basal heave; (d) geotechnical strength: lateral sliding; (e) geotechnical strength: pullout; (f) structural strength: nail in tension; (g) facing structural strength: bending; (h) facing structural strength: punching shear; (i) facing structural strength: headed stud in tension.

5.6 Overall Stability of Soil Nail Walls

5.6.1 Overview

Overall stability for soil nail walls needs to be analyzed and checked for internal and global stability. Internal stability involves contribution from both the soil and soil nails; and global stability (as defined herein) involves no contribution from the soil nails. Overall stability may also be considered for special conditions such as basal heave stability.

5.6.2 Internal Stability

Internal stability of soil nail walls must be considered for potential slip surfaces extending through the soil and some or all of the nails. The soil strength can be modeled as frictional, cohesive, or both, depending on the soil type and on whether the loading rate generates drained or undrained shear resistance. If the slip surface intersects some or all of the nails, the mobilized soil nail pullout resistance behind the slip surface also contributes to stability. The nail tensile resistance is treated separately as a structural strength limit state, as discussed in Section 5.8.1 "Nail Tensile Resistance."

Internal stability of soil nail walls is routinely evaluated using two-dimensional, limit-equilibrium analyses as employed in conventional ASD-based slope stability analyses. Limit-equilibrium analyses consider several potential slip surfaces and provide an FS for each. Typically, numerous analyses are performed to find the lowest FS value. The lowest calculated value of FS must be equal to or greater than the minimum acceptable FS established for the structure. The analyses also provide a critical slip surface corresponding to the lowest FS value, and the tensile forces for each of the nails supporting the wall under consideration.

Soil nail design procedures to evaluate internal stability consider various shapes for the slip surfaces, including: (i) planar (Sheahan and Oral 2002); (ii) bi-linear (Stocker et al. 1979; Caltrans 1991); (iii) parabolic (Shen et al. 1981); (iv) log spiral (Christopher et al. 1990); and (v) circular (Golder 1993; Siel 2014). The safety factors obtained for different slip surfaces generally do not significantly affect the calculated safety factor (Long et al. 1990), as long as these surfaces exit in front of the wall and the ground surface behind the wall at approximately the same locations.

Internal stability analyses for soil nail walls are most commonly performed using computer programs specifically developed for their design. These programs give engineers greater ability to quickly analyze multiple design scenarios. Programs such as SNAIL (Caltrans 1991), SNAILZ (Caltrans 2007) and GoldNail (Golder 1993) have been used for several years. FHWA released the software Soil Nail Analysis Program 2 (SNAP-2), (Siel 2014).

Of these programs, only SNAP-2 checks the strengths of the facing. Any of these programs can be used to meet the guidance presented in this manual provided the user understands how parameters are used in the particular software. Other available programs include the updated SNAIL from Caltrans and SNAIL Plus, a commercial software. Any of the programs mentioned above may be used to analyze and design soil nail walls.

General slope stability computer programs, such as Slope/W (Geostudio 2012), XSTABL (Softpile 2013) or Slide 6.0 (Rocscience 2013), can model multi-level, generic reinforcement, which would be similar to soil nails. Hence, these general programs can also assess internal stability of soil nail walls. However, their use can be cumbersome because these programs do not evaluate different design scenarios with ease, and the user generally needs to make separate calculations. Dedicated soil nail wall software is generally more convenient to use than general slope-stability software. However, the use of general slope-stability software may be needed to verify global stability, which cannot be correctly addressed by most dedicated soil nail software.

Minimum safety factors, such as those recommended in Table 5.1, are entered into ASD-based computer programs for soil nail wall analysis. The programs then calculate an overall stability FS. For example, SNAILZ and GoldNail use allowable bond stresses, which are calculated as the bond strength (or equivalently a nominal bond unit resistance) times a reduction factor selected by the user. Note that the reduction factor coincides with the inverse of the factor of safety for pullout, $1/FS_{PO}$. Preliminary dimensions and the yield strength of the tendon must be entered as a "trial design" in most computer runs.

Users of this manual should realize that limit-equilibrium programs have limitations in reproducing actual distributions of soil nail forces over the height of the wall. Observations of soil nail force distributions made in full-scale instrumented soil nail walls (see Section 5.2.1b Maximum Tensile Force Distribution) indicate that the largest soil nail forces occur most commonly in the upper two thirds of the wall. This trend is the result of the top-down construction of soil nail walls and the mobilization of soil nail forces in the staged construction. Conversely, limit-equilibrium programs tend to predict larger soil nail forces near the bottom of the wall (at least for homogeneous subsurface conditions and uniform soil nail lengths), as these programs only deal with equilibrium requirements but ignore kinematic constraints and deformations. While these limit-equilibrium programs would yield safe designs meeting the minimum stability criteria (see Section 6.6.3 "Resistance Factors for Overall Stability"), their computational limitation would tend to yield nail force distributions that might not be optimal for reducing wall deformations near the top of the wall, or will be larger than loads actually developed in the soil nails.

A simple approach to counter this artifact of limit-equilibrium methods is to select the soil nails in the lower half of the wall with lengths increasingly shorter toward the bottom. This approach will be described in detail in Section 6.3.3e "Distribution of Soil Nail Lengths in Elevation."

5.6.3 Global Stability

Global stability of soil nail walls is also routinely evaluated using two-dimensional, limit-equilibrium analyses with a conventional ASD framework. Global stability is more adequately performed using general slope stability computer programs than with dedicated soil nail design software. Most of the dedicated soil nail design software has limitations to performing adequate global stability analyses. For example, although slip surfaces in SNAIL and SNAILZ extend under the toe of the wall, these programs use a simple approach based on passive resistance to estimate the soil resistance under the toe. Slip surfaces only pass through the toe of the wall in GoldNail. On the other hand, slip surfaces in SNAP-2 can extend above, at, or under the toe of the wall; and the soil resistance under the toe is computed using appropriate limit-equilibrium based formulation. For the purpose of demonstrating the design procedure of this manual, the design example in Appendix C has been prepared using SNAP-2.

The minimum safety factors for global stability analysis are similar to those for internal stability analysis. Special cases of global stability will be considered in Section 5.6.6 "Basal Heave" and Section 5.7.3 "Lateral Sliding Stability."

5.6.4 Idealized Example

This section presents a manual check of internal stability to illustrate the interaction of forces in a typical soil nail wall design and how these forces may be considered. Note, however, that manual calculations of stability for complete design of soil nail walls are rarely performed in practice. Manual calculation can be used to spot-check computer results. Checks in LRFD and ASD are presented.

Figure 5.9 shows a generic, single-nail wall of height H as a simplified trial design. While the soil nail has an inclination, i, from the horizontal, the slip surface is selected as a plane with inclination, α, from the horizontal to define a wedge, and with an angle β with respect to the nail. T is mobilized up to its nominal value of pullout resistance, which develops in the nail behind the slip surface. Loads consist of an external surcharge, Q, and the weight of the wedge, W. R_S is the resultant of the nominal soil resistance along the slip surface. For simplicity, it is assumed that the soil is purely frictional. Although, in general, separate load factors need to be applied for each load component, the same load factor is assumed for Q and W in this example for simplicity.

The resisting and driving force components acting parallel to the slip plane are then considered. The component of T parallel to the slip plane is considered to be a driving force and is not included at this point. A factored resistance can be derived from the effect caused by the components of W, Q, and T that are normal to the plane as follows:

$$\text{Factored resistance} = \phi N \tan\varphi_s = \phi \left[(W+Q)\cos\alpha + T\sin\beta \right] \tan\varphi_s$$

Equation 5.6: Factored resistance.

Where:

ϕ = resistance factor for soil strength

N = normal force on slip surface

φ_s = ultimate friction angle of soil

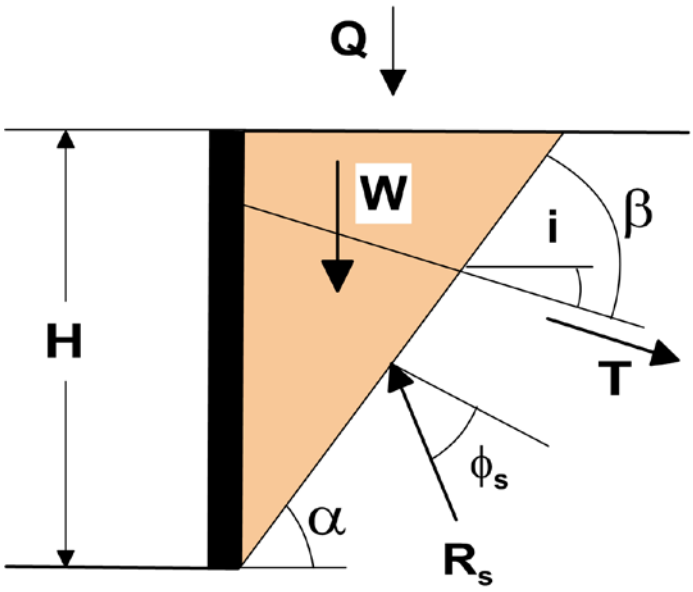


Figure 5.9: Illustration. Simple wedge for overall stability verification.

The component of T that is parallel to the slip surface is now considered a driving force. Load factors are applied to the components of (W+Q) but not to T, for simplicity of this illustration. However, the relevant component of T can also be modified with a load factor. In this example, the factored driving force is:

$$\text{Factored driving force} = \gamma (W+Q)\sin\alpha - T\cos\beta$$

Equation 5.7: Factored driving force.

For equilibrium along the slip plane, the factored driving force must be equal to the factored resistance. Therefore, T can be calculated as:

$$T = \frac{(W+Q)\left[\frac{\gamma}{\phi}\sin\alpha - \cos\alpha\tan\varphi_s\right]}{\frac{\cos\beta}{\phi} + \sin\beta\tan\varphi_s}$$

Equation 5.8: Mobilized pullout resistance under equilibrium.

The length and section of the nail, as well as the characteristics of the facing, may be developed once T is obtained.

A similar equation can be formulated using an ASD approach, where the frictional resistance of soil normal force would be reduced by FS, to N tan φ_s/FS in Eq. 5.6, and unfactored loads (i.e., $\gamma = 1.0$) would be considered in Eq. 5.7. Similarly, T can be calculated as:

$$T = \frac{(W+Q)\left[FS\sin\alpha - \cos\alpha\tan\varphi_s\right]}{FS\cos\beta + \sin\beta\tan\varphi_s}$$

Equation 5.9: Mobilized pullout resistance using ASD approach.

Overall stability of soil nail walls should be investigated at Service I Load Combination, and using an appropriate resistance factor. The load factor, γ, at Service I limit state is equal to 1.0 for permanent loads. A comparison between Eqs. 5-8 and 5-9 indicates that ϕ is approximately equal to 1/FS if the value of γ is equal to 1.0.

Slip surfaces are in reality more complex than the planar surface of the simple wedge example above. The simple expressions could be extended to more complex slip surfaces, and to include other loads and other assumptions. However, it is impractical to perform overall stability of soil nail walls by hand. Computer programs, such as those discussed earlier, can perform these calculations far more efficiently. The manner in which a resistance factor can be back-calibrated from the factor of safety, once the load factor has been defined, is still applicable for computer-based overall stability analysis.

5.6.5 Resistance Factors for Overall Stability

Resistance factors for overall stability are presented in Table 5.4. They were back-calibrated from ASD-based safety factors reproduced in Table 5.1 and rounded up to the nearest 0.05 value.

Table 5.4: Resistance Factors for Overall Stability

Condition	Symbol	Resistance Factor
Static	ϕ_{OS}	0.65
Seismic loading	ϕ_{OS}	0.90

These resistance factors are valid regardless of whether the slip surface intersects the nails. They provide a design consistent with past ASD practice and in alignment with AASHTO (2014).

5.6.6 Basal Heave

5.6.6a Methodology

The potential for basal heave should be evaluated when soft, fine-grained soils are present beneath a soil nail wall excavation (Figure 5.8c). If the excavation depth is excessive for existing soft soil conditions, unbalanced loads may cause the bottom of the excavation to heave and ultimately result in basal movement. Basal heave is uncommon in soil nail walls because these structures are not often constructed over (or in) soft, fine-grained soils. However, this scenario is considered to allow a feasibility evaluation for soil nail walls when there is a potential for basal heave at a site.

The commentary of Article 11.9.3 of AASHTO (2014) provides guidance for assessing the stability of anchored walls. This information may also be applied to other top-down construction, including soil nail walls. In addition, commentary of Section 11.9.5.2 of AASHTO (2014) indicates that heave may occur under a wall constructed in/over soft clays having an undrained shear strength, $S_u < 0.15\ \gamma_s' H$ (where γ_s' = buoyant unit weight of the soil, and H = wall height), and that such a wall should be embedded to mitigate heave. Soil nail walls constructed over soft clays are more susceptible to basal heave than other top-down retaining systems with embedded segments because most soil nails in a wall are not embedded below the wall toe. An alternative support-of-excavation system may be necessary if the potential for basal heave is found to be significant. A methodology is presented in this section to evaluate the potential for instability of the base of an excavation.

If the entire excavation is in fine-grained soils, the nominal soil resistance, R_S, at the base of the excavation develops along a composite slip surface (Figure 5.10). The width B'_e is controlled by the excavation width, B_e (ideally with no influence from soil nails), or whether a deposit of stiff soil exists within a depth D_B under the excavation. The value of R_S is given in units of force per linear foot of wall and can be calculated using the following expressions:

If $B'_e < D_B$ (Figure 5.10a):

$$R_S = S_u N_c \, B'_e$$

Equation 5.10: Nominal soil resistance.

If $B'_e \geq D_B$ (Figure 5.10b):

$$R_S = S_u N_c \, D_B$$

Equation 5.11: Nominal soil resistance.

Where:

S_u = undrained shear strength of fine-grained soil below the soil nail wall

N_c = factor for cohesion (see Figure 5.11)

B'_e = width of soil block causing heave ~0.707 B_e

D_B = depth to top of stiff soil deposit under excavation

Because the width of the soil block may extend behind all nails for wide excavations, the contribution of soil nails is disregarded in Eqs. 5-10 and 5-11. If the excavation is not sufficiently wide, nails may contribute some shear resistance; however, soil nails are particularly inefficient in resisting this limit state.

N_c is a function of the ratios H/B_e and B_e/L_e, where H, L_e, and B_e are the excavation height, length, and width, respectively (Figure 5.11). For very long soil nail walls ($B_e/L_e \sim 0$), supporting very wide excavations, ($H/B_e \sim 0$), it can be conservatively assumed that $N_c = 5.14$. The total pressure, q, at the base of the soil block is:

$$q = \frac{\sum Q_i}{B'_e} = \frac{B'_e \; H \gamma_s + Q - S_{ul} H}{B'_e}$$

Equation 5.12: Total pressure at the base of the soil block.

Where:

Q_i = total load at the base of the soil block

γ_s = unit weight of the soil behind the wall

Q = external load (dead or live load, depending on the conditions)

S_{u1} = undrained shear strength of the fine-grained soil behind the soil block

Note that S_{u1} was not affected by a load or resistance factors as a simplification. This assumption appears to be neutral. Also note that if $B'_e \geq D_B$, D_B must replace B'_e in Eq. 5.12.

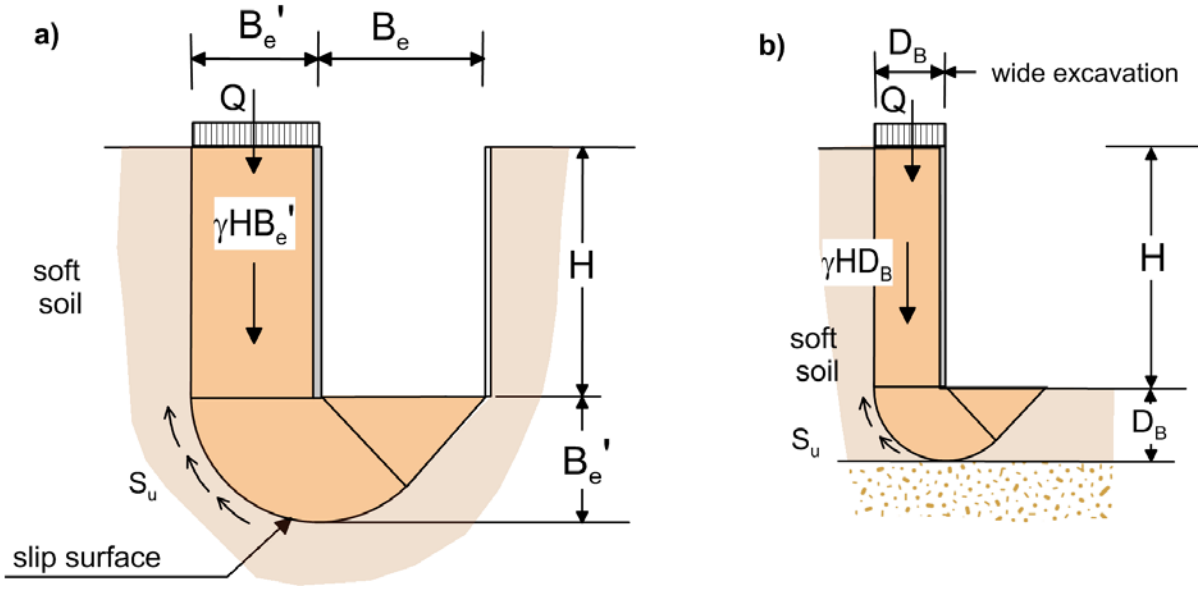


Figure 5.10: Illustration. Basal heave for: (a) deposit of soft-fine grained soils, and (b) deposit of soft fine-grained soil underlain by stiff layer. Modified after Terzaghi et al. (1996) and Sabatini et al. (1999).

The stability for basal heave can then be expressed as:

$$\phi_{BH}\, 5.14\, S_u \geq \gamma_{EV}\, q$$

Equation 5.13: Basal heave stability.

With ϕ_{BH} = resistance factor, and γ_{EV} = 1.35 from Table 5.3. If the shear resistance of the soil behind the wall is disregarded as is typical in Terzaghi theory, basal heave will not happen if:

$$\phi_{BH}\, 5.14 S_u \geq \gamma_{EV} \left[H\gamma_s - \frac{Q}{B_e'} \right]$$

Equation 5.14: Basal heave stability with external load.

D_B must replace B'_e in Eq. 5.14 if $B'_e \geq D_B$. Furthermore, if Q = 0, the excavation will be stable against basal heave if:

$$\phi_{BH}\, 5.14\, S_u \geq \gamma_{EV} \left(H\gamma_s \right)$$

Equation 5.15: Simplified expression for basal heave stability.

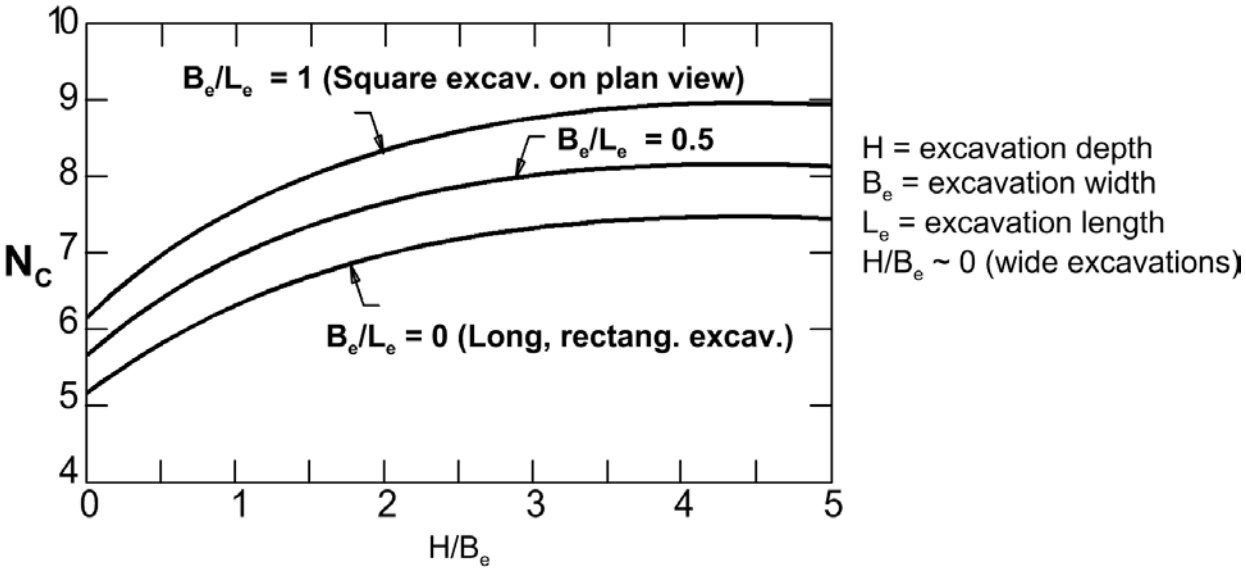


Figure 5.11: Graph. Bearing capacity factor (N_c) for determining basal heave. Modified after Terzaghi et al. (1996) and Sabatini et al. (1999).

5.6.6b Resistance Factors for Basal Heave

The resistance factor for basal heave can be calibrated by fitting to past ASD-based safety factors. Eq. 5.15 can be rewritten as:

$$FS = \frac{5.14\, S_u}{\left(H\gamma_s \right)} = \frac{\gamma_{EV}}{\phi_{BH}}$$

Equation 5.16: Factor of safety against basal heave.

Where FS is the safety factor against basal heave. Minimum factors of safety of 2.0 and 2.5 for short-term and long-term conditions, respectively, are shown in Table 5.1. Recommended resistance factors are listed in Table 5.5.

Table 5.5: Resistance Factors for Basal Heave

Condition	Symbol	Resistance Factor
Permanent wall, short-term conditions	ϕ_{BH}	0.65
Permanent wall, long-term conditions	ϕ_{BH}	0.50

5.6.6c Lateral Squeeze

Soil nail walls forming part of a bridge abutment may overlie soft soils, and thus be subject to an unbalanced embankment load behind the wall. Lateral squeeze should be considered a service limit state, where deformations, not strengths, control. However, lateral squeeze is not presented herein since it is considered a variation of basal heave because the two mechanisms are controlled by relatively low strength and high compressibility of fine-grained soils.

5.7 Geotechnical Strength Limit States

5.7.1 Overview

Geotechnical strength limit states occur when the soil resistance is fully mobilized along a potential slip surface. The two geotechnical strength limit states described in this section include nail pullout and lateral sliding of the reinforced soil mass.

5.7.2 Nail Pullout Resistance

The resistance factors for overall stability are back-calibrated from ASD-based safety factors. These values are presented in Table 5.6.

Table 5.6: Resistance Factors for Pullout

Condition	Symbol	Resistance Factor
Static	ϕ_{PO}	0.65
Seismic loading	ϕ_{PO}	0.65

As mentioned earlier, the resistance factors presented above are valid as long as bond strengths are verified by field load testing conducted with the minimum frequency described in Section 5.3.4 and Chapter 9 "Construction Inspection and Performance Monitoring."

5.7.3 Lateral Sliding Stability

5.7.3a Methodology

Lateral sliding is not present in most soil nail designs. However, lateral sliding can be considered to be a special case of global stability limit state, which may arise when a weak soil layer underlies the block of reinforced soil (Figure 5.8d), and when the soil nails to do not contribute to stability.

Lateral sliding stability can be readily verified using a general slope-stability program where the reinforced soil can be modeled such that the slip surfaces are forced to extend as a plane along the layer of weak soil. Note that computer programs for soil nail wall design generally cannot model slip surfaces with horizontal segments.

As an alternative to the computational methods described above, this limit state can be considered similarly to the sliding of gravity-retaining structures, where simplified, manual procedures can be used. While the simplified procedure presented herein is used to check the lateral sliding for horizontal basal slip surfaces, it can also illustrate the use of the LRFD in soil nail design. The soil nail wall is modeled as a rigid block against which a generally non-horizontal force generated by the earth pressure is applied (Figure 5.12). The earth pressure is assumed to act beyond the longest soil nail so that no contribution from the nails is considered. In addition, a permanent earth surcharge (Q_{ES}) is assumed to act behind the reinforced soil. The horizontal effect of Q_{ES} is defined as ES.

The case is treated as strength limit (Strength I). Lateral earth loads can be estimated using Coulomb or Rankine lateral-earth pressure theory. The reader is referred to Article 3.11.5.1 of AASHTO (2014) for additional information on analysis procedures.

The horizontal component of the earth load (EH) is $P_a \cos \delta$, where P_a is the lateral earth thrust, and δ is the friction angle at the surface between the block and the retained soil

defining the inclination of P_a from the horizontal (Figure 5.12). Angle δ would typically be the internal friction angle of the soil, although it may be assumed equal to the inclination (β_{eq}) of the ground surface behind the wall for simplicity and if safe to do so.

Lateral sliding stability is verified using the following expression:

$$\phi_{LS} R_s \geq \gamma_{EH} P_a \cos\delta + \gamma_{ES} ES$$

Equation 5.17: Lateral sliding stability.

Where:

ϕ_{LS} = resistance factor for lateral sliding

R_s = nominal soil resistance per unit width acting at the base of the soil block

γ_{EH} = maximum load factor for horizontal earth loads = 1.5 (from Table 5.3 for Strength I, and Table 5.3 for permanent loads, active conditions)

γ_{ES} = maximum load factor for horizontal effect of permanent surcharge = 1.50 (from Table 5.3 for Strength I)

ES = horizontal component of the permanent earth surcharge

δ = friction angle between the reinforced block and the retained soil, which may be considered equal to β_{eq}, as defined below

R_s can be estimated as:

$$R_s = c_b B_L + (W + P_a \sin\delta) \tan\varphi'_{fb}$$

Equation 5.18: Nominal resistance of soil at the base of soil block.

Where:

c_b = effective soil cohesion strength along the base

B_L = length of the horizontal slip surface where c_b is effectively acting

W = weight of reinforced soil mass

β_{eq} = equivalent backslope angle for broken or "infinite" slopes:

$\beta_{eq} = \tan^{-1}(\Delta H/2H)$ for broken slopes (see Fig. 5.12)

$\beta_{eq} = \beta$ for infinite slopes

β = backslope angle from horizontal

φ'_{fb} = effective friction angle at base (remolded or residual values may be applicable)

Note that W and $P_a \sin \delta$ are implicitly affected by the minimum value of $\gamma_{EV} = 1.00$ (load factor for the vertical component of earth pressure). The effect of Q_{ES} on the vertical pressure at the base of the reinforced soil block is not shown here as a simplification.

The lateral earth thrust is the active force per unit width:

$$P_a = \frac{\gamma_s H_1^2}{2} K_a$$

Equation 5.19: Lateral earth thrust.

Where:

γ_s = unit weight of the soil behind the wall

H_1 = effective height over which the earth pressure acts

K_a = active earth pressure coefficient

and:

$$H_1 = H + (B_L - H \tan\alpha) \tan \beta_{eq}$$

Equation 5.20: Effective height over which earth pressure acts.

Where:

α = face batter angle

If the slope has no breaks within a horizontal distance 2H from the wall (Figure 5.12), the slope is considered "infinite" and $\beta_{eq} = \beta$. If the slope has a break within a distance 2H from the wall, the slope has an equivalent inclination $\beta_{eq} = \tan^{-1} (\Delta H/2H)$, where ΔH is the slope rise over a distance 2H.

The active earth pressure coefficient, K_a, can be obtained using Coulomb or Rankine theory for cohesionless soil (assuming no cohesion in the soil behind the block).

According to the Coulomb theory:

$$K_a = \frac{\sin^2(\theta + \varphi_f)}{\sin^2\theta \sin(\theta - \delta) \left[1 + \sqrt{\frac{\sin(\varphi_f + \delta)\sin(\varphi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2}$$

Equation 5.21: Active earth pressure coefficient according to Coulomb theory.

Where:

φ_f = friction angle of soil retained behind the soil nail reinforced mass

θ = inclination of the back of the wall face from horizontal (i.e., $\theta = \alpha + 90^\circ$)

Eq. 5.21 derived from the Coulomb theory is more general than equations based on the Rankine theory. For walls with $\alpha < 8^\circ$, according to the Rankine theory:

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi_f}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi_f}} \right]$$

Equation 5.22: Active earth pressure coefficient according to Rankine theory.

For the case of a vertical, or near vertical, wall ($\alpha = 0$) and horizontal ground behind the wall ($\beta = 0$), and assuming no friction at the soil interface ($\delta = 0$), the Rankine theory provides:

$$K_a = \tan^2\left(45 - \frac{\varphi_f}{2}\right)$$

Equation 5.23: Active earth pressure coefficient (vertical wall, horizontal backfill, and no friction at the soil interface).

In cases where the sliding plane occurs along stiff, fissured clays, the fully softened shear strength should be selected for φ_f. If it has been determined that significant shearing and displacement have taken place along the base, the residual strength should be selected.

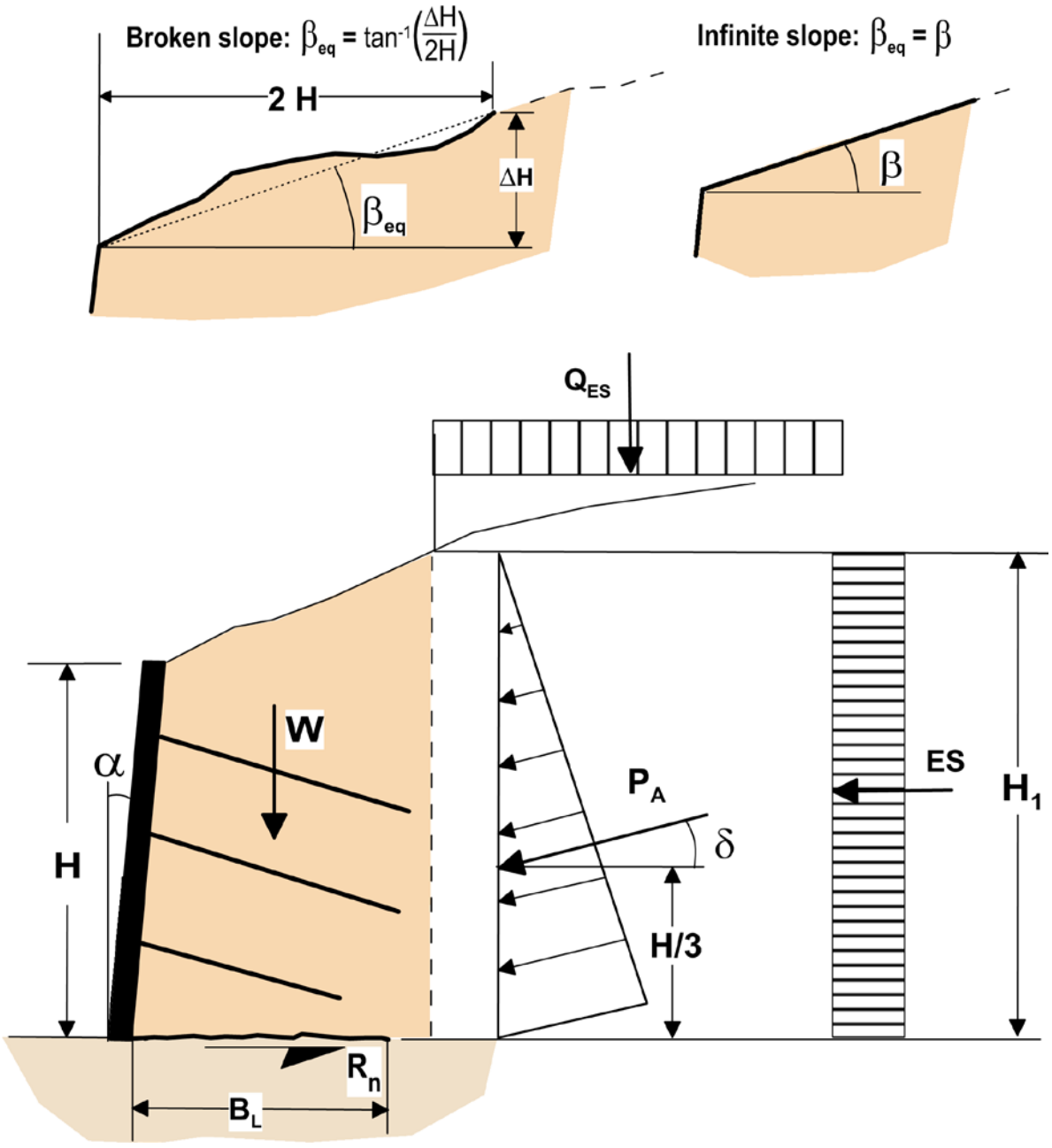


Figure 5.12: Illustration. Lateral sliding of a soil nail wall.

5.7.3b Resistance Factors for Lateral Sliding

The resistance factors for lateral sliding are back-calibrated from ASD-based safety factors, considering that the maximum load factors for both EH and ES are 1.50, and that the minimum load factor for EV is 1.0 (from Table 5.3). The resistance factors for lateral sliding are presented in Table 5.7.

Table 5.7: Resistance Factors for Lateral Sliding

Condition	Symbol	Resistance Factor
Static	ϕ_{LS}	1.00
Seismic loading	ϕ_{LS}	0.90

5.8 Structural Strength Limit States

5.8.1 Nail Tensile Resistance

Resistance factors for tensile resistance were back-calibrated using load factor values $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading, to match ASD-based designs. These resistance factors are presented in Table 5.8.

Table 5.8: Resistance Factors for Tendon in Tension

Condition	Case	Symbol	Resistance Factor
Static	Tendon Grades 60/75 (ASTM A615)	ϕ_T	0.75
Static	Tendon Grade 95/150 (ASTM A722)	ϕ_T	0.65
Seismic loading	Tendon Grades 60/75 (ASTM A615)	ϕ_T	0.75
Seismic loading	Tendon Grade 95/150 (ASTM A722)	ϕ_T	0.65

5.8.2 Overview of Facing Limit States

The strength limit state modes at the facing-nail head connection are presented in Figure 5.13, and are as follows:

- Bending/Flexure Limit State: This limit state takes place when the bending moment on the facing exceeds its resistance. This mode needs to be considered separately for initial and final facings. The contribution of the initial facing is disregarded in the design of the final facing (Figure 5.13b).
- Punching Shear Limit State: This limit state occurs in the facing around the nails and must be evaluated separately for the initial and final facings (Figure 5.13c and d).

- Headed Stud Tensile Limit State: This limit state occurs when the tensile resistance of the headed studs is exceeded. This limit state only applies to the final facing (Figure 5.13e).

An adequate design must ensure that the facing and other nail head elements provide sufficient resistance for each of these potential limit states, in excess of the maximum nail head tensile force (T_o). Appropriate dimensions, strength, and reinforcement of the facing and suitable nail head hardware (bearing plate, nut, and headed studs) must be provided to achieve the design resistances and to perform adequate LRFD verifications for these potential limit states. The following sections describe the methodology to design the facing and other related elements, and to perform verifications for each of the limit states.

5.8.3 Facing Flexural Strength Limit State

5.8.3a Introduction

Lateral earth pressures acting against the facing cause flexural/bending moments. The facing is considered a continuous two-way slab, while the nails support the slab for the purposes of its design and the strength limit state verification. A flexural/bending limit state is reached when the lateral earth pressures increase, deform the facing, and ultimately produce a yielding mechanism (i.e., plastic hinges in cross section that are actually yield lines across the slab, Figure 5.14).

Bending moments produce tension on the outside of the facing between nails (positive moments in conventional structural engineering practice), and tension on the inside of the facing around the nails (negative moments). Because the facing is reinforced in two directions, it acts as a two-way slab, whereby bending moments are generated around a horizontal axis (vertical moments, m_V, Figure 5.13b) and a vertical axis (horizontal moments, m_H). The maximum bending moments that can be achieved depend on: the horizontal and vertical nail spacing (S_H and S_V), the thicknesses of the initial and final facing (h_i and h_f, respectively), the layout and type of the facing reinforcement, the strengths of the facing reinforcement (WWM and additional rebar, if present), and the shotcrete/concrete resistance (Seible 1996).

5.8.3b Bending Mechanisms

Bending resistances are estimated using conventional formulas for reinforced concrete design, as shown below. Figure 5.15 shows schematically a generalized section of reinforced shotcrete/concrete of thickness h_i for the initial facing (or thickness h_f for the final facing), and with reinforcement located in the middle of the section. The reinforcement could consist of WWM alone or WWM with rebar. The thickness h_i (or h_f) would represent the height of

an equivalent beam sustaining pure bending, in accordance with reinforced concrete design. Figure 5.15 shows the moment per unit length (m_v) acting around a horizontal axis. The figure also shows the forces generated by this moment: a compressive force per unit width (force "c" divided by "a," the effective width of the area in compression), and a tensile force per unit length (t). The distance from the outer edge in compression on the left to the centroid of the reinforcement in tension is "d." The distances d_i and d_f are half of h_i and h_f, for the initial and final facings, respectively.

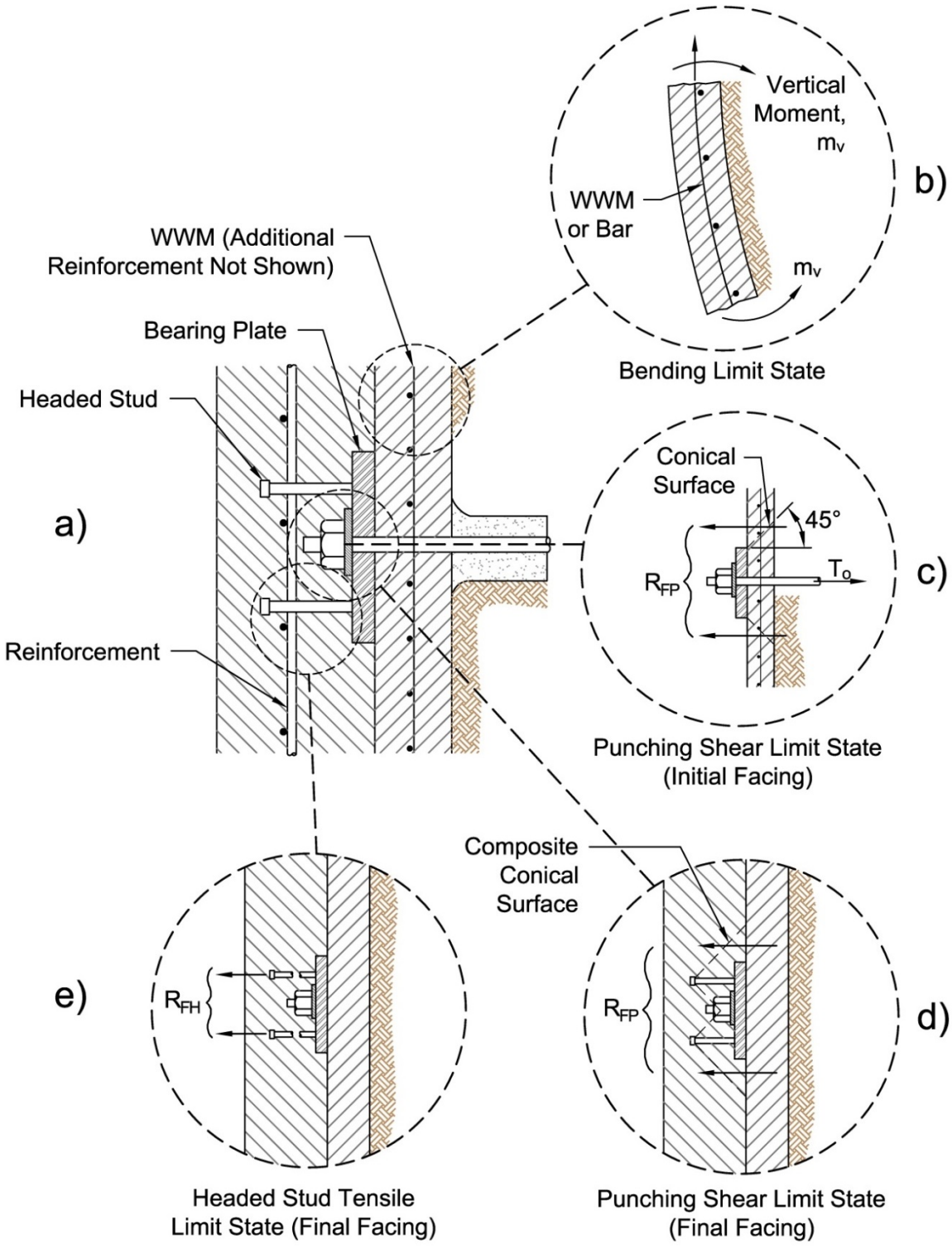


Figure 5.13: Illustration. Limit states in soil nail wall facings. (a) typical facing layout; (b) bending of a facing section; (c) punching shear in initial facing; (d) punching shear in final facing; (e) tension of headed studs. Modified after Lazarte (2011).

The nominal compressive resistance is calculated as follows:

$$c = 0.85 \times f'_c \times a$$

Equation 5.24: Nominal compressive resistance.

Where:

f'_c = shotcrete/concrete compressive strength

a = extent of area in compression across section (stress is assumed to be uniform for simplification)

The horizontal spacing of soil nails (S_H) can be considered as the section width in Figure 5.15 (along a horizontal direction and normal to the page); however, because forces and moments are expressed per unit length, the width is not relevant in these equations.

The nominal tensile resistance (force per unit length) is:

$$t = f_{y_W} \times a_{s_eq}$$

Equation 5.25: Nominal tensile resistance.

Where:

f_{y_W} = yield strength of WWM (assumed to be the primary reinforcement)

a_{s_eq} = equivalent cross sectional area per unit width (accounts for any type of reinforcement placed in the section, as defined below)

The force, t, can be the result of the combined use of WWM and rebar in the section, and it can be expressed more generally as:

$$t = f_{y_W} \times a_{s_W} + f_{y_R} \times \frac{A_{s_R}}{S} =$$

$$= \left(a_{s_W} + \frac{f_{y_R}}{f_{y_W}} \times \frac{A_{s_R}}{S} \right) f_{y_W} = \left(a_{s_W} + \frac{A'_{s_R}}{S} \right) \times f_{y_W}$$

Equation 5.26: Bending force in initial facing.

Where:

f_{y_W} = yield strength of WWM

a_{s_W} = cross sectional area per unit width of WWM (cases for vertical and horizontal reinforcement will be defined below)

f_{y_R} = yield strength of rebar

A_{s_R} = cross sectional area of rebar

S = S_H or S_V, depending on the case

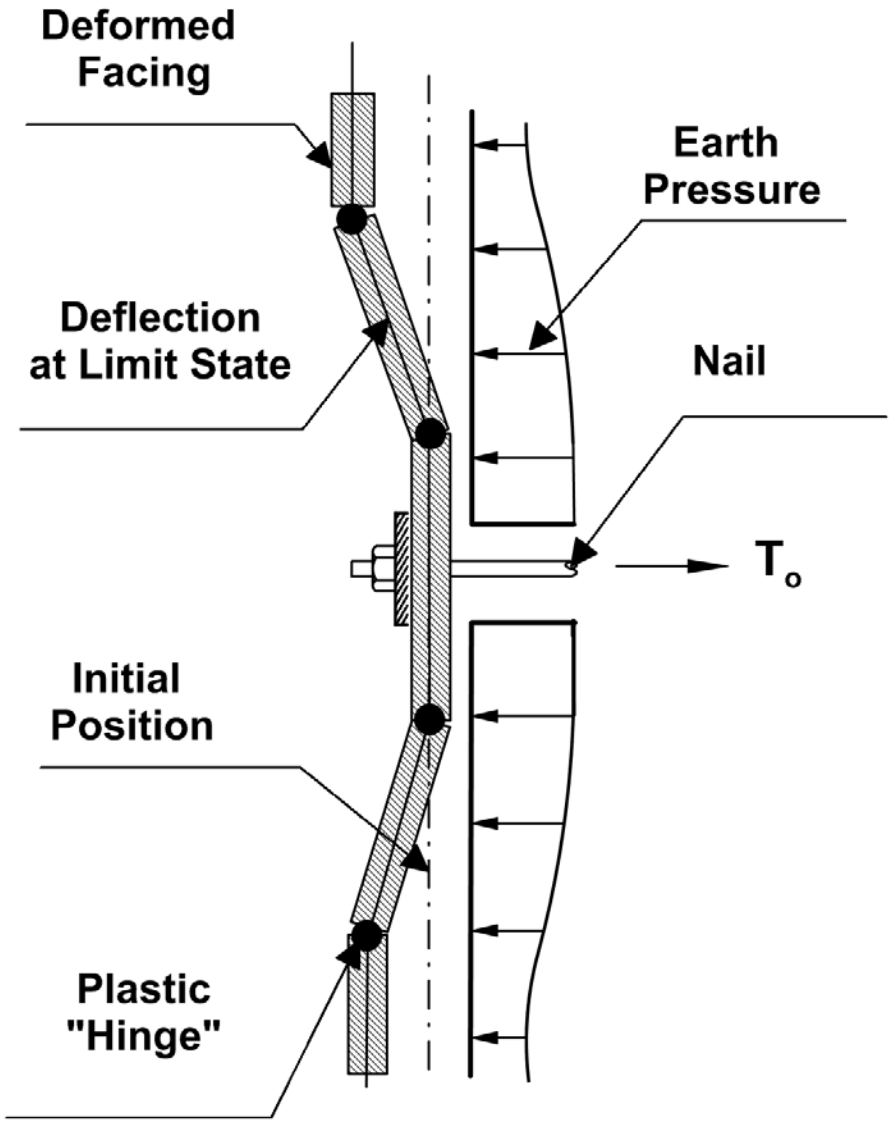


Figure 5.14: Illustration. Bending mechanism and nail force in facing. Modified after Lazarte (2011).

A general equivalent cross-sectional area of rebar (A'_{s_R}) is defined below for the case when the yield strengths of the rebar and WWM are different.

$$A'_{s_R} = \left(\frac{f_{y_R}}{f_{y_W}} \right) \times \frac{A_{s_R}}{S}$$

Equation 5.27: Equivalent cross-sectional area of rebar.

The total equivalent area per unit of length (a_{s_eq}) is defined as:

$$a_{s_eq} = a_{s_W} + \frac{A'_{s_R}}{S}$$

Equation 5.28: Total equivalent area per unit length.

The consideration of force equilibrium (c = t) and moment equilibrium ($m_v = t \times z$) leads to an expression for calculating the maximum bending moment. In these equations, z is the moment arm between a and t and is defined as $z = d - a/2$. Considering that $h_i = d_i/2$ for the initial facing, and after replacing terms for t and z, the following expression is obtained:

$$m_v = a_{S_eq} f_{y_W} \left(\frac{h_i}{2} - \frac{a_{S_eq} f_{y_W}}{1.7 f'_c} \right)$$

Equation 5.29: Bending moment.

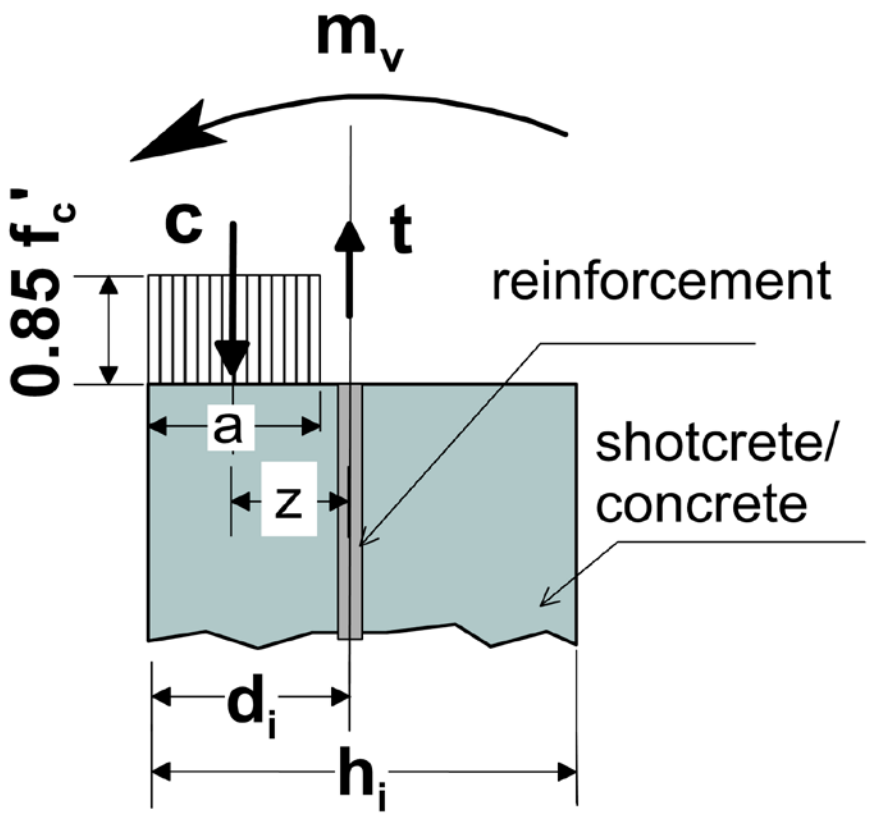


Figure 5.15: Illustration. Section showing bending mechanism in initial facing.

It is assumed that all reinforcement achieves yield strength in this equation. It has also been assumed that the centroid of all reinforcement in either direction is at the center of the section, as a reasonable estimation of a more complex geometry. Eq. 5.29 will be used in Chapter 6 to estimate the flexural resistance of the facing.

The bending moments in the horizontal and vertical directions must be evaluated around the nail heads and in the span between nails because these are the locations where the maximum magnitudes of bending moment occur. Consequently, the reinforcement for each of these locations and directions must be evaluated. As described in the Chapter 3 section "Shotcrete Reinforcement," it is common to place both WWM and rebar around the nails heads, and only WWM extending over the span between nails in the initial facing. It is also common to place only rebar in the final facing, where the rebar is placed in both the vertical and horizontal directions.

5.8.3c Resistance Factors for Flexure at Facing

Resistance factors for flexure resistance at the facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.9.

Table 5.9: Resistance Factors for Flexure Resistance at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FF}	0.90
Seismic loading	Initial and final facing	ϕ_{FF}	0.90

5.8.4 Punching Shear Strength Limit State

Resistance factors for punching at the facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.10.

Table 5.10: Resistance Factors for Punching Shear at Facing

Condition	Case	Symbol	Resistance Factor
Static	Initial and final facing	ϕ_{FP}	0.90
Seismic loading	Initial and final facing	ϕ_{FP}	0.90

5.8.5 Headed Stud in Tension in Final Facing

Resistance factors for headed stud in tension punching in the final facing are back-calibrated for a load factor of $\gamma_{EV} = 1.35$ for static conditions and $\gamma_{EV} = 1.00$ for seismic loading to match ASD-based designs. These values are presented in Table 5.11.

Table 5.11: Resistance Factors for Headed Stud in Tension in Final Facings

Condition	Case	Symbol	Resistance Factor
Static	A307 Steel Bolt (1)	ϕ_{FH}	0.70
Static	A325 Steel Bolt	ϕ_{FH}	0.80
Seismic loading	A307 Steel Bolt (1)	ϕ_{FH}	0.65
Seismic loading	A325 Steel Bolt	ϕ_{FH}	0.75

Note: (1) This is equivalent to AWS D1.1 Type B studs, with f_y = 60 ksi.

5.9 Service Limit States (Deformations)

5.9.1 Introduction

The maximum lateral and vertical movements of the wall must be estimated as part of the design of soil nail walls. These deformations must be less than tolerable deformation limits for the wall. Design considerations for the movements are described below.

5.9.2 Soil Nail Wall Displacements

A soil nail wall is a passive reinforcement system and some deformation of the wall should be expected during its service life. As discussed at the beginning of this chapter, a soil nail wall and the soil behind it tend to deform outwards during and after construction. Most of the movement of soil nail walls supporting predominantly granular soils occurs during or shortly after excavation. Post-construction deformation is generally small unless there is deterioration of the wall, its foundation, or the retained soil. Creep of retained fine-grained soils may take place with the consequent increase of tensile forces in the soil nails and deflection of the wall after construction.

The maximum horizontal displacements commonly occur at the top of the wall. Settlements of the wall are generally small, and are on the same order of magnitude as the horizontal movements measured at the top of the wall. Lateral deflections are generally larger for taller walls and for increasing nail spacing, nail inclination, and surcharge magnitude. Conversely, lateral deflections tend to be smaller for an increase of: (i) wall batter, (ii) soil stiffness, or (iii) a decrease in soil nail spacing. Vertical displacements are also affected by similar factors. They are generally largest near the facing and commonly smaller than lateral deflections measured at the top of the wall.

The maximum long-term horizontal and vertical displacements at the top of the wall, δ_h and δ_v (Figure 5.20), can be estimated as follows (Clouterre 1991):

$$\delta_v \approx \delta_h = \left(\frac{\delta_h}{H}\right)_i \times H$$

Equation 5.30: Maximum long-term horizontal and vertical displacement at the top of wall.

Where $(\delta_h/H)_i$ is a ratio that depends on soil conditions, as indicated in Table 5.12.

This equation is valid as long as: (i) the ratio $L/H \geq 0.7$, where L = soil nail length, H = wall height; (ii) the surcharge is negligible; and (iii) $FS_{OS} \geq 1.5$ for overall stability.

Table 5.12: Values of $(\delta_h/H)_i$ and C as Functions of Soil Conditions

Variable	Weathered Rock and Stiff Soil	Sandy Soil	Fine-Grained Soil
$(\delta_h/H)_i$	1/1000	1/500	1/333
C	0.8	1.25	1.5

Note: Modified from Clouterre (1993) and Byrne et al. (1998).

Ground deformation can be significant up to a distance (D_{DEF}) behind the wall (Figure 5.16), which can be estimated as:

$$D_{DEF} = C\,(1 - \tan\alpha)\,H$$

Equation 5.31: Distance of significant soil deformation behind the wall.

Where:

α = wall batter angle

C = soil-dependent coefficient included in Table 5.12

If critical or sensitive structures are located near the soil nail wall, a project-specific assessment of the potential impact of wall movement on these structures is warranted. This assessment can be performed through a variety of available numerical methods, or through the examination of data of similar walls in similar ground conditions, or both.

Horizontal deflections greater than 0.005H during construction should be a cause for concern, as they generally represent an upper limit of acceptable performance. It is possible

to mitigate deformations, when necessary, by using a battered wall, installing longer nails in the top portion of the wall, or increasing the overall steel section of the nails per unit area of wall. It is even possible in some cases to use ground anchors in conjunction with the soil nails.

Post-construction monitoring data from soil nail walls suggest that movements tend to continue after wall construction, sometimes up to 6 months, with the longer duration corresponding to soils with creep potential. Long-term post construction deformations may increase up to 15 percent beyond those observed soon after construction. Additional tensile forces are developed in the nails as a result of this post-construction movement. Fine-grained soils of high-plasticity (approximately PI > 20) and high water contents (such that LI > 0.2) tend to incur deformation for longer periods of time due to their potential for creep.

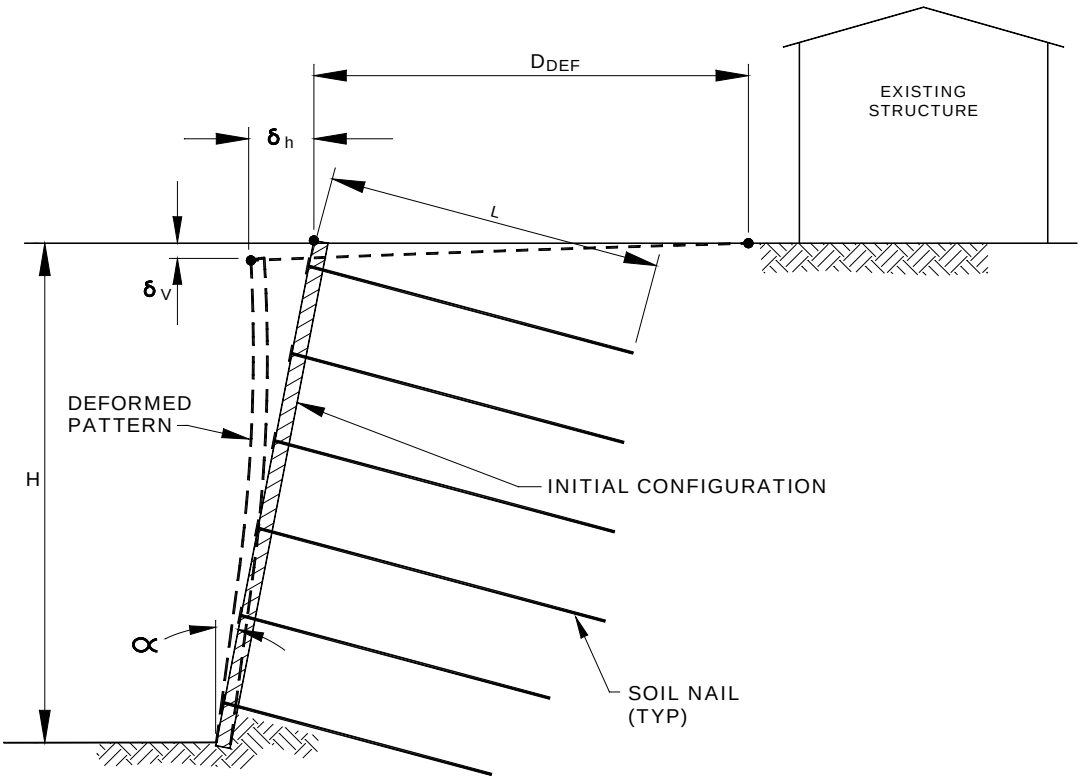


Figure 5.16: Illustration. Deformation of soil nail walls. Modified after Clouterre (1991) and Byrne et al. (1998).

Chapter 6: Design of Soil Nail Walls

6.1 Introduction

6.1.1 Overview

This chapter presents step-by-step procedures for the design of soil nail walls. The principal activities throughout the duration of a soil nail project take place in the following main phases:

- Initial design considerations
- Design
- Pre-construction
- Construction and post-construction

The sequence of the major steps and sub-steps is listed in Tables 6.1 and 6.2. The Design phase is the focus of this chapter. The other three phases are introduced for completeness, but are only briefly discussed in this chapter.

Table 6.1: Main Steps – Initial Design Considerations

Step No.	Description
Step 1	**Project Requirements**
a	Establish project requirements, standards and constraints
b	Establish project performance
c	Assemble preliminary geotechnical information
Step 2	**Subsurface Exploration and Development of Parameters for Design**
a	Plan and conduct subsurface exploration
b	Conduct soil laboratory testing program
c	Establish soil corrosion potential and level of corrosion protection
d	Develop subsurface profiles for analysis
e	Develop soil parameters for design
f	Obtain seismic parameters
g	Conduct a risk analysis
Step 3	**Load Definition**
a	Define unfactored, service loads
b	Select load combinations and load factors

Table 6.2: Main Steps – Design

Step No.	Description
Step 4	**Soil-Nail Configuration and Material Selection**
a	Develop wall layout
b	Develop soil nail cross sections
c	Select soil nail pattern on wall face
d	Evaluate soil nail horizontal splaying
e	Detail corrosion protection
f	Select soil nail type and material properties
Step 5	**Selection of Resistance Factors**
Step 6	**Overall Stability Analyses**
a	Evaluate internal stability
b	Evaluate global stability
c	Evaluate basal heave (if applicable)
d	Evaluate sliding stability (if applicable)
Step 7	**Strength Limit States (Geotechnical and Structural)**
a	Verify pullout resistance
b	Verify sliding stability (if applicable)
c	Verify nail tensile resistance
d	Verify facing bending/flexural resistance
e	Verify facing punching shear resistance
f	Verify facing headed stud resistance
g	Other facing design considerations
Step 8	**Service Limit States (Deformations)**
a	Evaluate wall lateral and vertical displacements
b	Evaluate lateral squeeze (if applicable)
Step 9	**Seismic Design**
a	Select design seismic parameters
b	Adjustment of design seismic coefficients
c	Evaluate overall stability with seismic loads
Step 10	**Drainage and Drainage Details**
a	Evaluate internal drainage
b	Evaluate surface water runoff
c	Develop drainage details
d	Specialty items (if present)
Step 11	**Other Design Considerations**
a	Develop final constructability evaluation
b	Prepare plan for load-testing program
c	Prepare plan for geotechnical monitoring program
Step 12	**Preparation of Construction Drawings and Specifications**

The sequence of work shown in Tables 6.1 and 6.2 is independent from who does the design. The sequence is applicable when an Owner (e.g., typically a public Agency), or the Owner's

consultant, performs the analysis, design and plan preparation for a project in a Design/Bid/Build contract. The sequence of work should be approximately the same if the Owner procures the project as a Design/Build contract. The difference is the proportion of design work performed by the Owner's design team and the Contractor's design team.

6.1.2 Initial Design Considerations

The Initial Design Considerations phase includes establishing the project performance requirements and constraints (Step 1), collecting geotechnical information (Step 2), and defining loads (Step 3).

6.1.2a Step 1 Project Requirements

In this step, the Owner must establish or confirm the project requirements and constraints, confirm the design standards to be used, select expected project performance (depending if structure is critical or not, permanent or not, etc.). In addition, the Owner must collect and assemble preliminary geotechnical information that may exist and is relevant for the project.

6.1.2b Step 2 Subsurface Exploration and Development of Parameters for Design

The Owner performs or commissions the preliminary evaluations and analysis in Step 2 to establish baselines and develop criteria in specifications and to evaluate feasibility and constructability.

Developing a thorough understanding of the geotechnical conditions at the site is of utmost importance. These conditions define the feasibility of soil nailing and impact the design. The detailed and complete characterization of site variability, definition of soil design parameters, and establishing the site corrosion and frost potentials are significant aspects of soil nail wall design that need to be completed before the Design phase is initiated.

Generally, the Owner performs Step 2 of the Initial Design Considerations phase to establish baselines and develop criteria so that the designers (whether working for Owner or Contractor) can evaluate feasibility, develop the core design and review constructability.

The Owner or Owner's consultant also often conducts other activities in the Initial Design Considerations phase. For example, the Owner should select the level of corrosion protection of soil nails, develop preliminary plan layouts, and perform preliminary overall stability and basal heave analyses (the latter, when applicable). In addition, if a sensitive structure exists next to the planned excavation, the Owner is expected to take responsibility for defining the performance criteria and for performing a preliminary deformation evaluation of the planned wall. If the Owner has strict limits for deformation (less than approximately 0.5 in.), the Owner may consider a soldier pile anchor wall or a secant pile

wall. For preliminary evaluation, the designer may need to consult existing publications dealing with the performance of top-down excavation systems (e.g., Clough and O'Rourke 1990).

The required level of corrosion protection for soil nails should be determined based on test results of soil corrosion potential, and following the procedure developed for selecting soil nail corrosion protection levels presented in Chapter 7 for solid bar soil nails and Chapter 10 for HBSNs.

Deformation analyses can be of significant consideration if structures exist adjacent to the planned soil nail wall. Preliminary deformation analyses at this stage will help identify problem areas, and possible measures to limit displacements. Performance requirements must stipulate that these issues be further addressed in final design. Where deformations are of concern, the project documents must include monitoring requirements to verify deformations of the wall during and after construction.

Owners must also conduct a risk evaluation. This evaluation will help identify other potential project risks (e.g., assessment of right-of-way constraints, traffic control and phasing, schedule, access restrictions, subsurface hazards, and seismic exposure), and help select strategies to mitigate these risks. Owners are expected to determine wall face aesthetic requirements and the soil nail wall procurement method in this Design phase. The engineer should consider all apparent aspects that affect constructability, including access, overhead limitations, obstructions, type of soil, drainage, traffic, etc.

6.1.2c Step 3 Load Definition

The engineer must define the loads that act upon the soil nail wall. These loads include permanent, transient, and extreme-events as described in Chapter 5. The engineer should determine load combinations according to AASHTO (2012). For soil nail design in highway projects, the load combinations are generally applicable for the following limit states:

- Strength I
- Service I
- Extreme-Event I (involving earthquake loads)

6.2 Design Phase

6.2.1 Overview

The Design phase involves six main steps, which are discussed below. These are Steps 4 through 9 in Table 6.2. Some steps may need to be repeated until the design is adequate.

The focus of the description is solid bar soil nails, but steps are generally applicable for HBSNs.

6.3 Step 4 Soil Nail Configuration and Material Selection

6.3.1 Introduction

Once the existing geometric constraints are identified, the engineer must establish preliminary wall cross sections, including number of nails, vertical and horizontal nail spacing, and nail inclination and length. However, all of these quantities are expected to be adjusted to some degree as the design progresses. At this stage, the engineer should evaluate corrosion protection requirements and establish the corrosion protection features that meet the selected level of corrosion protection. Details about the adequate level of corrosion protection can be defined at a later stage (see Step 4e). Finally, the engineer must select a soil nail type and its mechanical properties before starting the verification of strength limit states.

6.3.2 Step 4a Develop Wall Layout

Using longitudinal profiles, plan views, cross sections, location of appurtenances (e.g., traffic barriers, utilities, and drainage systems), ROW limits, and other information obtained in Step 1, the engineer determines the wall alignment, height range, and wall length. Constructability issues may include obstructions encountered during a pre-construction site survey, limitations that are specific to the drilling equipment, and potential cost-saving and construction scheduling adjustments.

6.3.3 Step 4b Develop Soil Nail Wall Cross Sections

6.3.3a Wall Batter

The engineer must select the wall batter considering its design, cost, and aesthetic implications. The wall face for a soil nail wall can be vertical or nearly vertical. However, a moderate (up to about 10 degrees) batter reduces the forces transferred to nails and allows the use of shorter nails. A larger batter can contribute to face stability, may eliminate the need for temporary berms in unstable sections, and allow the use of soil nails in not so advantageous soil conditions. However, the savings in shorter nails can be offset partially by the increase in wall face area and excavation volume created with the batter. A mild batter may also contribute to aesthetics, especially around horizontal curves. Truly vertical walls often appear to be leaning inward and may cause discomfort to the public.

6.3.3b Soil Nail Spacing

Soil nails are installed in a grid pattern. The horizontal nail spacing, S_H, is often the same as the vertical nail spacing, S_V (Figure 6.1). Nail spacing in both directions generally ranges from 4 to 6 ft and occasionally up to 6.5 ft, and is routinely selected at 5 ft. The spacing can be checked such that $S_H \times S_V$ is less than approximately 36 to 42 ft^2.

The first row of nails should not be installed deeper than approximately 2 to 3.5 ft from the top edge of the wall to reduce the potential for instability of the upper excavation lift and to reduce cantilever effects on the temporary facing. The lowermost row of nails should be installed about 2 to 3 ft above the base of the excavation. These requirements are the result of the limited ability of the facing to work as a cantilever at the top and bottom of the wall. However, these limits may be adjusted for project-specific conditions, and when based on suitable analysis.

The engineer should consider adopting uniform nail spacing to simplify construction and quality control. However, nail spacing needs to be adjusted near the top and bottom of walls and where the grade changes. It may be difficult to obtain uniform nail spacing along the vertical edges of wall corners, where a wall ends (Figure 6.2a), and around existing utilities or underground structures. It is commonly more practical to install upper nails along one or two rows parallel or sub-parallel to the top of the wall, and install other nails in between, not exceeding a maximum spacing of approximately 6 ft (Figure 6.2a).

A minimum soil nail spacing of approximately 3.5 ft must also be specified so the potential for drilling into previously installed nails due to drilling deviations will be reduced. Further, close spacing of nails may result in stress overlap in the load transfer similar to closely spaced piles. Exceptions to this guidance may arise in areas where concentrated loads (such as those from existing foundations) are applied above or immediately behind the soil nail wall.

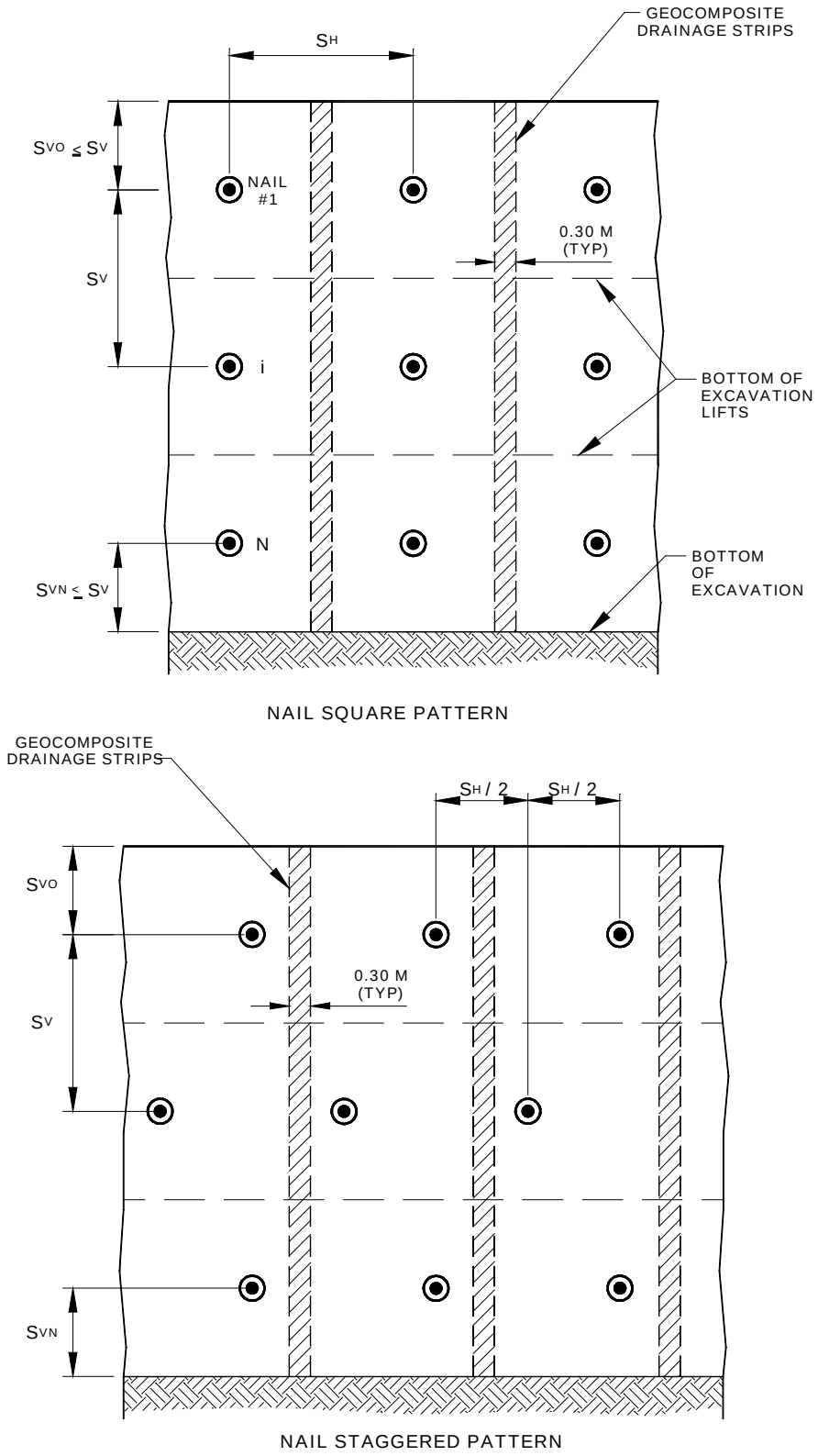


Figure 6.1: Illustration. Soil nail patterns on wall face.

6.3.3c Soil Nail Inclination

Soil nails are installed at 10 to 20 degrees from the horizontal, and most commonly at 15 degrees. The grout can flow at these inclinations from the bottom of the drill hole to the head. Grout generally can fill the hole without leaving air pockets for typical drill-hole dimensions and grout mixes.

Nail inclination angles less than 10 degrees should be avoided to prevent creating voids in the grout and an extended "bird's beak" at the nail head. Voids can reduce the pullout resistance and decrease corrosion protection.

Project conditions may require nails to be installed steeper than 15 degrees. Figure 6.2b illustrates that when utilities or underground structures are too close, the nails need to be installed at varying inclinations around these utilities or structures. The presence of utilities only impacts the upper few rows of nails because utilities are generally shallow. Nails should not be installed above utilities because future excavations for maintenance will likely disturb the wall stability. For example, Nail 1 in Figure 6.2b would need to be shorter than others to avoid reaching the utility pipeline. Note that Nail 2 Figure 6.2b, which is shown as a dashed line, may be compromised, depending on the type of utility if this nail is installed as shown. If layers of competent or highly competent soils exist below and near the base of the wall, some nails could be installed at steeper inclinations to extend into the competent soils and develop a higher pullout resistance. However, nail inclinations that are too steep may cancel out this benefit because the direction of the nail is not optimal for stability. Nails that are steeper than about 40 to 45 degrees may be in compression.

Overhead restrictions in road widening projects under existing bridges can often impose a restriction on the inclination of the first few rows of nails by obstructing the positioning of the drill rig mast. This situation may require installing soil nails with shallow inclinations, or reinforcing the wall facing to allow cantilevers taller than 3.5 ft. Also, operating the drill rig at the bottom of a narrow, deep excavation can require installing nails at inclinations steeper than 15 degrees.

The inclination of individual nail rows can be readily input in soil nail wall design computer programs. Steep soil nails will require a vertical reaction provided by the wall facing that is not calculated by soil nail wall computer programs and should be accounted for during design (see Section 6.10.3 "Strut-Nails").

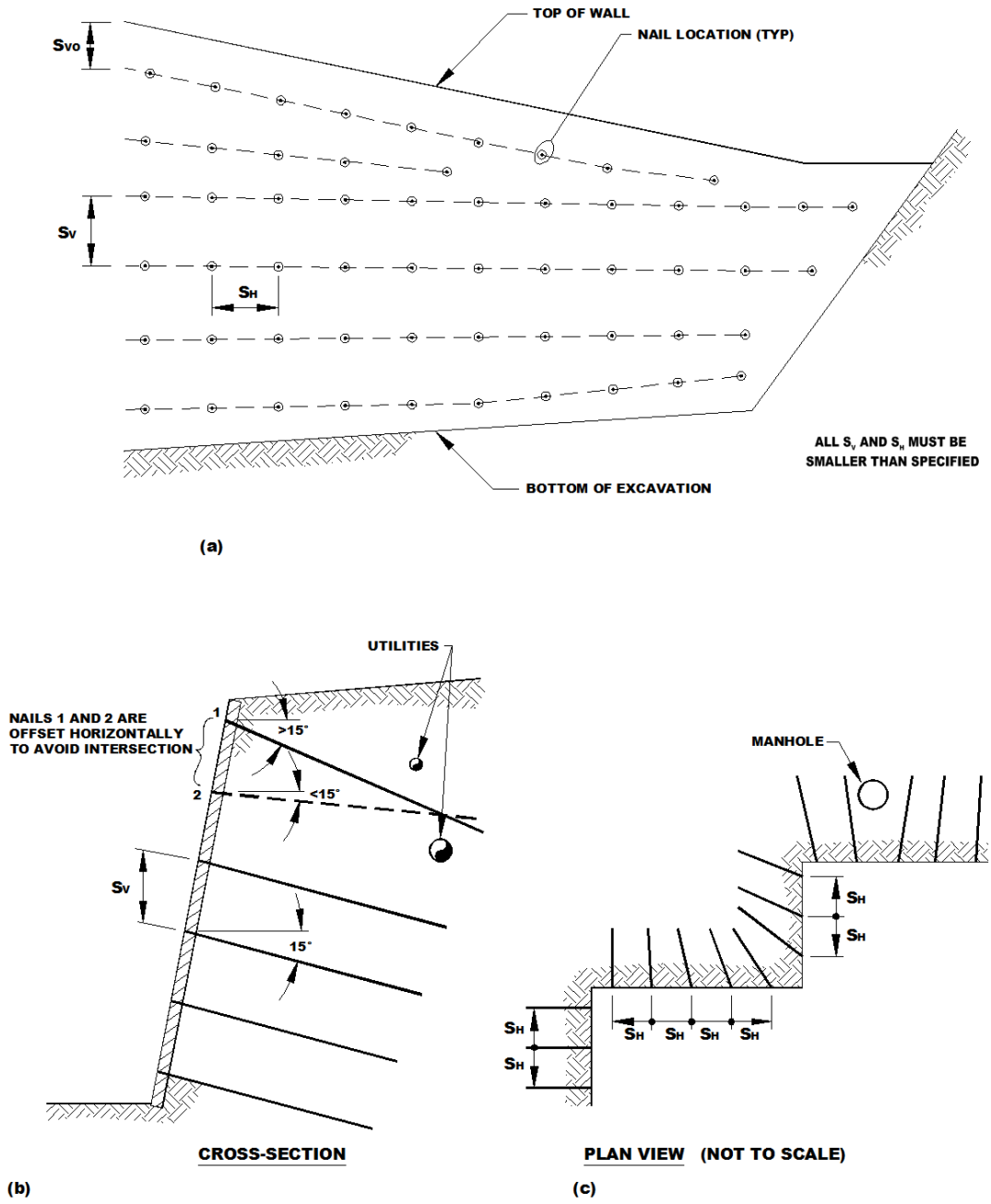


Figure 6.2: Illustration. Varying nail patterns: (a) example of nail arrangement for non-horizontal ground, (b) varying nail inclination and lengths around utilities, and (c) nail splaying at corners.

6.3.3d Soil Nail Length

To develop a preliminary cross section for analysis, the soil nail length can be estimated to be approximately 0.7H, where H is the wall height. Longer nails will be needed: (i) in weak

soils; (ii) where deep-seated critical slip surfaces exist; (iii) when large surcharge loads exist behind the wall; and/or (iv) when a rising backslope exists behind the wall.

Battered walls typically result in slightly shorter nails. A batter of 10 percent reduces the soil length requirements by approximately 10 to 15 percent when compared to walls with vertical faces, with all other conditions equal.

If it is determined with well-selected parameters and appropriate stability analysis that nail lengths would have to be greater than about 1.2H, this result may suggest that ground conditions are not favorable for soil nail walls. On the other end of the range, nail lengths less than 0.5H will not likely satisfy sliding stability requirements. Nail lengths less than approximately 0.6H are rare.

6.3.3e Distribution of Soil Nail Lengths in Elevation

All nails in a wall can have uniform length for simplicity. This pattern is common in many walls. However, design engineers must take into account other considerations related to a uniform versus a non-uniform soil nail length pattern.

Section 5.6.2 "Internal Stability" presented a discussion about how limit-equilibrium-based soil nail wall design programs tend to be unable to reproduce actual distributions of soil nail forces. In modeling soil nails of uniform length, these programs tend to predict the largest forces occurring in the lower sections of the wall; however, the largest soil nail forces tend to actually occur in the upper two thirds of the wall (as long as the upper nails are as long as or longer than the lower nails). In addition, the load distribution indicated in these programs tends to be inefficient across all nail rows. While a uniform soil nail pattern is not inherently unsafe, it is not optimal for reducing wall deformation.

To offset this drawback, the design engineer may need to consider shorter nails in the lower half of the wall, with nails increasingly shorter toward the bottom. To achieve the same factor of safety as for uniform-length nails, if the lower rows are shortened, the upper rows must be longer. Therefore, smaller overall values of T_{max} will be obtained, and a more realistic and better load distribution among nails will be obtained. In addition, longer nails in the upper half will result in smaller wall movements, other conditions being equal.

The following guidelines can be followed when selecting nail lengths to be modeled in limit-equilibrium programs to achieve a more uniform load distribution among rows and reduce wall deformations (based on a procedure by Byrne et al. 1996).

- Upper half of the wall: Soil nails whose heads are in this zone should have a uniform length, L.

- Lower half of the wall: Soil nails whose heads are in this zone should be increasingly shorter toward the bottom. The lengths of these nails must be determined by linear interpolation from value L at the wall mid height, to R × L at the base of the wall. R is < 1.0 and is selected depending on subsurface and geometric conditions and other factors, as indicated below.

    - For very dense, coarse-grained granular soils: $0.15 \leq R \leq 0.30$
    - For silty sand, sand, to gravelly sand: $0.25 \leq R \leq 0.40$
    - For fined-grained soils: $0.30 \leq R \leq 0.45$

    R has been estimated for the following conditions: safety factor for pullout $FS_{PO} = 2.0$, drill hole diameter (D_{DH}) between 4 to 8 in., horizontal and vertical nail spacing (S_H and S_V) between 4 and 6 ft, and typical ranges of bond strengths (q_U) for the soil types listed above.

    In addition, the following ranges of soil properties were considered to be consistent with the listed soil types: soil unit weight of retained soils (γ_s) between 110 and 130 pcf, and ratio of maximum soil nail length to wall height (L/H) between 0.75 and 1.0. In general, larger values of D_{DH} and q_U, in conjunction with lower values of S_H, S_V, and γ_s, would produce lower values of R.

Using a similar approach to that described above, SNAP-2 can perform a correction of the nails lengths if they were initially selected with the same length. This correction must be selected by the user because it is not a default setting. In other programs, the nail length can be easily modified. Regardless of the selected soil length pattern, stability analyses must be performed to verify that trial lengths and distribution of nails are adequate.

Note, however, that other soil nail length patterns can be selected to obtain lower estimates of the maximum soil nail load and to promote smaller wall deflections. For example, soil nail lengths can be selected to be longest in the upper third of the wall height, with an intermediate length in the central third, and shortest in the lower third.

The presence of utilities or underground structures behind a soil nail wall may also require altering the lengths of some rows or individual nails. To achieve a target factor of safety, some nails may need to be longer, to compensate for other nails that need to be shorter, to avoid obstructions.

6.3.4 Step 4c Select Soil Nail Pattern on Wall Face

Soil nails are installed on the excavation face in "square" or, more commonly, "staggered" (also referred to as triangular or offset) patterns (Figure 6.1). The pattern of nails on the excavation face can become irregular at locations with space restrictions.

In the square pattern, nails are vertically aligned in rows. This pattern allows the easy construction of vertical joints in shotcrete and an easier installation of precast concrete panels (if used). Drain strips are equidistant from nails in this pattern. A staggered pattern results in more uniform earth-pressure distributions, better soil arching effects, and provides a slightly larger resistance compared to those from a square pattern.

6.3.5 Step 4d Evaluate Soil Nail Horizontal Splaying

Nails may need to be splayed on plan view to: (i) avoid manholes and other obstructions, (ii) avoid external corners due to interference with adjacent nails (Figure 6.2c); or (iii) to possibly improve stability at internal corners. The engineer must consider nail splaying before using a design computer program because these programs do not account for the splay angle.

6.3.6 Step 4e Detail Corrosion Protection

The designer must select the corrosion protection technique or techniques that meet the level of corrosion protection established during the Initial Design Considerations phase. This selection involves specifying a material or process that is suitable for the nail type and installation procedures. Guidelines for selection of corrosion protection materials are provided in Chapters 7 and 10.

6.3.7 Step 4f Select Soil Nail Type and Material Properties

The engineer must select a grade of steel for the soil nail bar and other metallic parts. Information on steel grades and sizes is presented in Chapter 3 and Appendix A.

In traditional Design/Bid/Build contracts, the engineer may estimate a practical minimum drill hole diameter to provide the bond resistance required for stability. However, the drill hole diameter is ultimately selected by the Contractor to obtain the specified, nominal pullout resistance, and to possibly allow cleaning the drill hole, or accommodating a tremie pipe, tendon couplers, and centralizers.

6.4 Step 5 Select Resistance Factors

The engineer should select resistance factors for LRFD verification and in accordance with the formulations presented in Chapter 5. Resistance factors are summarized in Table 6.3.

As discussed in Section 5.3.4 "Calibration of Resistance Factors," the selected resistance factors are consistent with the minimum required frequency of verification and proof testing, as discussed in detail in Chapter 9.

Table 6.3: Resistance Factors for Soil Nail Wall Design

Limit State	Condition	Symbol	Resistance Factor
Overall Stability	Static	ϕ_{OS}	0.65
Overall Stability	Seismic loading	ϕ_{OS}	0.90
Basal Heave	Permanent wall, short-term conditions	ϕ_{BH}	0.50
Basal Heave	Permanent wall, long-term conditions	ϕ_{BH}	0.40
Pullout	Static	ϕ_{PO}	0.65
Pullout	Seismic loading	ϕ_{PO}	0.65
Lateral Sliding	Static	ϕ_{LS}	0.90
Lateral Sliding	Seismic loading	ϕ_{LS}	1.00
Tensile Resistance	Static – Grades 60/75 (ASTM A615)	ϕ_{T}	0.75
Tensile Resistance	Static – Grades 95/150 (ASTM A722)	ϕ_{T}	0.65
Tensile Resistance	Seismic loading – Grades 60/75 (ASTM A615)	ϕ_{T}	0.75
Tensile Resistance	Seismic loading – Grades 95/150 (ASTM A722)	ϕ_{T}	0.65
Facing Flexural Resistance	Static	ϕ_{FF}	0.90
Facing Flexural Resistance	Seismic loading	ϕ_{FF}	0.90
Punching Shear at Facing	Static	ϕ_{FP}	0.90
Punching Shear at Facing	Seismic loading	ϕ_{FP}	0.90
Headed Stud in Tension	Static – A307 Steel Bolt	ϕ_{FH}	0.70
Headed Stud in Tension	Static – A325 Steel Bolt	ϕ_{FH}	0.80

Limit State	Condition	Symbol	Resistance Factor
Headed Stud in Tension	Seismic loading – A307 Steel Bolt	ϕ_{FH}	0.65
Headed Stud in Tension	Seismic loading – A325 Steel Bolt	ϕ_{FH}	0.75

6.5 Step 6 Overall Stability

6.5.1 Step 6a Evaluate Internal Stability

Internal stability is typically analyzed using two-dimensional, limit-equilibrium, ASD-based slope stability procedures, with readily available computer programs. A well-established computer program for designing soil nail walls should be preferred so that it can model multiple soil strata, groundwater, external loads, seismic loads, and also various geometric features.

The ASD-based slope stability program will calculate the safety factors for overall stability (FS_{OS}) and the maximum loads of all nails. Of these loads, which are limited in the program by tensile, pullout or facing resistances, the maximum among all nails (T_{max}) will be selected for LRFD verifications, which are discussed in subsequent sections. If uniform soil length patterns are selected, larger values of T_{max} will be obtained when compared to soil nail loads for nails that are increasingly shorter toward the base of the wall (for the same FS_{OS} and other conditions being the same).

The engineer must enter all applicable parameters into the computer program, including bond strengths for each representative layer (developed in Step 3e), wall geometry, nail layout, groundwater location, and external loads (Steps 4a – 4d). The soil parameters must be characterized according to Chapter 4, including soil unit weight, effective cohesion, effective angle of friction, and bond strengths. Preferably mean values should be input.

Select an appropriate, minimum safety factor, FS_{OS}, for internal stability as follows (Table 5.1):

- For static conditions of permanent structures that the Owner considers to be "critical," minimum $FS_{OS} = 1.5$.
- For static conditions of permanent structures that the Owner considers to be non-critical, minimum $FS_{OS} = 1.35$. This value may also apply to a critical structure and when the design uncertainty is considered to be limited (e.g., availability of abundant geotechnical information and successful local experience on soil nailing).

- For static conditions and temporary excavation (e.g., excavation lifts are unsupported for up to 2 days before nails are installed) for permanent structures that the Owner considers to be critical, minimum $FS_{OS} = 1.30$.
- For static conditions and temporary excavation for permanent structures that the Owner considers to be non-critical, minimum $FS_{OS} = 120$.
- For seismic conditions, when permanent seismic deformations are not taken into account, minimum $FS_{OS} = 1.1$.
- For seismic conditions, when the horizontal seismic coefficient is based on estimated, allowable seismic deformations, minimum $FS_{OS} = 1.0$.

Some ASD computer programs require input of values of factors of safety, or alternatively reduction factors for pullout, tension, and facing flexure, as shown in Step 5. Recommended minimum factors of safety for these limit states were also presented in Table 5.1.

The user has to first estimate (outside of the program) the ultimate tensile and facing punching resistance, per Table 6.3 or via an iterative process (see Step 7); and then enter them into the program. The user should select mean values of the strengths (or nominal resistances) for consistency with LRFD practice. Programs such as SNAP-2 calculate the strengths of the facing based on the user's input of facing design parameters (thickness, strengths, etc.) and factors of safety specific to these failure modes, also entered by the user. The user must also enter a trial design (soil nail lengths, spacing, size, facing details, etc., per Step 4) into the computer program.

The following iterative procedure is used in ASD stability analyses:

- Enter trial design and calculate the factor of safety.
- Compare it to the selected minimum factor of safety, FS_{OS}.
- Modify nail lengths (globally or in selected rows) until the calculated factor of safety is equal to or higher than the minimum FS_{OS}.
- Inspect the calculated loads of all nails, and record the maximum value of each nail for later use. Select T_{max} as the largest of all these nail loads.
- If the program indicates that nail length should be much longer than the range of nail lengths listed in Step 4e, consider adjusting the nail resisting force by increasing the nail hole diameter (more cost-effective), decreasing the nail spacing (less cost-effective), or by combining these effects.

Soil nail design programs are generally sufficient for producing complete ASD-based designs for typical and routine structures. However, the range of potential slip surfaces investigated may vary. Potential slip surfaces should be investigated: (i) above and below a wall; (ii) completely behind and beneath the soil nails; and (iii) at each stage of construction. Additionally, for complex wall geometries (multi-tiered walls, sloping terrain above or at the

toe of the wall), general slope-stability programs may need to be used to thoroughly investigate instability concerns.

Note that general slope-stability programs may require different types of input. For example, some programs require that the nail pullout resistance be entered as an allowable value (e.g., one of the options in SNAIL or SNAILZ). Other programs allow the user to enter either bond stresses or resistance per unit length. In SNAP-2, the ultimate (i.e., nominal) value of the bond strength is input.

Soil nail design programs usually indicate which structural limit state is controlling the analysis: pullout, tensile, or facing mechanisms. With this feature, the engineer may iterate an analysis more efficiently by modifying the features associated with the controlling limit state: length of nail or drill hole diameter for pullout, nail bar size for tensile resistance, or facing thickness and/or facing reinforcement for facing resistance mechanisms. If the failure mode is pullout, some programs, such as SNAP-2, indicate in which soil layer pullout is occurring.

When soil nail walls are evaluated for seismic loading, the horizontal seismic coefficient, k_h, needs to be input in design programs. Guidance for selecting the horizontal seismic coefficient is presented in Section 6.8.

6.5.2 Step 6b Evaluate Global Stability

The global stability of soil nail walls must be performed using two-dimensional, limit-equilibrium analyses in a conventional ASD framework. In these analyses, the slip surfaces must be considered to extend behind the soil nails, hence disregarding any contribution to stability from the nails. The Owner is responsible for providing global stability in most projects before the project construction is put out for bid. Depending on the type of contract, the Contractor may be responsible after bid for verifying internal stability of the soil nail wall. Because the length of soil nails is only assumed until a design is complete (by the Owner in Design/Bid/Build procurement, or by the Contractor in a Design/Build contract), global stability analyses may need to be repeated considering the final design nail lengths.

General slope stability computer programs are more commonly used to evaluate global stability than dedicated soil nail design software, as discussed in Section 5.6.3 “Global Stability.” Soil nail design software may provide reasonable results for simple wall geometries and regular stratigraphies; however, these results are generally not comprehensive because slip surfaces do not extend under the toe of a wall, or, if they do, the passive resistance below the wall is generally not modeled correctly. While slip surfaces in SNAP-2 can extend under the toe of the wall, the program does not provide the user full control to select trial slip surfaces.

Both internal and external stability must be evaluated in complex geometries, such as that of a stepped wall shown in Figure 6.3. Internal stability analysis must include slip surfaces intersecting some or all of the soil nails of each wall and the retained soil. These slip surfaces are labeled "compound" and are shown with dashed lines in Figure 6.3. All soil nail design software can handle short, compound slip surfaces behind individual stepped walls. However, these programs, with the exception of SNAP-2, cannot model compound slip surfaces extending below the toe of individual walls. On the other hand, none of the soil nail design programs can properly model the long slip surface shown as a filled line in Figure 6.3.

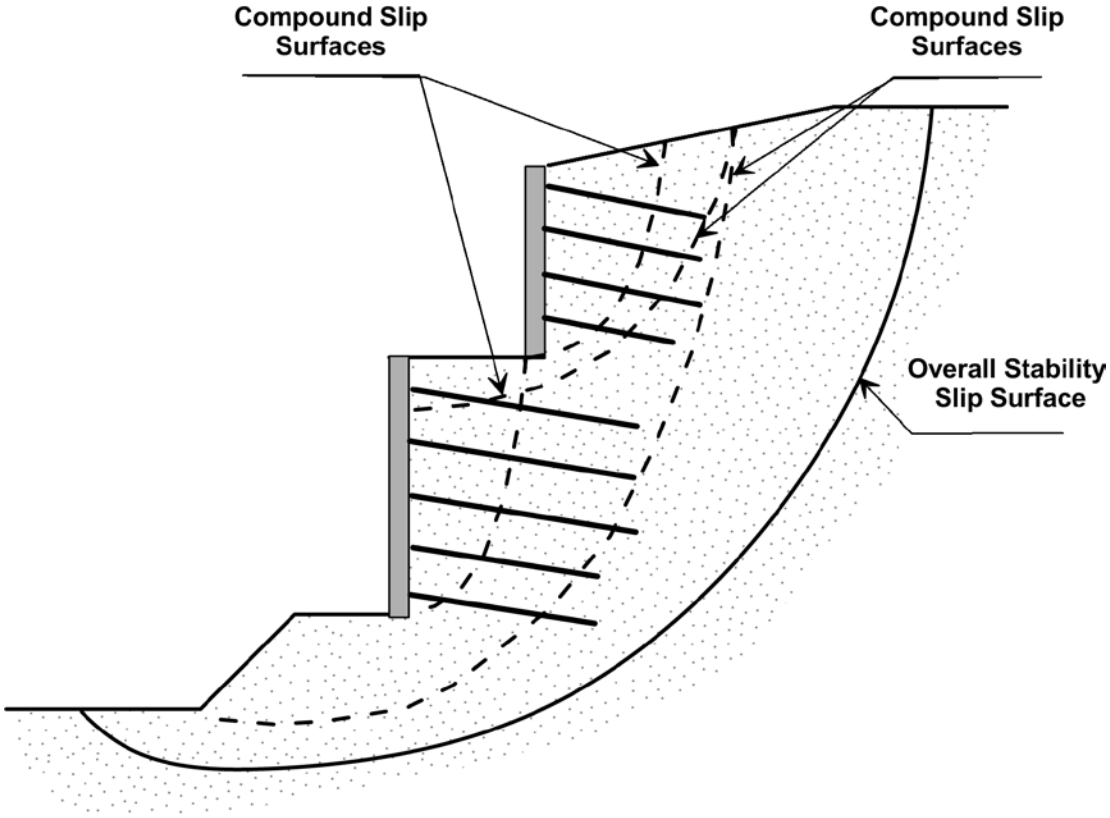


Figure 6.3: Illustration. Overall and compound slip surfaces in complex soil nail walls. Modified after AASHTO (2014).

6.5.3 Step 6c Evaluate Basal Heave (if applicable)

If soft fine-grained soils exist at the base of a soil nail wall, the potential for heave at the base of the excavation must be evaluated. If computations indicate a high potential for basal heave, an alternative support-of-excavation system may need to be recommended.

Preliminary evaluation of basal heave should be performed during the Initial Design Considerations phase. If appropriate, a detailed evaluation should be performed for the final wall layout. In some cases, general slope-stability programs can be used to evaluate stability

for both drained and undrained conditions of the fine-grained soils. The slip surfaces should be selected to extend under the wall and to daylight at the base of the excavation. For soft fine-grained soils, undrained conditions for short-term conditions are more critical than drained resistance conditions.

However, the analysis of basal heave may require special considerations with slope stability software to force the slip surfaces along vertical planes. The simplified method described in Section 5.6.6 "Basal Heave" can be used to evaluate basal heave.

6.5.4 Step 6d Evaluate Sliding Stability (if applicable)

The potential for sliding along the base of the nail-reinforced block must be evaluated if a horizontal (or sub-horizontal with unfavorable orientation) layer of weak soil is present immediately under the zone of nail-reinforced soil, and this layer leads to instability after the wall excavation is initiated.

General, modern ASD-based slope-stability programs can be used to evaluate sliding stability, as they can model planar slip surfaces, which would pass along the layer of weak soil. On the other hand, most soil-nail design computer programs cannot appropriately model horizontal slip surfaces. If ASD-based slope stability programs are used for these evaluations, LRFD verifications of sliding stability should be conducted considering the resistance factors for overall stability of Table 5.4 and load factors equal to 1.0.

Alternatively, if the simplified procedure of Section 5.7.7 "Lateral Sliding Stability" is used for these evaluations, LRFD verifications of sliding stability should include the resistance factors for sliding of Table 5.7 and the EV and ES load factors from Table 5.3.

6.6 Step 7 Verify Geotechnical and Structural Resistances

6.6.1 Overview

At this stage, the designer first must obtain from the stability analysis the load at each nail level and select T_{max} as the largest value in all nails. This will represent the load in the LRFD verification of strength limit states. The load will be multiplied by a load factor that is selected for LRFD verification in this manual. The selected load factor is $\gamma = \gamma_{EV} = 1.35$, where γ_{EV} is the load factor for vertical earth loads per AASHTO (2012). The selected load factor will be appropriate in most cases. While vertical and horizontal earth loads are the predominant loads in soil nail walls, analyses of idealized soil nail walls suggest that the effect of the vertical earth loads is more significant than that of horizontal earth loads. Therefore, the load factor selected is associated with vertical earth loads. However, the load factor selected for verification may need to be adjusted in cases where surcharge loads are

much larger than earth loads. Loads other than soil self-weight (for example, surcharge or traffic loads) may be considered using an ASD-based stability program by entering these loads as adjusted values, where an unfactored, original load is modified by a factor γ/γ_{EV}, where γ is the load factor associated with that load component.

6.6.2 Step 7a Verify Pullout Resistance

Pullout resistance is mobilized behind the slip surface, along the length, L_p, and contributes to overall stability. The length, L_P, can be estimated from the graphical output of soil nail design programs, where critical slip surfaces and soil nails are shown to an appropriate scale. The nominal (i.e., ultimate) pullout resistance per unit length, r_{PO}, is expressed as:

$$r_{PO} = \pi \, q_u \, D_{DH}$$

Equation 6.1: Nominal unit pullout resistance.

Where:

q_u = bond strength of the nail-grout-soil interface (force/unit area)

D_{DH} = diameter of the drill hole

Distributions of bond stresses along the grout-soil interface can be complex and exhibit variations along L_p (Figure 6.4).

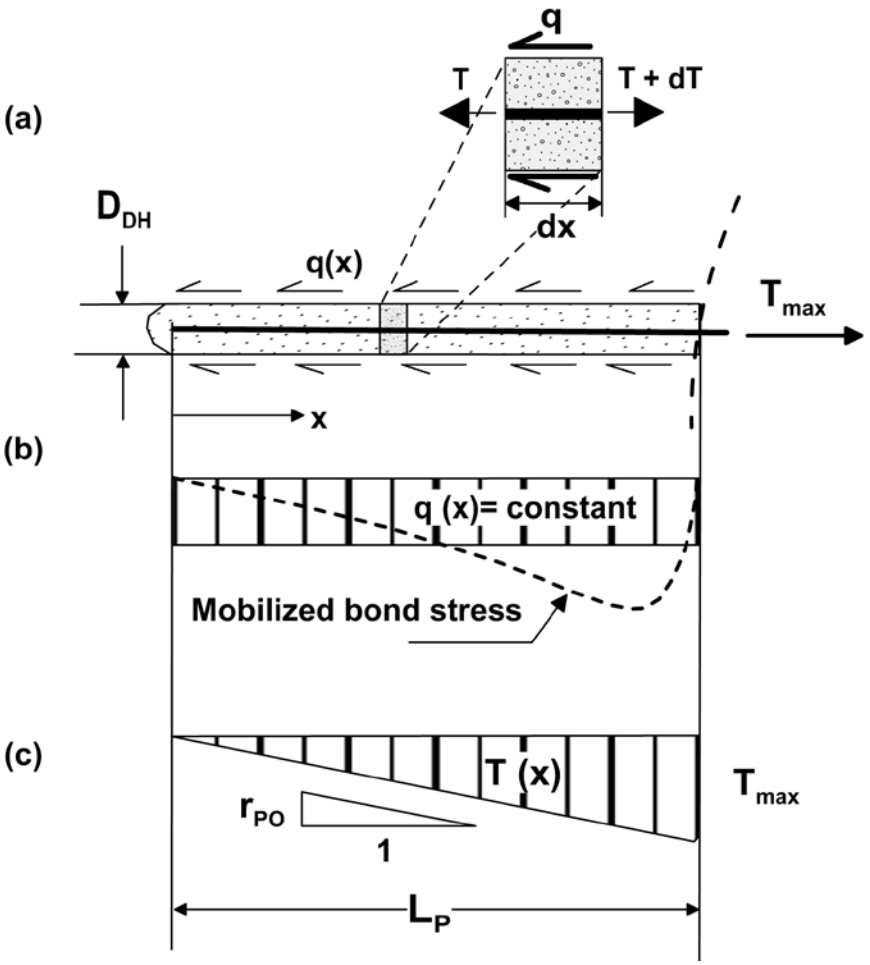


Figure 6.4: Illustration. Single nail stress-transfer mode: (a) soil nail layout, (b) distribution of mobilized bond stresses, and (c) hypothetical distribution of loads along the nail.

The distribution is assumed to be constant along the nail pullout length for simplicity, and the bond stress is considered to have an apparent, average value. When the bond stress increases to its maximum value, the bond strength, q_u, is mobilized.

The nominal pullout resistance, R_{PO}, is calculated as follows:

$$R_{PO} = r_{PO}\, L_P$$

Equation 6.2: Nominal pullout resistance.

Pullout resistance is evaluated as follows:

$$CDR = \frac{\phi_{PO}\ R_{PO}}{\gamma\ T_{max}} \geq 1.0$$

Equation 6.3: Capacity-to-demand ratio (CDR).

Where:

γ = $\gamma_{EV} = 1.35$

T_{max} = maximum nominal tensile force

6.6.3 Step 7b Evaluate Sliding Stability (if applicable)

The sliding stability should be evaluated if a layer of weak soil underlies the block of reinforced soil. A simplified hand-calculation, as presented in Section 5.7.3, can be used when the sliding plane is horizontal. For more complex subsurface conditions, the sliding evaluation can be performed using a general slope-stability program. Sliding stability should be checked with service limit state load factors. Minimum factors of safety for the ASD-based slope stability analysis are presented in Table 5.1 for static and seismic design cases. In general, soil nail computer programs do not have the capability of modeling these types of slip surfaces.

6.6.4 Step 7c Verify Nail Tensile Resistance

The capacity-to-demand ratio will be:

$$CDR = \frac{\phi_T \ R_T}{\gamma \ T_{max}} \geq 1.0$$

Equation 6.4: CDR definition.

Where:

ϕ_T = resistance factor for tensile resistance of the tendon

R_T = nominal tensile resistance of the tendon

γ = load factor selected for verification

T_{max} = as defined before

The nominal tensile resistance of the tendon is:

$$R_T = A_t f_y$$

Equation 6.5: Nominal tensile resistance of tendon.

Where:

A_t = cross-sectional area of tendon

f_y = nominal yield resistance of tendon (force per area)

In selecting the nominal yield resistance of the tendon, consider that Grade 150 bars achieve an apparent yield at 120 ksi and a tensile rupture at 150 ksi. Grade 75 bars yield at 75 ksi and rupture at 100 ksi. The grout surrounding the bar does not contribute to the tensile resistance of the soil nail.

Select the smallest bar size that meets the required cross-sectional area (see Appendix A). Although the ultimate selection of the drill hole diameter will be the Contractor's responsibility, the designer should verify during design if the tendon diameter is compatible with: (i) the estimated drill hole diameter; (ii) the requirements for corrosion protection plus grout cover, and (iii) couplers.

6.6.5 Step 7d Verify Facing Bending/Flexural Resistance

6.6.5a Initial Calculations

Programs like SNAP-2 perform a rigorous calculation of the flexural resistance and compute the reinforcement ratio checks. For programs that do not have this capability, the designer must verify the flexural resistance at the facing using the following steps:

(1) Select the thickness of the initial facing (h_i) and the final facing (h_f):

h_i (in in.): 4 (typical) or 6
h_f (in in.): 6, 8 (typical, for final shotcrete), 10 and thicker (CIP facing)

(2) Select type and grades of facing reinforcement:

Grade 60 rebar is typically used. Grade 60 WWM is typical, but alternative grades such as 70, 75 and 80 are also available. Select yield strengths f_{y_R} and f_{y_W} for rebar and WWM, respectively.

Refer to Appendix A for WWM and rebar sizes.

(3) Select shotcrete strength (f'_c): (typically 3000 to 4000 psi, with the latter value being more commonly used).

(4) Select headed stud characteristics (Appendix A).

Select bearing plate geometry:
Recommended minimum dimensions: 8 in. × 8 in.
Recommended minimum thickness: 0.75 in.

(5) Calculate the reinforcement ratio, ρ_{ij}, as:

$$\rho_{ij} = \frac{a_{ij}}{0.5\,h}\ 100$$

Equation 6.6: Reinforcement ratio.

Where, a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) and h = thickness of the facing being designed, whether initial or final. The direction "i" can be "v" (for vertical) or "h" (for horizontal); the location "j" can be "n" (nail head) or "m" (mid-span between nails).

The cross-sectional areas of reinforcement per unit width in the vertical or horizontal direction and around and between nails are shown schematically in Figure 6.5. The nomenclature for the reinforcement areas per unit width is presented below and in Table 6.4:

$$a_{vn} = a_{vm} + \frac{A'_{VN}}{S_H}$$

Equation 6.7: Definition of reinforcement cross-sectional area per unit width in the vertical direction at the nail head.

$$a_{hn} = a_{hm} + \frac{A'_{HN}}{S_V}$$

Equation 6.8. Definition of reinforcement cross-sectional area per unit width in the horizontal direction at the nail head.

Table 6.4: Nomenclature for Cross-Sectional Area per Unit Width of Facing Reinforcement

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail Head (1)	a_{vn}
Vertical	Mid-Span (2)	a_{vm}
Horizontal	Nail Head (1)	a_{hn}
Horizontal	Mid-Span (2)	a_{hm}

Notes: (1) Both WWM and the rebar contribute.
(2) Only WWM contributes.

Where:

a_{vn} = reinforcement cross-sectional area per unit width in the vertical direction at the nail head. If the yield strengths of the WWM and the rebar are different in the initial facing, this is an equivalent value.

a_{vm} = reinforcement cross-sectional area per unit width in the vertical direction at midspan

a_{hn} = equivalent 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

A'_{VN} = equivalent cross-sectional area at the head in the vertical direction to consider different yield strengths for the WWM and rebar. $A'_{VN} = (f_{y_R}/f_{y_W}) \times A_{VN}$, where f_{y_R} = rebar yield strengths and f_{y_W} = WWM yield strength

A'_{HN} = equivalent cross-sectional area at the head in the horizontal direction. $A'_{HN} = (f_{y_R}/f_{y_W}) \times A_{HN}$

S_H = nail horizontal spacing

S_V = nail vertical spacing

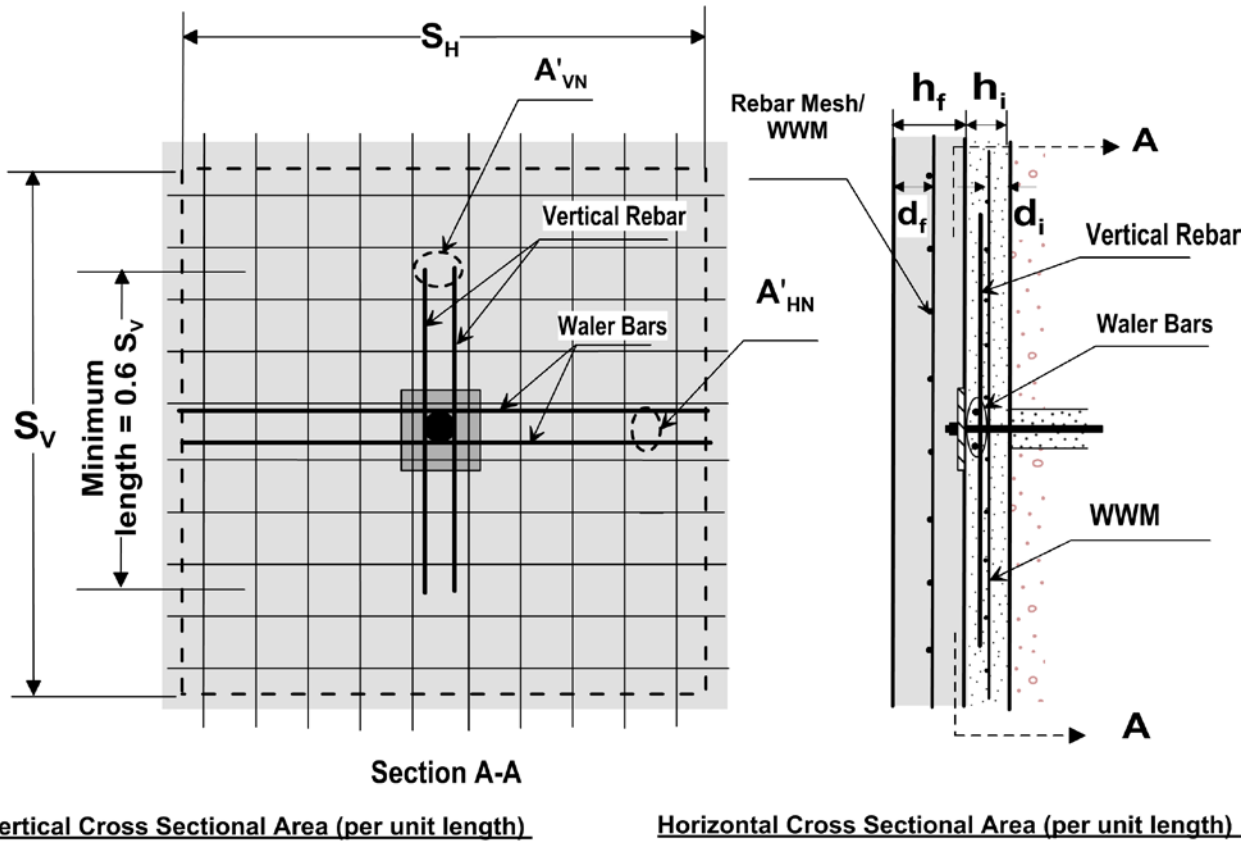


Figure 6.5: Illustration. Reinforcement nomenclature for facing in bending.

The additional horizontal and vertical rebar in the initial facing must be placed near the nail heads, usually within a distance h_i from the walls of the drill hole. Horizontal bars should have a length of at least $0.60S_H$; however, it is not uncommon to place waler bars with a length equal to S_H (Figure 6.5). Vertical rebar should be at least $0.60S_V$ long. Laps between adjacent bars must be in the mid-span between nails. The wire spacing in WWM typically used in soil nail walls must meet the requirements for maximum spacing of reinforcement per Section 10.6 "Distribution of Flexural Reinforcement" of ACI (2011).

6.6.5b Estimation of Nominal Flexural Resistance

The nominal flexural resistance, R_{FF}, is the force at the nail head that arises when the facing reaches its maximum bending resistance. This force is calculated based on the contribution of bending capacities around the nails and in the mid-span between nails of equivalent two-way slabs. If the vertical and horizontal spacings are different or if different size reinforcements are used in the two directions (a relatively uncommon situation), the maximum force must be calculated separately for the vertical and horizontal directions, and the smaller of these forces must be adopted as R_{FF}. This evaluation must be carried out for both the initial and final facings, as shown below.

Initial Facing

To estimate R_{FF} for the initial facing, the bending resistance (m_v) of the initial facing must be first evaluated at the nail head and then in the mid-span, for both vertical and horizontal directions, and considering equivalent cross-sectional areas in each case. For each of these four scenarios, the bending resistances are:

$$m_{vm} = a_{vm} f_{y_W} \left(\frac{h_i}{2} - \frac{a_{vm} f_{y_W}}{1.7 f'_c} \right)$$

Equation 6.9: Vertical bending resistance of initial facing at midspan.

$$m_{vn} = a_{vn} f_{y_W} \left(\frac{h_i}{2} - \frac{a_{vn} f_{y_W}}{1.7 f'_c} \right)$$

Equation 6.10: Vertical bending resistance of initial facing at nail head.

$$m_{hm} = a_{hm} f_{y_W} \left(\frac{h_i}{2} - \frac{a_{hm} f_{y_W}}{1.7 f'_c} \right)$$

Equation 6.11: Horizontal bending resistance of initial facing at midspan.

$$m_{hn} = a_{hn} f_{y_W} \left(\frac{h_i}{2} - \frac{a_{hn} f_{y_W}}{1.7 f'_c} \right)$$

Equation 6.12: Horizontal bending moment of initial facing at nail head.

R_{FF} is the smaller of the following two quantities:

$$R_{FF} = C_F \times \left(m_{vm} + m_{vn} \right) \times \left(\frac{8 S_H}{S_V} \right)$$

Equation 6.13: Nominal bending resistance for bending in the vertical direction.

$$R_{FF} = C_F \times \left(m_{hm} + m_{hn} \right) \times \left(\frac{8 S_V}{S_H} \right)$$

Equation 6.14: Nominal resistance for bending in the horizontal direction.

C_F is a factor that considers the effect of non-uniform soil pressures acting behind the facing. The distribution of soil pressure behind the wall is affected by the magnitude of the wall displacement, soil conditions, and the facing stiffness and thickness. As depicted in Figure

6.6, the soil pressure is smaller between nails, where the displacements are larger and tend to produce a stress relief. The soil pressure immediately behind the nails is larger than that in the mid-span because the soil confinement is greater.

Eqs. 6.9 through 6.14 are also valid for the final facing provided the parameters of the equations (thickness and material strengths) are selected appropriately for the final facing.

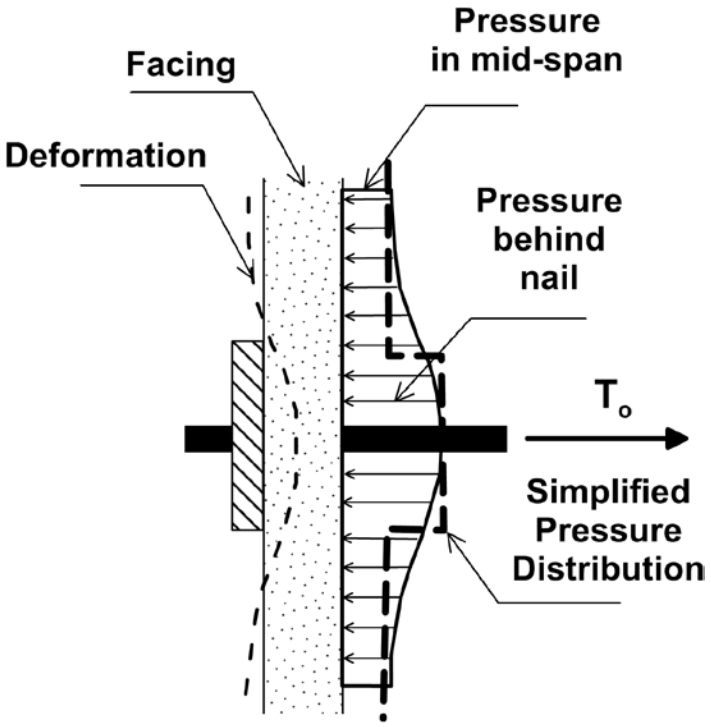


Figure 6.6: Illustration. Soil pressure distribution behind facing. Modified after Byrne et al. (1998).

If Grade 60 steel is used for WWM and rebar, and f'_c = 4,000 psi for shotcrete, the following simplified equations can be used:

$$R_{FF}[\text{kip}] = 3.8 \times C_F \times f_y[\text{ksi}] \times F = 228 \times C_F \times F$$

Equation 6.15: Nominal bending resistance for Grade 60 steel WWM/rebar and 4,000 psi shotcrete.

Where:

$$F = \text{smaller of} \begin{cases} \left(a_{vn} + a_{vm}\right)\left[\text{in}^2/\text{ft}\right] \times \left(\dfrac{S_H \; h\,[\text{ft}]}{S_V}\right) \\ \left(a_{hn} + a_{hm}\right)\left[\text{in}^2/\text{ft}\right] \times \left(\dfrac{S_V \; h\,[\text{ft}]}{S_H}\right) \end{cases}$$

Equation 6.16: Definition of F.

Assuming that $a_{vn} = a_{vm} =$ and $a_{hn} = a_{hm} = a_s$, and $S_H = S_V$, the following equation can be used for the final facing, where $C_F = 1.0$:

$$R_{FF}\left[\text{kip}\right] = 456 \times a_s \left[\text{in}^2/\text{ft}\right] \times h_f\,[\text{ft}]$$

Equation 6.17: Nominal bending resistance.

C_F is selected from Table 6.5.

Table 6.5: Factor C_F

Facing	Facing Thickness, h_i or h_f (in.)	C_F
Initial	4	2.0
Initial	6	1.5
Initial	8	1.0
Final	All	1.0

The facing flexural nominal resistance (R_{FF}) for the initial facing is calculated using Equations 6.15 or 6.17. Alternatively, R_{FF} can be estimated from Table 6.6 using results for initial calculations (Section 6.6.5a).

Table 6.6: Nominal Flexure Resistance, R_{FF} (Initial Facing)

Facing thickness, h_i	Nail Spacing Ratio[1]	ρ_{tot} = 0.5 % [2]	ρ_{tot} = 1.0 % [2]	ρ_{tot} = 2.0 % [2]
(in.)	(ft/ft)	R_{FF} (kip) [3] [5]	R_{FF} (kip) [4] [5]	R_{FF} (kip) [4] [5]
4	0.67	12	24	48
4	1	18	36	71
6	0.67	20	40	81
6	1	30	60	120
8	0.67	24	48	95
8	1	36	71	143

(1) Nail spacing ratio = smaller of S_V/S_H or S_H/S_V.

(2) $\rho_{tot} = \rho_n + \rho_m$, ρ_n and ρ_m are the nail head and mid-span reinforcement ratios, respectively, in either direction.

(3) The above values are valid for Grade 60 steel. Multiply the values above by 1.24 for Grade 75 steel.

(4) Divide R_{FF} by 2 for final facing thickness h_f = 4 in. Divide R_{FF} by 1.5. for h_f = 6 in. For h_f = 8 in., use same R_{FF}.

Final Facing

The same steps shown above for the initial facing must be completed for the final facing, considering the appropriate reinforcement, section thickness and material properties.

The facing resistance is verified in LRFD for each of the facings as follows:

$$CDR = \frac{\phi_{FF} \, R_{FF}}{\gamma \, T_o} \geq 1.0$$

Equation 6.18: Capacity-to-demand ratio for bending in facing.

Where:

ϕ_{FF} = resistance factor for bending/flexure in the facing

R_{FF} = nominal resistance for bending/flexure of facing

γ = load factor selected for verifications

T_o = maximum tensile force at soil nail head, as estimated with Eq. 5.1 (Section 5.2.1)

If the resistance is insufficient, increase the thickness of facing, amount of steel, and/or strength of steel and/or of concrete.

6.6.6 Step 7e Verify Facing Punching Shear Resistance

For programs that do not have the capability of verifying the punching shear resistance, the designer must verify the punching shear resistance at the facing using the procedure presented below.

Connectors installed at the nail head may be subjected to a punching shear limit state, which may occur if the nominal shear resistance of the reinforced shotcrete/concrete section around the nails is exceeded. The nominal punching shear resistance must be evaluated for both initial and final facings (Figure 6.7) for:

- Initial facing: bearing plate connection
- Final facing: headed stud connection

At the limit state, a conical shear surface forms in the facing around the nail head. The size of the conical shear surface is affected by the thickness of the facing, and the dimension of the bearing plate (initial facing) and headed studs (final facing).

The nominal facing punching shear resistance, R_{FP}, for either situation must meet the following condition:

$$CDR = \frac{\phi_{FP}\ R_{FP}}{\gamma\ T_o} \geq 1.0$$

Equation 6.19: Capacity-to-demand ratio for punching shear resistance.

Where:

ϕ_{FP} = resistance factor for punching shear in the facing

γ = load factor selected for verification

T_o = maximum tensile force at soil nail head, as defined previously

R_{FP} is estimated as:

$$R_{FP} = C_P\ V_F$$

Equation 6.20: Nominal punching shear resistance at facing.

Where:

C_P = dimensionless factor that accounts for the contribution of the soil support under the nail head to the shear resistance

V_F = concrete punching shear basic resistance acting through the facing section

C_P can be as high as 1.15 if the soil reaction is considered. The contribution from the soil support behind the wall is conservatively assumed to be negligible; therefore, $C_P = 1.0$. The punching shear resistance, V_F, can be calculated as:

$$V_F\,[\text{kip}] = 0.58\sqrt{f'_c\ [\text{psi}]}\ \pi\ D'_c[\text{ft}]h_c[\text{ft}]$$

Equation 6.21: Nominal punching shear resistance through facing.

Where:

f'_c = as defined before

D'_c = effective equivalent diameter of the conical slip surface

h_c = effective depth of the conical surface

D'_c and h_c must be selected separately for the initial and final facing, as follows:

Initial facing (Figure 6.7a)

$$D'_c = L_{BP} + h_i$$

Equation 6.22: Effective, equivalent diameter of conical slip surface through initial facing.

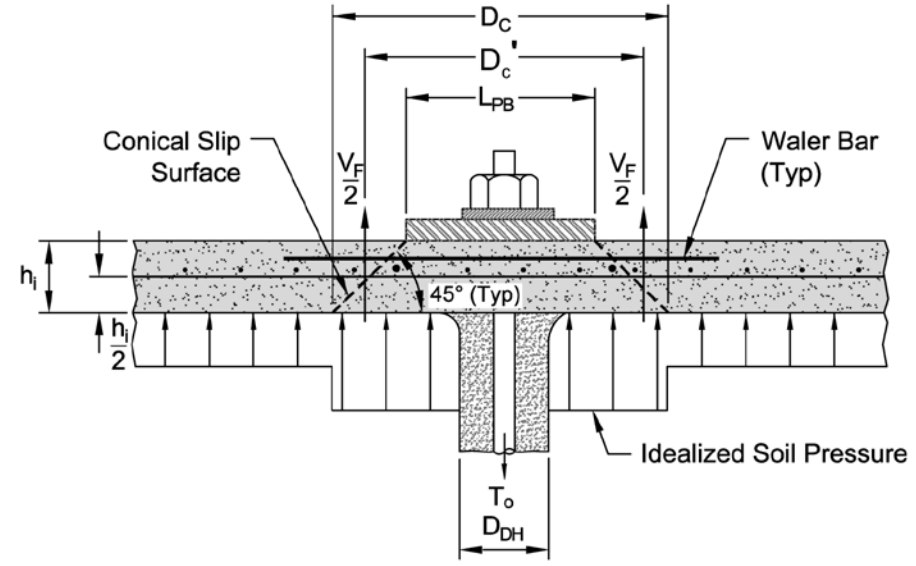
$$h_c = h_i$$

Equation 6.23: Effective depth of the conical slip surface, initial facing.

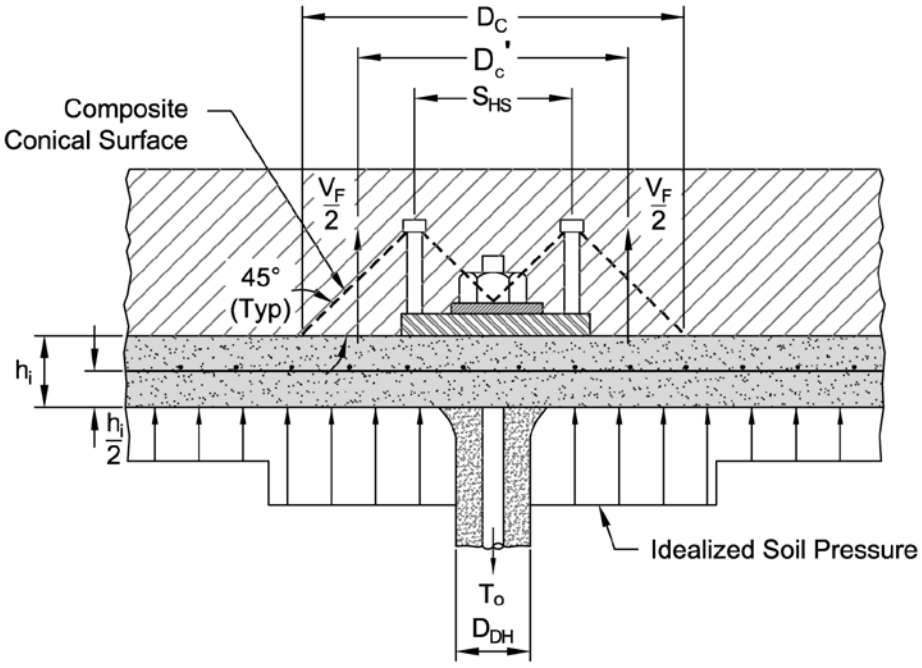
Where:

L_{BP} = bearing plate size

h_i = thickness of initial facing



(a) Bearing Plate Connection



(b) Headed Stud Connection

Figure 6.7: Illustration. Limit states for punching shear in facing – horizontal cross sections: (a) bearing plate connection, (b) headed-stud connection. Modified after Byrne et al. (1998).

Final facing (Figure 6.7b)

$$D'_c = \text{smaller value of} \begin{cases} S_{SH} + h_c \\ 2h_c \end{cases}$$

Equation 6.24: Effective, equivalent diameter of conical slip surface, final facing.

Where:

$$h_c = L_S + t_P - t_{SH}$$

Equation 6.25: Effective depth of conical slip surface, final facing.

And:

S_{SH} = headed stud spacing

L_S = headed stud length

t_{SH} = thickness of the stud head (Figure 6.8)

t_P = bearing plate thickness

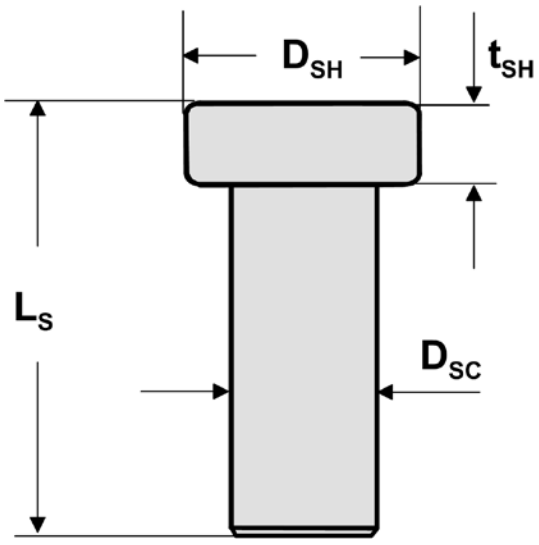


Figure 6.8: Illustration. Geometry of headed studs.

Note that L_S and h_f must be selected such that the cover of final shotcrete or concrete over the stud is at least 2 in. according to Section 7.6.7 "Nail-Head Corrosion Protection." Available sizes of headed studs can be found in Appendix A and in references provided by manufacturers.

A simplified method is presented below for estimating the punching shear resistance (R_{FP}).

(1) Adopt values for concrete strength (f'_c), facing thickness (h), and bearing plate length (L_{BP}).

(2) Calculate the nominal facing punching shear resistance (R_{FP}) for the initial and final facings using Tables 6.7a and b.

Table 6.7a: Facing Resistance – Shear Punching, R_{FP} (Initial Facing)

Facing thickness, h_i	f'_c	L_{BP} = 8 in.	L_{BP} = 9 in.	L_{BP} = 10 in.
(in.)	(psi)	R_{FP} (kip)	R_{FP} (kip)	R_{FP} (kip)
4	3,000	32	35	37
4	4,000	37	40	43
6	3,000	56	60	64
6	4,000	65	69	74
8	3,000	85	91	96
8	4,000	99	105	111

Notes:
(1) f'_c = concrete compressive strength.
(2) L_{BP} = bearing plate length.

Table 6.7b: Facing Resistance – Shear Punching, R_{FP} (Final Facing)

h_c (1)	f'_c	S_{SH} = 4 in.	S_{SH} = 5 in.	S_{SH} = 6 in.
(in.)	(psi)	R_{FP} (kip)	R_{FP} (kip)	R_{FP} (kip)
4	3,000	21	21	21
4	4,000	25	25	25
6	3,000	30	33	33
6	4,000	35	39	39
8	3,000	40	44	48
8	4,000	46	51	55

Note: (1) $h_c = L_s - t_H + t_P$, where:
L_s = effective length of headed stud (Table A.7).
t_P = thickness of bearing plate (typically 0.75 in.).
t_H = head thickness of headed stud (Table A.7).

6.6.7 Step 7f Verify Facing Headed Stud Resistance

For programs that cannot verify the headed stud resistance, the designer can verify this resistance at the facing using the following steps:

(1) Compute R_{FH} as follows:

$$R_{FH} = N_H \; A_S \; f_{y\text{-}hs}$$

Equation 6.26: Headed stud resistance at facing.

Where:

N_H = number of headed studs in the connection (usually 4)

A_S = cross-sectional area of the headed stud shaft

$f_{y\text{-}hs}$ = tensile yield strength of headed stud

(2) Verify that:

$$CDR = \frac{\phi_{FH} \; R_{FH}}{\gamma \; T_o} \geq 1.0$$

Equation 6.27: Capacity-to-demand ratio for a headed stud.

Where:

ϕ_{FH} = resistance factor for headed stud tensile resistance

R_{FH} = nominal tensile resistance of headed studs in final facings

γ = load factor selected for verification

T_o = as defined earlier

(3) Verify that the compression on the concrete behind headed studs is within tolerable limits by assuring that:

$$A_H \geq 2.5 \; A_S$$

Equation 6.28: Verification for sufficient cross-sectional area of headed stud.

And:

$$t_{SH} \geq 0.5 \; (D_H - D_S)$$

Equation 6.29: Verification of sufficient thickness of headed stud.

Where:

A_H = Cross-sectional area of the stud head

D_{SH} = Diameter of head

D_{SC} = Diameter of shaft

(4) Provide sufficient embedment to headed studs beyond the middle of the facing section and behind the WWM in the final facing.

(5) Provide a minimum of 2 in. of cover over connectors or as suggested by ACI 318. Select worst condition.

(6) If threaded bolts are used instead, replace A_S above with the effective cross-sectional area of the bolts, A_E:

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

Equation 6.30: Effective cross-sectional area of threaded bolts.

Where:

D_E = effective diameter of the bolt core

n_t = number of threads per unit length

Alternatively, Table 6.8 could be used to estimate R_{FH}.

Table 6.8: Headed Stud Resistance in Tension, R_{FH} (Final Facing)

Headed Stud Shaft Diameter, D_{SC} (in.)	R_{FH} [(1)] (kip)
3/8	28
1/2	48
5/8	75
3/4	108
7/8	146

(1) Based on four (4) headed studs and Grade 60 steel. For Grade 75 steel, multiply values in this table by 1.24.

6.6.8 Step 7g Other Facing Design Considerations

Several other items should be considered during the facing design:

(1) Select bearing plates of mild steel with minimum Grade 36.

(2) Specify the nuts at the nail head to be heavy-duty, hexagonal, to include corrosion protection, and to be compatible with corrosion protection systems as specified by the manufacturers.

(3) Select beveled washers at the nail head to be of plain or galvanized steel. If the plate and other hardware elements are not within the ranges for strength and dimension provided earlier, a formal calculation of capacities should be performed. Note that some proprietary systems employ spherical seat nuts that do not require beveled washers.

There are examples of permanent walls where the initial shotcrete facing also serves as the final facing. In these cases, the soil nail head protrudes from the final facing and no headed studs are installed. The verifications for flexural and punching shear resistances would be required once. The parameter C_f selected for verifications of bending capacity should be that corresponding to an initial facing. The verification for headed stud resistance is unnecessary in this case. The engineer would have to design for a minimum cover of 2 in. on the outside face and 3 in. on the soil side over the reinforcement in the facing and over the nail head elements. Because the soil nail head protrudes, effective, long-term corrosion protection for the soil nail head may also be provided with external caps, grease compounds, etc.

6.7 Step 8 Service Limit States (Deformations)

6.7.1 Step 8a Evaluate Wall Lateral and Vertical Displacements

A soil nail wall should be designed to limit movements of the wall within tolerable ranges. Estimate the maximum lateral and vertical displacements of the ground behind the wall using the approximate method presented in Chapter 5. These limits are listed in the SNAP-2 output; however, the program does not evaluate deformations.

For complex geometries or when the wall will be constructed adjacent to a critical structure that lies within D_{DEF} (Figure 5.16), the engineer could use numerical methods to evaluate the magnitude and distribution of these deflections. The most common numerical methods are 2D finite element method (FEM) and the finite difference method (FDM).

Numerical methods may be especially useful when the soil nail wall resists relatively large surcharge loads. Plasticity-based soil models should be preferred over elasticity-based models to predict vertical displacement of the ground behind the soil nail wall. In certain cases where the excavation induces drainage of pore pressures within the soil, long-term compression of the soil and the ensuing settlement and lateral movement must also be considered in the analysis.

The maximum lateral movement should be limited to approximately less than 0.3% H (where H = wall height) for fine-grained soils, 0.2% H for granular soils, 0.1% H for weathered rock and stiff or dense soil of the wall height for most soil types. The zone of influence that is

affected by lateral movement extends a distance D_{DEF} that can be up to 1.5 times H for a vertical soil nail wall constructed in fine-grained soils.

6.7.2 Step 8b Evaluate Lateral Squeeze (if applicable)

Verification for lateral squeeze may be necessary if a soil nail wall becomes part of a bridge abutment overlying soft soils, and unbalanced embankment loads develop behind the wall. In these cases, the lateral deflections must be evaluated. Guidance for evaluating lateral squeeze, as well as methods for stabilizing soils to prevent problems related to lateral squeeze, are presented in Samtani and Nowatzki (2006).

6.8 Step 9 Seismic Design

6.8.1 Overview

The performance of soil nail walls during earthquakes has been historically observed to be much better than that of gravity retaining structures. Observations made after recent earthquakes indicate that soil nail walls did not show significant distress or permanent deflections, although some of the surveyed walls were subjected to ground accelerations as large as 0.7g (Felio et al. 1990; Tatsuoka et al. 1997; Tufenkjian 2002). Vucetic et al. (1993) and Tufenkjian and Vucetic (2000) observed similar trends in centrifuge tests performed on reduced-scale models of soil nail walls. These findings suggest that soil nail walls have an intrinsic satisfactory seismic performance, which is attributed to their flexibility and design redundancy.

Soil nail walls still need to be fully designed for seismic loads when applicable. AASHTO (2014), Article 11.5.4.2, gives guidance about when earth-retaining structures require analysis for loads at the Extreme-Event I limit state. The criteria for MSE walls, another earth-supporting system regarded as flexible, can be considered appropriate for soil nail walls. It is stated in that article that a seismic design is not mandatory for MSE walls in AASHTO-defined Seismic Zone 1 (i.e., for a design response acceleration for 1-sec period, $S_{1D} \leq 0.15$), Seismic Zone 2 ($0.15 < S_{1D} \leq 0.30$), and Seismic Zone 3 ($0.30 < S_{1D} \leq 0.50$), or for walls at sites where the site-adjusted, design PGA_D is ≤ 0.4 g, and as long as the soils behind the wall do not liquefy (mainly cohesionless soils), or sustain significant strength degradation (mainly fine-grained soils) during dynamic loading.

Once it has been identified that seismic analyses are necessary, the steps listed in the next section, which are based on Kavazanjian et al. (2011), are recommended for the seismic evaluation of soil nail walls.

6.8.2 Step 9a Select Design Seismic Parameters

(1) Obtain seismic parameters (PGA and S_1) from AASHTO (2014) for a 1,000-year return period for a reference Site Class B, as described in Section 4.9. Select alternative values for a different return period if required by Owner.

(2) Determine the average shear-wave velocity ($v_{_bar}$), average N_{60} ($N_{_bar}$), or average S_{u_bar} (for cohesive soils) in the upper 100 ft from the site ground surface. Consider the 100 ft from the planned top of wall.

(3) Assign a Site Class (from A through F) using Table 4.8.

(4) If Site Class is between A and E, obtain site modification factors F_{PGA} and F_v from Tables 4.9 and 4.10, respectively. If Site Class is F, perform site-specific site response analysis (beyond the scope of this manual, see Kavazanjian et al. 2011).

(5) Calculate the design, site-corrected peak ground acceleration and 1-sec response acceleration as: $PGA_D = F_{PGA}\, PGA$ and $S_{D1} = F_v\, S_1$.

(6) Determine the maximum seismic coefficient, k_{max}, which will be the basis for input in the limit-equilibrium analysis, as follows:

For Site Classes C through E:

$$k_{max} = PGA_D$$

Equation 6.31: Design, site-corrected peak ground acceleration (Site Classes C through E).

For Site Classes A and B (not typical conditions for soil nail walls):

$$k_{max} = 1.2\, PGA_D$$

Equation 6.32: Design, site-corrected peak ground acceleration (Site Classes A and B).

6.8.3 Step 9b Adjustment of Design Seismic Coefficients

Wall height

If the wall is 20 ft high or more, the maximum seismic coefficient can be reduced. This reduction is done to account for the decrease of seismic wave intensity as the waves propagate through the soil behind the wall from the base to the top of the wall. The longer

the path the waves traverse, the larger the wave scattering (or spatial incoherence) and the larger the intensity reduction. The adjustment is effectively an averaging of the seismic wave intensity over the height of the wall. The adjusted k_{max} value is the seismic coefficient, k_h, which is defined as:

$$k_h = \alpha \times k_{max}$$

Equation 6.33: Seismic coefficient corrected for wall height.

Where α is a height adjustment factor defined as:

$$\alpha = 1 + 0.01\ H\ (0.5\ \beta - 1)$$

Equation 6.34: Wall height correction factor.

Where:

H = wall height

β = S_{1D}/k_{max}

Seismic deformations

Because soil nail walls are more flexible than gravity walls and other similar systems, it is reasonable to expect that these walls will sustain some lateral displacement during earthquakes, and that these displacements can be tolerated. Experience suggests that permanent deformations of relatively flexible walls, including soil nail walls, tend to be small when $0.5 \leq k_h/PGA \leq 0.67$, and as long as the strength of soils behind the wall does not decrease significantly during strong ground motions.

The first step is to assess whether the wall system can accept seismically-induced, lateral, permanent movements. Categories of tolerable movement to consider are (Kavazanjian et al. 2011):

- Rigid, retaining-earth systems that are unable to sustain permanent displacements (e.g., these wall types have certain structural features that cannot sustain large deformations, or they support an adjacent structure that is sensitive to deformations).
- Wall systems that are able to sustain between approximately 1 and 2 in. of lateral displacement.
- Wall systems that are able to sustain more than 2 in. of lateral displacement.

The next step is to estimate the permanent wall deformations, which can be estimated by evaluating the displacement of a sliding mass of soil in a manner analogous to the Newmark

method (1965) or sliding of a rigid block subjected to cyclic loading. Details of this method can be found in Kavazanjian et al. (2011). As a simplification, the permanent wall displacement after an earthquake, d_e, can be estimated as follows:

$$\log(d_e) = -1.51 - 0.74 \log\left(\frac{k_y}{k_{max}}\right) + 3.27 \log\left(1 - \frac{k_y}{k_{max}}\right) - 0.80\, k_{max} + 1.59\, PGV$$

Equation 6.35: Estimated wall seismic displacement.

Where:

k_y = yield acceleration, as calculated from overall stability analysis

PGV = peak ground velocity, which is estimated as PGV = 38 S_{D1}

The yield acceleration is the seismic coefficient that will result in $FS_{OS} = 1.0$ in stability analysis. This value is obtained when the wall is being designed for static loads using a limit-equilibrium-based computer program.

Finally, the modified seismic coefficient is calculated as follows:

- Rigid retaining wall systems:

  $$k_h = k_{max}$$

  Equation 6.36: Seismic coefficient for rigid wall.

- Walls able to sustain 1-2 in. of lateral displacement:

  $$k_h = 0.5 \times k_{max}$$

  Equation 6.37: Seismic coefficient assuming wall can tolerate a lateral movement up to 2 in.

- Walls able to sustain more than 2 in. of lateral displacement:

  $$k_h = r \times k_{max}$$

  Equation 6.38: Seismic coefficient assuming wall can tolerate a lateral movement greater than 2 in.

  Where r is a ductility reduction factor defined as:

  $$r = k_y / k_{max}$$

  Equation 6.39: Reduction factor.

It can be considered that for rigid walls, r = 1.0, and for walls able to sustain more than 2 in. of lateral displacement, r = 0.5.

6.8.4 Step 9c Evaluate overall stability with seismic loads

Perform overall stability analysis as discussed in previous sections while adding the seismic coefficient, k_h, as input. Use minimum factors of safety for seismic conditions as included in Table 5.1. Verify that the computed factors of safety for seismic conditions are met. If the minimum criterion is not met, modify nail length, and/or other design parameters as applicable and repeat analysis until the calculated factors of safety exceed minimum recommended values.

6.8.5 Alternative Evaluation of Seismic Effects on Overall Stability

The most commonly used pseudo-static procedure has been the Mononobe-Okabe method, which is an extension of the Coulomb theory (Mononobe 1929; Okabe 1926). This method may be used as an alternate method to evaluate the seismic stability of soil nail walls. However, this method has limitations (Kavazanjian et al. 2011). Despite these limitations, the method continues to be used due to its simplicity and acceptable results for certain conditions.

The Mononobe-Okabe method, which is described by Seed and Whitman (1970), Richards and Elms (1979), and Kavazanjian et al. (2011), was originally developed for gravity walls but may be used for soil nail walls. The following is assumed in this method:

- The wall and the soil mass behind it act as a rigid block
- Active earth pressures develop behind the wall
- Soils are cohesionless
- No pore pressures develop behind the wall

In this method, the inertial forces acting on a retaining earth system in an earthquake can be considered as equivalent pseudo-static forces in overall stability evaluations using simplified procedures. These pseudo-static forces act at the centroid of the potentially unstable soil block being analyzed (Figure 6.9). The horizontal component (k_hW) and the vertical component (k_vW) of the equivalent pseudo-static force are shown:

Where:

W = weight of soil wedge

k_h = horizontal seismic coefficient

k_v = vertical seismic coefficient

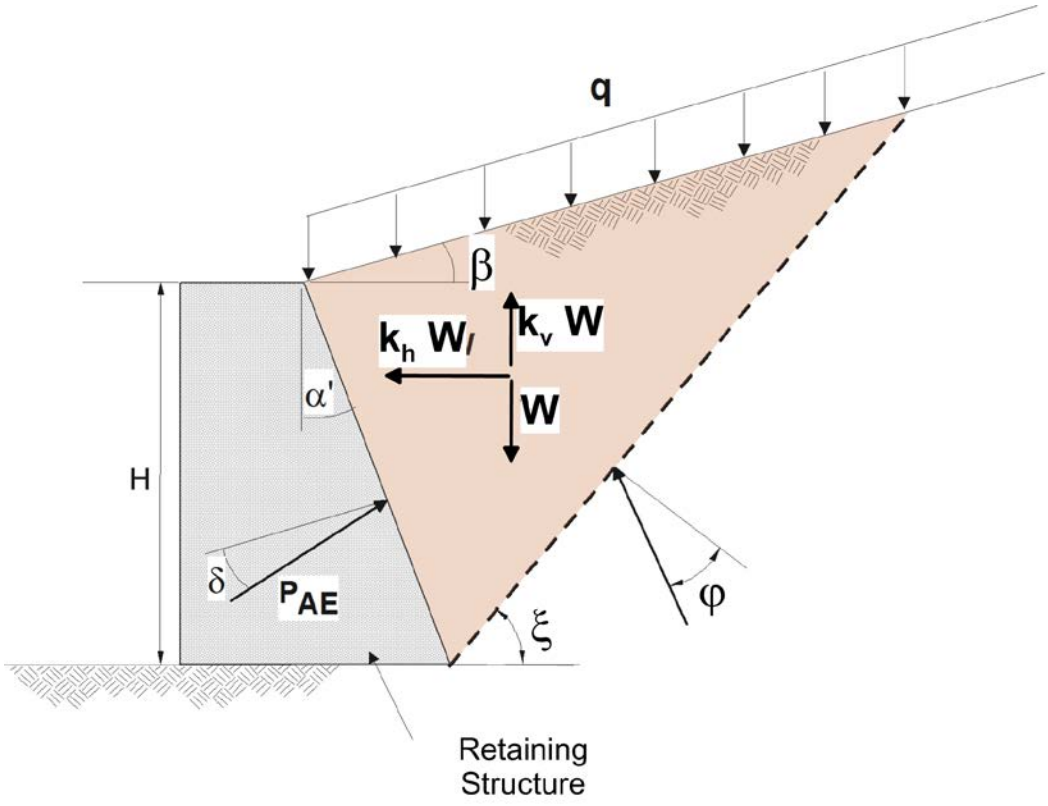


Figure 6.9: Illustration. Forces acting behind a soil wedge (Mononobe-Okabe Method).

The non-dimensional seismic coefficients are a function of the horizontal and vertical components of the ground acceleration, respectively. The horizontal and vertical components are represented by the seismic coefficients, k_h and k_v. As mentioned, the vertical component is disregarded.

The total active thrust, P_{AE}, behind the soil wedge behind a wall is (Ebeling and Morrison 1992; Kavazanjian et al. 2011):

$$P_{AE} = \frac{\gamma_s H^2}{2} K_{AE} (1 - k_v)$$

Equation 6.40: Total active thrust per Mononobe-Okabe method.

Where:

γ_s = unit weight of soil behind wall

H = wall height

K_{AE} = active pressure coefficient

If the ground behind the wall has a slope (Figure 6.9), the active pressure coefficient can be calculated as follows:

$$K_{AE} = \frac{\cos^2(\varphi_f - \omega - \alpha')}{\cos\omega \ \cos^2\alpha' \ \cos(\alpha' + \delta + \omega) \ D}$$

Equation 6.41: Active earth pressure coefficient per Mononobe-Okabe method.

Where:

φ_f = angle of internal friction of soil behind wall

α' = batter angle (from vertical) of the wall internal face

β = backslope angle

δ = friction angle of the wall-soil interface

ω = angle relating seismic coefficients as defined by:

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

Equation 6.42: Relation between horizontal and vertical seismic coefficients.

D is defined as:

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

Equation 6.43: Term D as used in Mononobe-Okabe equations.

The slip plane behind the wall is oriented at an angle ξ, which can be much flatter than for static conditions (Zarrabi-Kashani 1979; Kramer 1996; Kavazanjian et al. 2011). $K_{AE,}$ based on the Mononobe-Okabe method, can also be estimated from Figure 6.10.

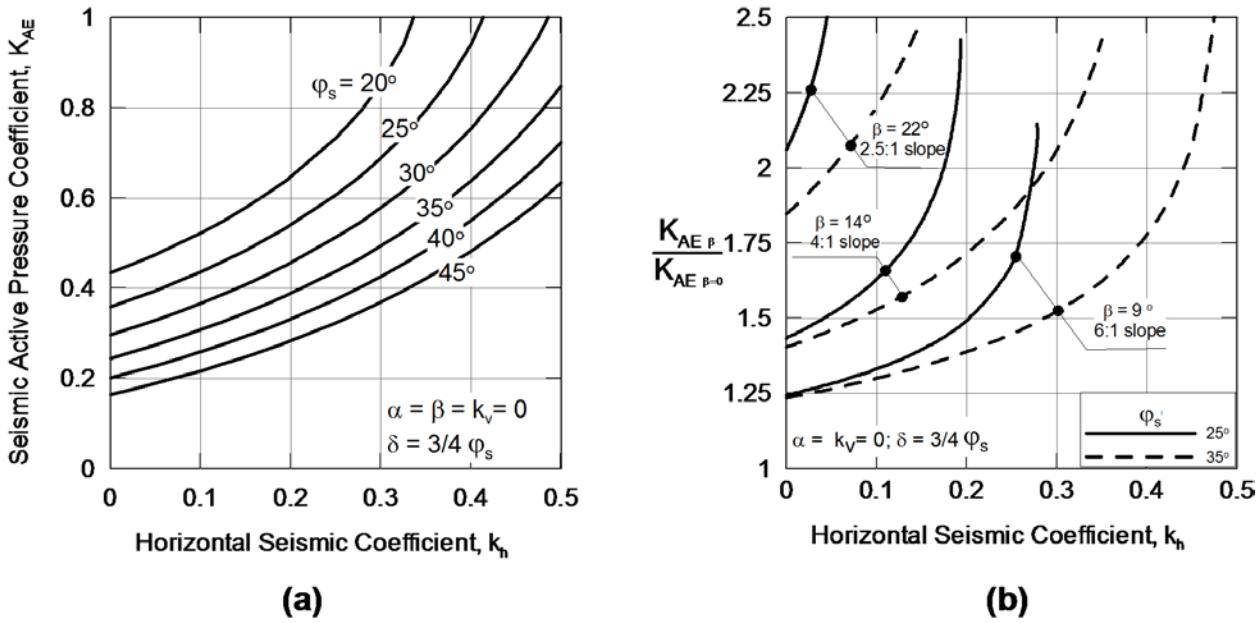


Figure 6.10: Chart. Seismic active pressure coefficients: (a) horizontal backslope, and (b) correction for non-horizontal backslope.

Figure 6.10a shows the active pressure coefficient as a function of the seismic coefficient, k_h, and the friction angle, φ_s, ranging from 20º to 45º; a horizontal backslope, $k_v = 0$; and $\delta = 0.75\ \varphi_s$. Figure 6.10b presents a correction factor for the active pressure coefficient when the backslope is not horizontal.

Note that the Mononobe-Okabe formulation does not provide solutions for increasing values of the backslope angle or the seismic coefficient. Alternative methods to counter this limitation can be found in NCHRP (2008).

The original Mononobe-Okabe method did not consider cohesive soil behind the wall. NCHRP (2008) and Kavazanjian et al. (2011) provide guidance to include cohesion in the estimation of seismic pressures. Small values of cohesion can significantly reduce the original values of the seismic active pressure coefficient provided by the Mononobe-Okabe method. The amount of cohesion to include in this formulation is limited to the values included in Table 6.9:

Table 6.9: Maximum Values of Apparent Cohesion to Use with Mononobe-Okabe Method (NCHRP 2008)

Percent Passing #200 Sieve by Weight (%)	Maximum Apparent Cohesion
5 - 15	50 psf
15 - 25	100 psf
25 - 50	200 psf

Figure 6.11 shows corrected values of K_{AE} as a function of cohesion (the latter is normalized as $c/\gamma_s H$) for $\varphi_s = 35°$ and $\beta = 0$. NCHRP (2008) includes additional plots for $\varphi_s = 30°$ to 40°.

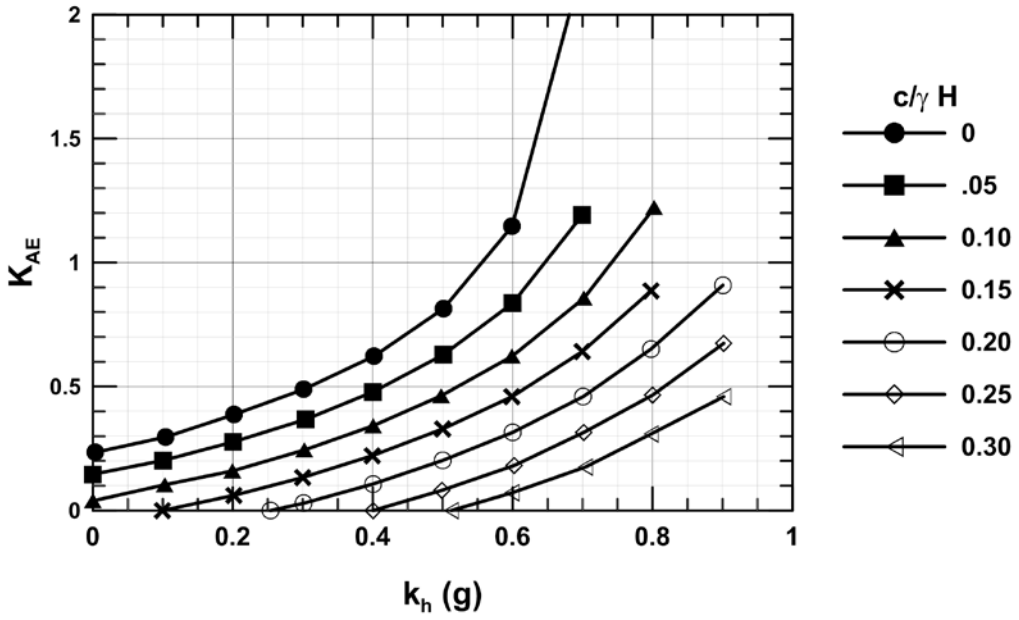


Figure 6.11: Chart. Effect of cohesion on the seismic active earth pressure coefficient. Source: NCHRP (2008).

6.9 Step 10 Drainage and Drainage Details

6.9.1 Introduction

The design engineer must provide adequate subsurface and surface drainage features in all walls unless it is determined that such features are not needed for a specific project. Soil nail wall facings are assumed to be relatively lightly loaded and therefore lightly reinforced, and

are not designed to resist hydrostatic forces. It is assumed that incidental water behind the wall face will be handled by the strip drains.

Uncontrolled surface water runoff and/or groundwater can cause difficulties during construction, increase the cost and duration of construction, impair long-term integrity, negatively affect the performance of soil nail walls, and degrade aesthetics. 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 mentioned that soil nail walls perform significantly better when an effective drainage system is installed to control water levels behind the wall. Drainage systems are discussed in Chapter 3. This section focuses on relevant considerations for design and constructability of drainage components.

6.9.2 Short-Term Surface Water and Groundwater Control

Dewatering measures during construction include, at 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 often used to control surface runoff toward the excavation face and to limit infiltration into the retained soil. It is important to control infiltration of water behind the wall during construction.

Diverted flow over the face may also cause erosion and may lead to localized instability before the initial face is constructed. Interceptor ditches must be properly constructed with suitable slopes to promote positive drainage while directing flow away from the excavation. Improperly designed ditches may create localized erosion, ponding, or infiltration. The decision to use an interceptor ditch must be made depending on the site conditions. An interceptor ditch is generally unnecessary if the area above the wall slopes away from the wall face or if the potential for significant runoff or infiltration behind the wall is small.

In cases where significant seepage is expected due to perched water conditions, wells or well points may be installed for temporary dewatering during construction. The wells would typically be installed beyond the length of the nails to prevent damage of the wells during nail drilling. Wells would obviously impact the construction costs and in such cases soil nailing might not be a cost-effective and practical solution.

The temporary control of surface runoff, ponding, infiltration, and groundwater are the Contractor's responsibility. The Contractor should provide drainage to convey collected water away from the wall by gravity or with the use of sump pumps and similar systems. If the Owner performs the geotechnical exploration, sufficient information should be collected during the design phase to identify and communicate the need for drainage, and to facilitate adequate selection and sizing of drainage and dewatering elements by the Contractor.

6.9.3 Long-Term Surface Water and Groundwater Control

6.9.3a Control for Low Groundwater or Low Volumes of Seepage

Specific design calculations based on seepage analysis are often not needed for designing strip drains. The design engineer most commonly would specify a type of strip drain and its spacing based on judgment and experience. The spacing of strip drains is typically selected to be between 1.0 S_H and 2.0 S_H with a spacing equal to S_H being more common. Spacing too close reduces the amount of soil facing in contact with the shotcrete. In these cases, problems with sloughing of shotcrete may arise during application. The design engineer also needs to consider available commercial widths of strip drains.

Weepholes can also be used to collect and discharge groundwater from strip drains. The diameter and spacing of weepholes are selected by the designer based on experience and judgment, and no specific calculations are performed. Weepholes are typically 2- to 4-in. diameter PVC pipes and have a length between 12 and 16 in., depending on the thickness of the facing. They are spaced 8 to 10 ft along the base of the wall. The back of weepholes must be protected with geotextile to prevent the migration of soil from behind the wall. A filter plug consisting of at least 1 ft^3 of filter aggregate wrapped in geotextile should be placed against each weephole behind wall.

Weepholes should also be positioned where localized seepage is encountered or anticipated. Where weepholes are provided, the designer must consider control of surface water flow in front of the wall in its final condition. The effect of freezing may also need to be considered in some climates when dealing with control of surface water.

As an alternative to weepholes, soil nail walls may have an underdrain system, where strip drains are connected to a short pipe (similar to a weephole but lying below ground) installed through the wall and to a toe drain, running parallel to the toe of the wall. The underdrain system commonly includes Schedule 40 PVC solid and slotted drainage pipe. The toe pipe is surrounded by clean, coarse, gravel-sized aggregate that must be free of fine particles. The aggregate trench must be wrapped with geotextile filter fabric, as the use of a geotextile directly over the slotted pipe must be avoided. The geotextile around the aggregate has the function of preventing the removal of fines by piping from the natural soil surrounding the aggregate trench. To prevent clogging, the geotextile should meet filter criteria (Holtz et al. 2008) and be selected from a range of available commercial products based on the particle size distribution of the soil being retained. The aggregate around the pipe has the function of allowing the collected groundwater to flow along the pipe and more readily enter into the slots in the collector pipe. This aggregate also provides a redundant pathway to groundwater discharge.

Some schematic details are provided in Figures 6.12 and 6.13. Some of the details might not be practical in all situations. For example, in steep terrain, it may be difficult to place a concrete ditch above the wall as shown.

Placing drain strips against the exposed face of highly irregular excavations may be difficult or impractical. If voids exist between the excavation face and the strip, the shotcrete application may damage the strip drain. In such cases, the strip drains may be installed between the initial facing and the final facing. Holes must then be drilled through the initial facing to allow flow to the strip drains.

6.9.3b Control for High Groundwater

When groundwater needs to be permanently lowered, the design engineer should perform seepage analyses and determine the expected flow rate immediately behind the wall. Strip drains and their spacing need to be selected as a function of the calculated flow rate, strip drain permittivity, and final piezometric heads considered acceptable. A redundant and/or over-sized drainage system should be used where the groundwater table exists behind a soil nail wall. If a seepage analysis and a formal design become necessary, the goal in designing the drainage system should be to maintain the groundwater levels at least 2 ft below the soil nails and base of the facing at all times. If this is not possible, the effects of elevated groundwater must be carefully evaluated for internal and global stability, added pressures on the facing, and increased corrosion potential.

When strip drains are not sufficient to lower the groundwater levels to adequate levels, drain pipes must be considered (Figure 6.12). Drains are installed horizontally or with mild upslope (5 to 10 degrees from the horizontal) to reduce pore water pressures within the mass of reinforced soil and soils beyond. This solution may be appropriate where the groundwater behind a wall is elevated and the potential for internal or global instability can be mitigated through drainage of pore pressures in the retained soil mass. Drain pipes can also help to drain perched water.

Drain pipes consist of 2-in. diameter PVC slotted or perforated tubes exiting through the face of the wall (Figure 6.13). Drain pipes of larger diameter or in multiple rows may be needed as indicated by seepage analysis. The lengths of the drain pipes depend on the application, and subsurface and groundwater conditions existing behind the wall; and will be determined by the lowered level of groundwater selected for design (Figure 6.12).

Drain pipes with lengths varying from 1 to 3 ft can be installed to provide drainage of shallow or perched groundwater occurring erratically close to the facing. Longer pipes may be installed if the groundwater is high behind the wall. Drain pipes are commonly installed

with a density of one per approximately 100 square foot of face area. Drain pipes should be plugged or capped while shotcrete is being applied to prevent clogging (Figure 6.13).

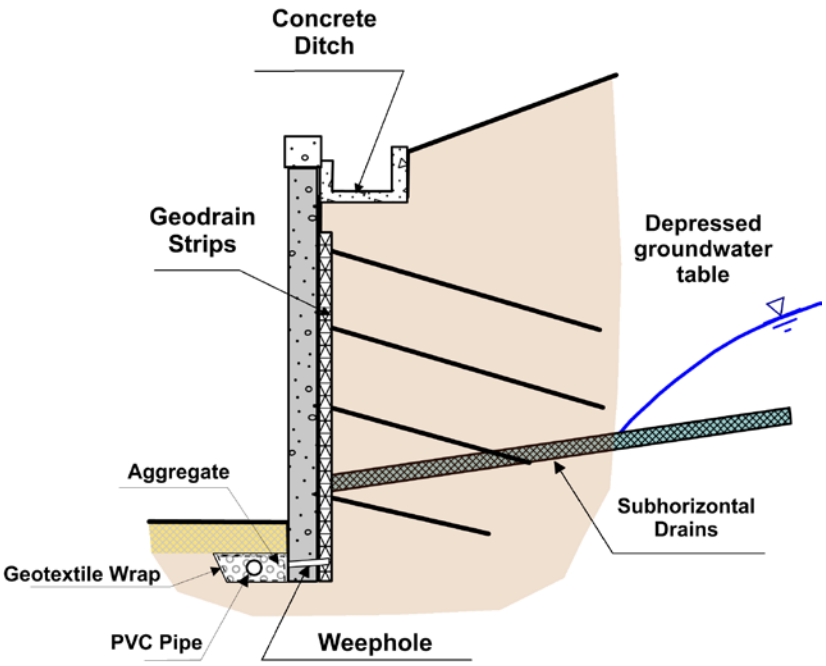
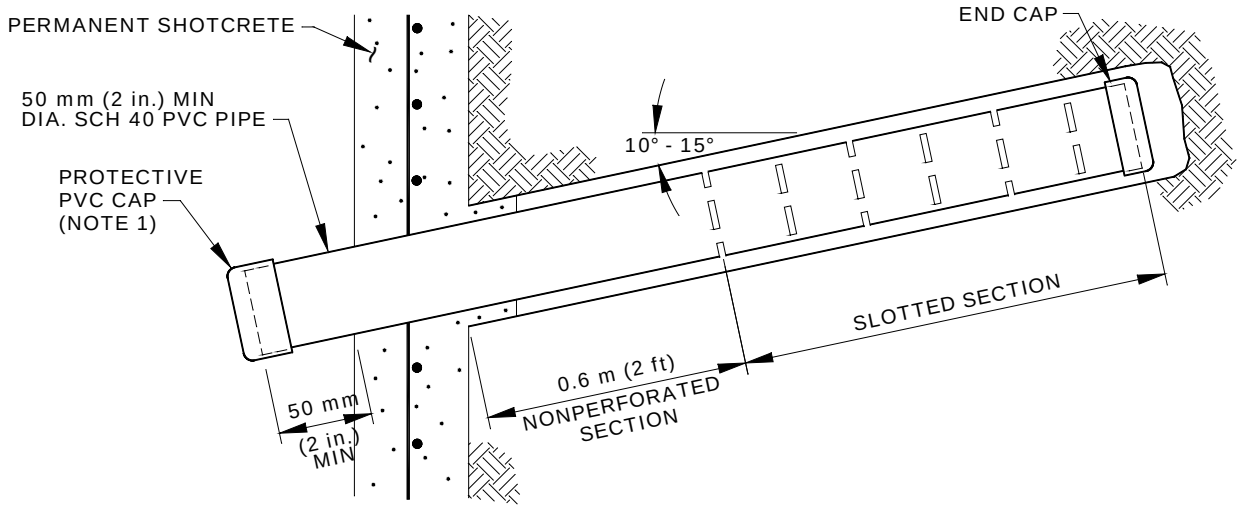


Figure 6.12: Illustration. Drainage of soil nail walls.

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 overall stability and/or sliding, and may adversely impact the internal stability by affecting soil/nail interaction.

6.9.3c Permanent Surface Water Control

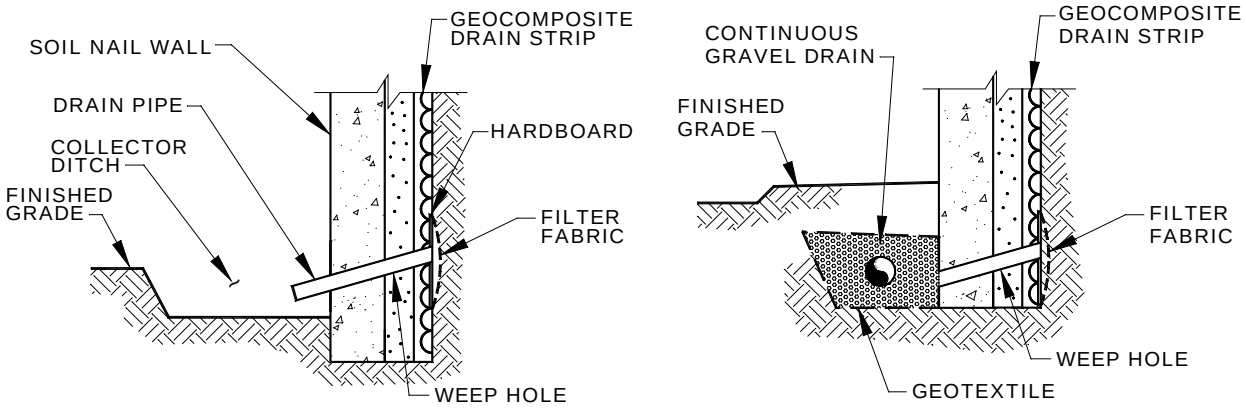
Permanent surface water control measures include installing an interception ditch behind the wall to prevent surface water runoff from infiltrating behind the wall or flowing over the wall edge.



NOTES

1. PROTECTIVE CAP NEEDS TO BE REMOVED AFTER FINAL SHOTCRETE IS APPLIED
2. SPACING OF DRAINS IS TYPICALLY 3.3 m (10 ft)

TYPICAL DRAIN DETAIL



TOE DRAIN

FOOTING DRAIN

Figure 6.13: Illustration. Typical drain pipe details. Modified after Byrne et al. (1998).

6.10 Specialty Items

6.10.1 Stepped Walls

There are a number of situations where it may be necessary or convenient to use stepped or benched soil nail walls. Stepped walls may consist of two or more soil nail walls with vertical or battered facing, with horizontal setbacks between individual wall sections (Figure 6.14). This section discusses the most common: two-tier stepped walls with vertical facing; however, the discussion presented is applicable to stepped walls with any number of tiers.

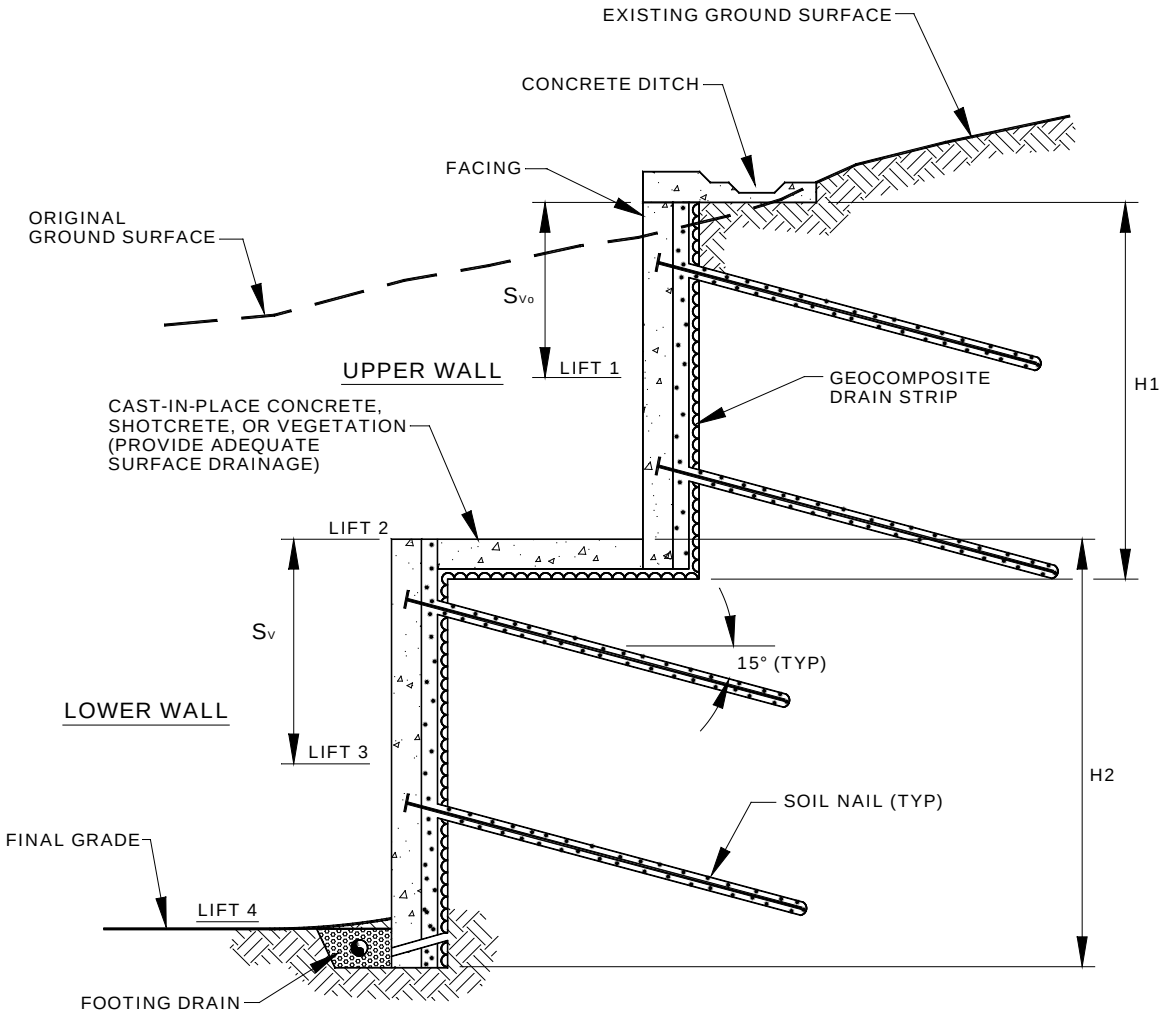


Figure 6.14: Illustration. Example of stepped soil nail wall. Source: Byrne et al. (1988).

In general, stepped soil nail walls require shorter nail lengths than non-stepped walls of the same total height, but are less frequently constructed on transportation projects. Therefore, they may serve as a cost-saving measure where the cost of additional excavation is comparatively low, and where soil nail installation is relatively expensive.

In cases where overall stability controls the design, full-height, non-stepped vertical face walls may require the use of relatively long soil nails with significant resistance requirements, and with significant impact to construction costs. In other cases, the potential for global instability or bottom heave may make it impossible to use a full-height soil nail wall. Stepped soil nail walls may be used in these circumstances. They would typically include a relatively wide horizontal setback between the upper and lower wall faces.

Stepped soil nail walls may offer aesthetic advantages by permitting the installation of planters between the walls, especially if used in combination with sculpted shotcrete or other architectural facing. Ultimately, either stepped walls or battered walls can be used to solve similar problems or project constraints, although stepped walls may offer certain construction advantages.

A stepped wall 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), the internal stability of each wall is independent of the other and they can be analyzed separately.

If the horizontal setback is smaller than the height of the lower wall, the lower wall must be analyzed considering the upper wall. For the purposes of designing the soil nails, the upper wall may be modeled using a soil nail wall design program as a combination of vertical surcharge and shear forces arising from the lateral earth pressures acting on the MSE wall. The vertical surcharge and the shear forces will be applied to the soil nail reinforced soil. This approach is a simplification, and is likely conservative, as it disregards the internal strength of the MSE wall. However, with this approach, the slip surfaces estimated from analyses of the soil nail wall may be incompatible with the slip surfaces obtained from conventional MSE wall design. A more accurate approach is to include both types of walls and reinforcements in an analysis conducted with a general slope stability program that can model both reinforcement types, hence considering the contribution of soil nails, MSE wall reinforcement, and the soil within and behind the reinforced zone of the MSE wall.

Surface drainage of the bench between the walls must be carefully addressed to avoid ponding and infiltration into the lower wall. Therefore, the bench should be designed with a suitable slope and lined as needed. If vegetation is planted on the bench, the potential water infiltration from sprinkler systems must be considered.

6.10.2 Composite Walls

The most typical composite wall type is a combination of a soil nail wall with ground anchors (Figure 6.15). In this case, nails and ground anchors are installed as excavation proceeds from top to bottom.

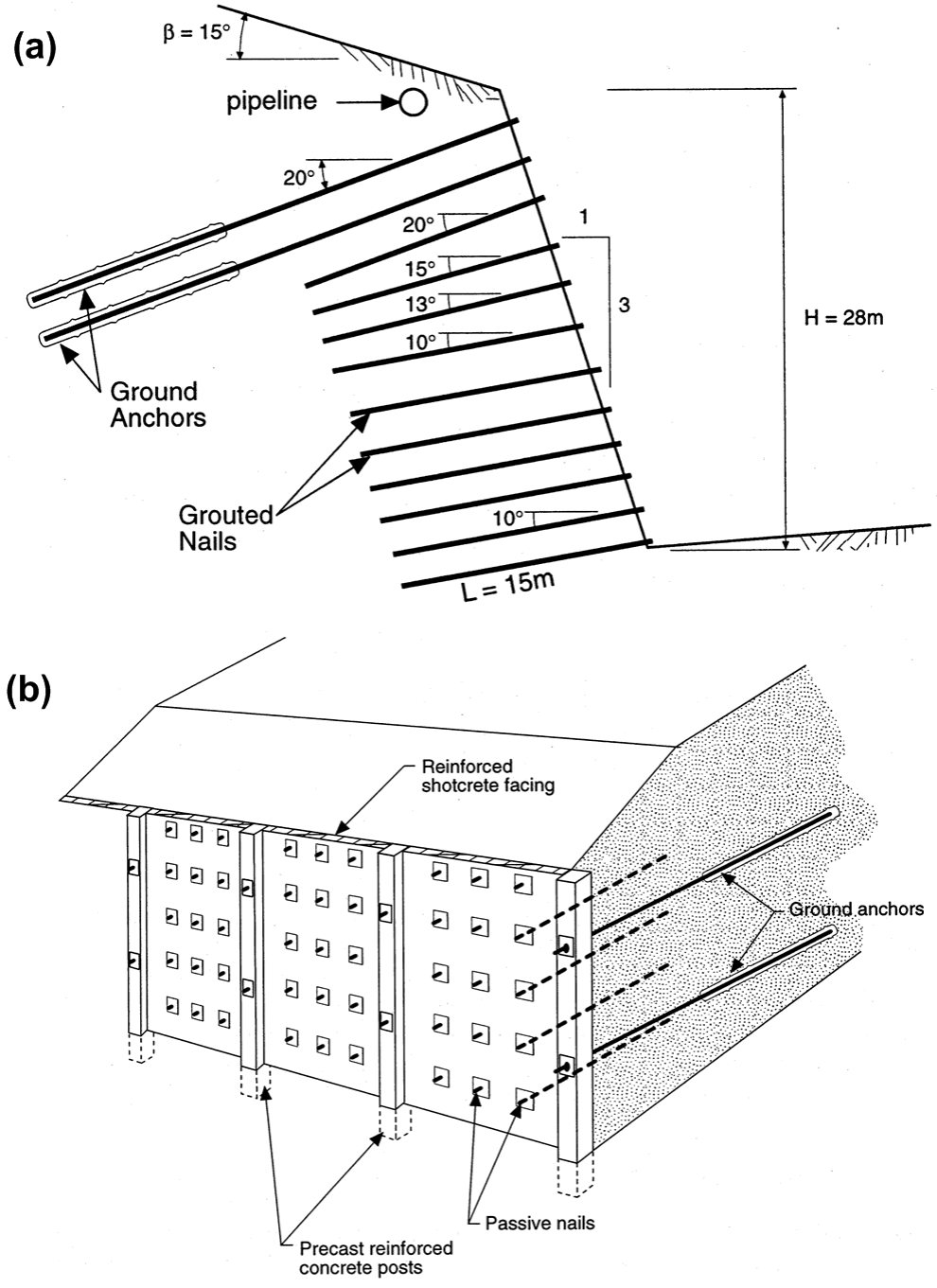


Figure 6.15: Illustration. Composite wall structures: (a) cross section, and (b) soil nail walls with concrete piles. Source: Byrne et al. (1988).

Figure 6.15a depicts a soil nail wall with ground anchors installed near the top of the wall. This configuration could be used to control movements of the ground surface above a soil nail wall. However, using ground anchors to control movement of soil nail walls is relatively rare because most structures or utilities can tolerate ground movements occurring behind soil nail walls. Also, in most cases where an especially sensitive structure exists near the excavation, movements can be reduced to tolerable levels by using soil nails with an increased cross section (i.e., the soil nail is stiffer and deforms less), or by underpinning the structure itself. One case where ground anchors may be specified at or near the top of a soil nail wall is that of a very deep excavation adjacent to a settlement-sensitive structure, and where underpinning may be impractical, or where the structure applies horizontal forces that cannot be countered by soil nails.

Ground anchors cannot typically be tensioned directly against initial shotcrete facing due to the potential for punching shear and bending failure of the facing under elevated ground anchor loads. In these situations, appropriate bearing pads must be devised or the thickness and reinforcing of the initial facing must be increased to support and transfer the anchor load to the ground. Space limitations may restrict the available options to support the ground anchor load and their dimensions; therefore, they must be considered by the Owner during preparation of the project documents, and should not be left entirely to the Contractor.

Ground anchors are typically installed at an angle from horizontal. Therefore, the stability of the wall under the vertical component of the ground anchor load during excavation of successive lifts must be considered. This is especially important if the ground anchors have inclinations greater than 20 degrees from horizontal, but it may still be an important issue for shallower inclinations. To mitigate potential movement of the wall under the vertical component of the anchor load, excavation may proceed following a checkerboard pattern, or vertical micropiles may be installed along the wall alignment extending below the proposed bottom of excavation.

Ground anchors are more commonly introduced in soil nail walls when there is a potential for global instability that would be impractical to address using the soil nails themselves. If the ground anchors are not necessary for temporary stability during excavation, they would preferably be installed near the bottom of the soil nail wall, and tensioned after the final facing is constructed. In cases where a significant number of ground anchors is necessary, they can be installed along the full height of the wall. In this case, transfer of the anchor load to the ground can be achieved by means of cast-in-situ or precast concrete posts (Figure 6.15b). This is the case where the soil-nailed sections of the wall effectively served as lagging between the soldier piles. The designer must consider if a solution entirely based on ground anchors without use of soil nails is more practical in such cases.

Because the soil nails and the ground anchors in combined systems are intended to address different limit states, the recommended design approach is to analyze the wall with soil nails only, but limiting the slip surfaces considered in the analysis to shallow surfaces (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 slip 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. Note that only a few soil nail wall design programs allow including ground anchors in the model.

Chapter 2 presented other types of composite walls including hybrid soil nail/MSE (Figure 2.6) where a soil nail wall supports an MSE wall. The soil nails can be designed using soil nail design programs.

6.10.3 Strut-Nails

6.10.3a Overview

Strut-nails or toe-nails are drilled and grouted bars installed at steep angles (up to about 15 or 20 degrees to 45 degrees from the vertical) near the bottom of the excavation to provide vertical support to the wall facing (Figure 6.16). Strut-nails can be used when the weight of the facing or uncommon, heavy loads applied at the top of the facing are large, and the required bearing resistance cannot be provided by the soils below the wall. Strut-nails are relatively short (a fraction of the wall height) and act in compression. The strut-nails are connected to the wall with a bearing plate and washers. Strut-nails are similar to Case I micropiles in terms of their construction and performance objectives.

Design procedures developed specifically for micropiles (Sabatini et al. 2005) can be used for the design of strut-nails. The design of these elements is straightforward as it is based on force equilibrium. The design can be accomplished by hand calculations after the design of the regular soil nails is completed. The forces to consider in this problem are (Figure 6.16): the facing weight (W), the compressive force in the strut-nail (C_{ST}) and the tensional force in the nail immediately above the strut-nail (T_T). The vertical component of C_{ST} must be equal in magnitude to the sum of W and the vertical component of T_T to achieve equilibrium. Note that T_T must be added to the axial load T of the soil nail head as calculated in the conventional soil nail wall design. The total axial force at the lower nail may require that its size be increased. The vertical component of the forces in the soil nails above are not considered in the design of the strut-nail because these forces are assumed to be supported by friction resistance between the facing and the soil. However, this assumption would be invalid if the soil nails above had significant inclination.

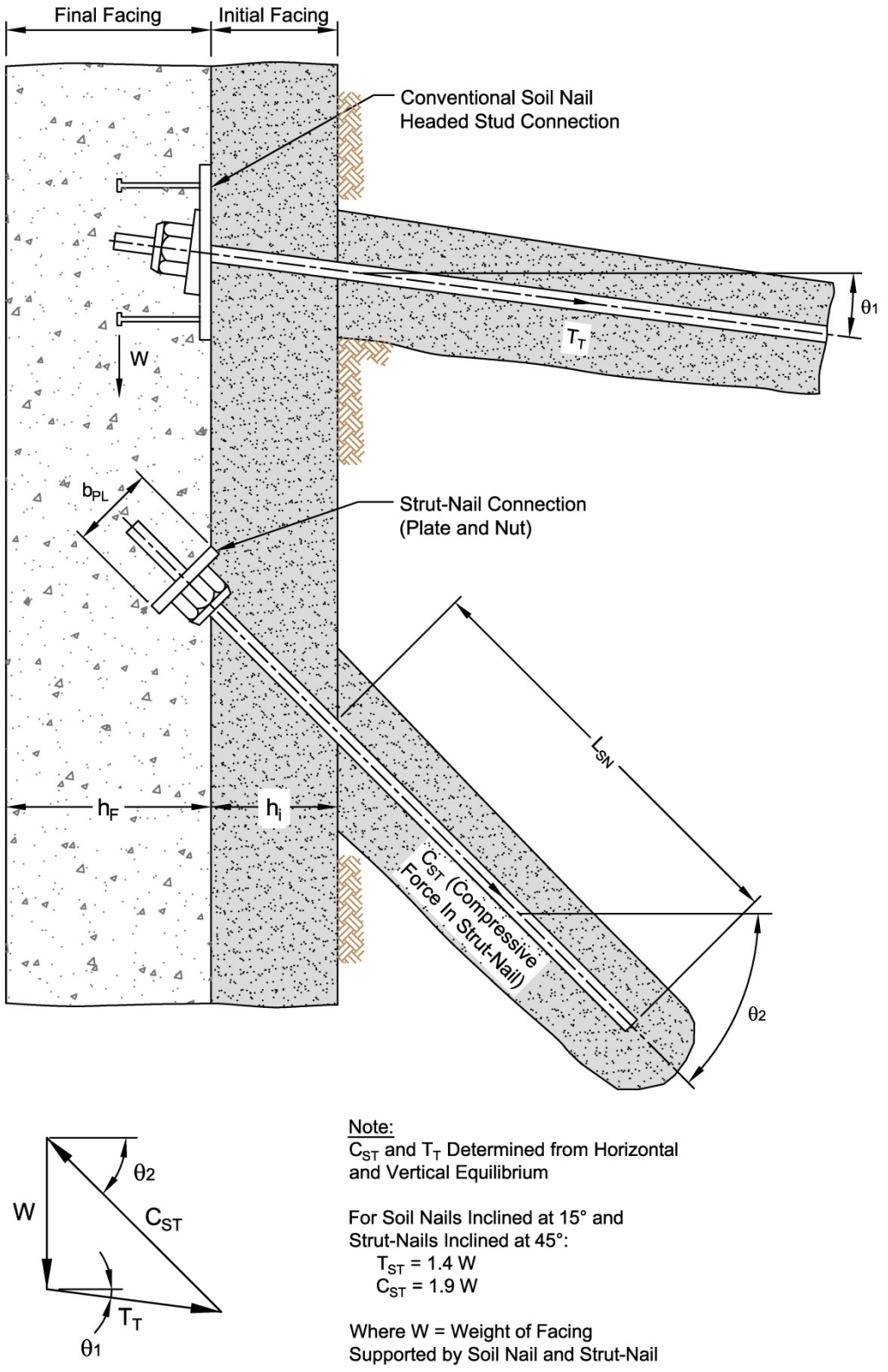


Figure 6.16: Illustration. Strut-nail concept. Source: Byrne et al. (1998).

The development of mutually opposing forces (i.e., the horizontal components of T_T and C_{ST}) generates a bending moment in the facing segment between the strut-nail and the nail row immediately above (Figure 6.16). It is important to check if the facing bending capacity is greater than this moment. The maximum moment due to these forces occurs at the nail above the strut-nail, and is additive to the moments obtained in conventional soil nail wall design. The magnitude of the added moment will decrease as the strut-nail is installed closer to the nail above. To reduce the added bending moment, strut-nails should be installed close to, or staggered within, the bottom row of nails.

Buckling of strut-nails may occur if the strut-nails are too close to the excavation face, or if the ground between the excavation face and strut-nails does not provide sufficient confinement. Buckling of strut-nails can be evaluated with procedures developed for micropiles (Sabatini et al. 2005). If buckling is found to be an issue, a stub casing may need to be inserted in the upper few feet of the strut-nail to increase its buckling resistance.

6.10.3b Applications

One application of strut-nails is to temporarily support the weight of the facing of the initial excavation lift that becomes unsupported when the next excavation lift is performed. This application of strut-nails is not common. For typical construction facings consisting of 4-in. thick shotcrete, experience has shown that soil nails can adequately support the weight of the facing. For thicker applied shotcrete facings, the support of the shotcrete facing by shear resistance of the nails and the bearing resistance of soils beneath the nails should be formally evaluated. In competent ground, the weight of a shotcrete facing with thickness of up to 8 to 10 in. has been successfully supported by soil nails without additional support measures.

Both hollow bars and solid bars have been used to allow the construction of soil nail walls in soils that are not conducive to this technique. The strut-nails may be installed vertically along the proposed wall alignment and extended below the proposed final excavation grade. As excavation progresses, these elements provide some lateral support to the excavated face in each lift before the soil nails and facing are installed, and also provide vertical support to the overlying shotcrete facing already installed. The spacing between these elements is typically about 2 to 3 ft.

A more typical application of strut-nails is the support of the final facing in difficult subgrade conditions. One common problem is the construction of a soil nail wall for supporting a cut extending through soil and bedrock. If the portion of the cut in rock does not need support, the soil nail wall may terminate at or slightly below top of bedrock. In such cases, strut-nails should be installed at the toe of the soil nail wall, extending into the bedrock to provide vertical support to the facing.

6.10.4 Considerations for Frost Protection

Unlike foundations, which are designed to be supported on subgrades lying below the frost penetration depth, soil nail walls are always susceptible to frost effects because of the exposed facing. Therefore, construction of soil nail walls in frost-susceptible soils would require frost protection measures.

The design of soil nail loads arising from frost action must be conservative due to the limited data available to assess the impact of frost on soil nail forces. The design engineer would need to consider: (i) preventing moisture build up behind the wall by providing porous backfill (e.g., gravel); (ii) thickening the facing to be slightly greater than the frost depth; (iii) adopting larger sizes of tendons; and (iv) applying insulation on the wall face.

Thermal insulation, as shown on Figure 6.17, can consist of extruded polystyrene (EPS) foam panels. This material is lightweight and cost effective. Significant thermal protection can be obtained with a relatively thin layer of this material: a 1-in. thick EPS panel is considered thermally equivalent to a 1-ft thick layer of gravel.

The selection of the thickness of EPS panels and other insulating materials should be based on the thermal resistance of the material (commonly referred to as the R-value). R-values express the ratio of the temperature drop across an insulating material to the amount of heat that is transferred per unit of area and per unit of time, for a given thickness of the material. The R-value depends on the intrinsic thermal resistance of the material to transfer conductive heat and is proportional to the material thickness. The higher the R-value of a material, the higher its ability to insulate. The selection of R-values or a detailed thermal design is beyond the scope of this manual.

Other factors can also be considered in design. For example, the direction in which the wall is installed can also affect the frost penetration. North-facing walls may have 1.4 to 2 times the depth of frost penetration than south-facing walls exposed to sunlight. Drainage of the ground above the wall should be provided to decrease infiltration and frost potential behind the wall.

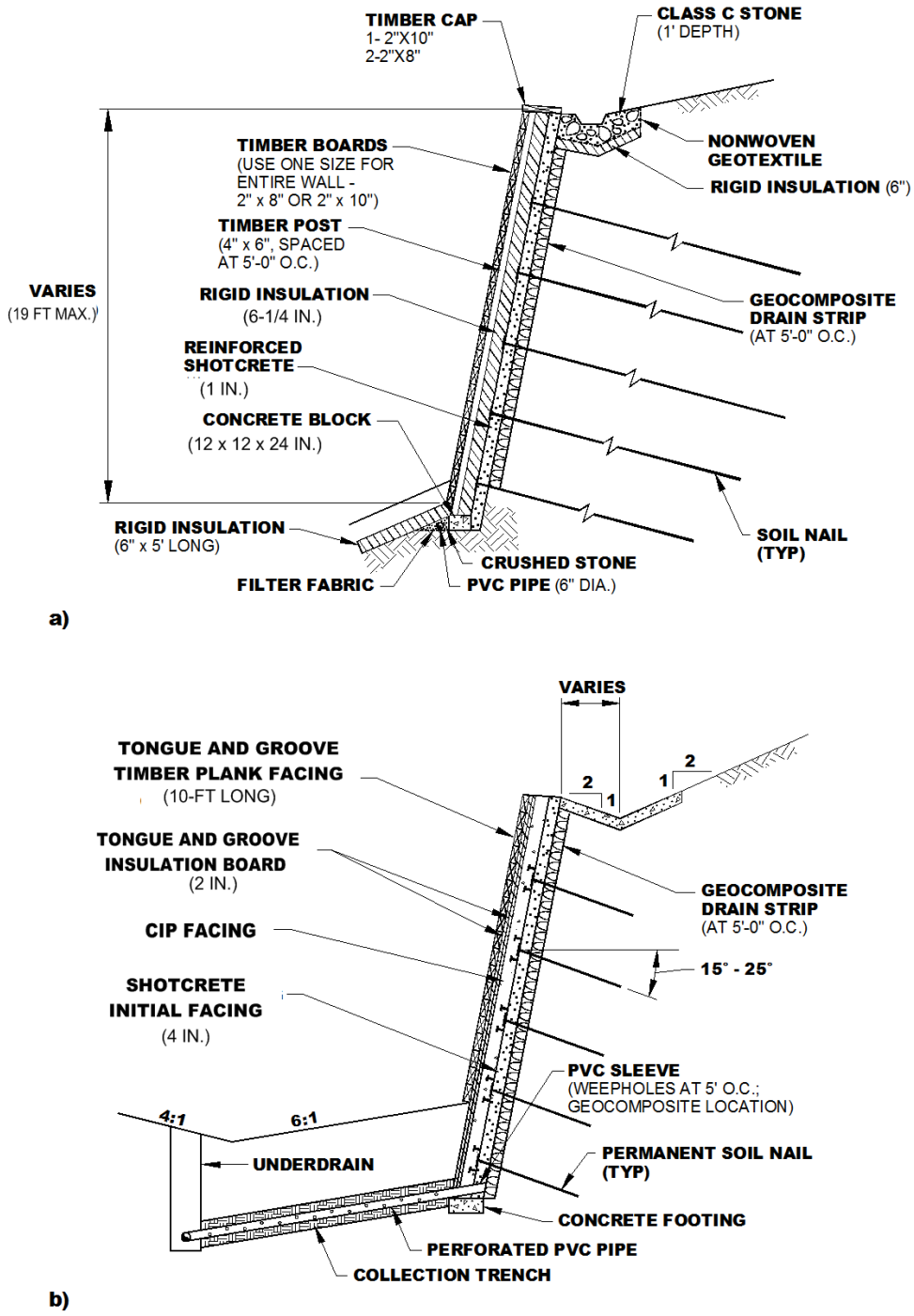


Figure 6.17: Illustration. Examples of frost protection: (a) first example, and (b) second example. Modified after Byrne et al. (1998).

6.11 Step 11 Other Design Considerations

Final evaluations of constructability may need to be carried out in this step if the design has requirements for special construction needs, such as temporary support of exposed lifts, or use of strut-nails.

The Owner's design engineer will always have to develop a load testing program to verify the assumed design values. Recommendations for load testing programs are presented in Chapter 9.

The Owner's design engineer may also need to prepare a geotechnical monitoring plan to specify how deformations of the wall will need to be evaluated during and after construction. In general, a geotechnical monitoring plan would only be needed if the wall deformations were found to be non-significant during design, or if the presence of a deformation-sensitive structure near the wall was considered a risk to the wall project.

6.12 Step 12 Pre-Construction, Construction, and Post-Construction

6.12.1 Pre-Construction

In Design/Bid/Build contracts, the oversight of the preparation of construction drawings and specifications by the same personnel performing the design is highly desirable to ensure that the design intent is unambiguously conveyed in construction documents. Drawing and specification preparation should be an integral part of the Design phase. The Pre-Construction phase includes activities such as:

- Preparation of Owner's engineering cost estimates
- Evaluation of bidders' experience
- Selection of contractor

Chapter 8 "Contracting Approaches and Specifications" provides guidelines for the activities listed above during the Pre-Construction phase.

6.12.2 Construction and Post-Construction

The Construction and Post-Construction phases include the following activities:

- Evaluation of Contractor's submittals
- Observation and evaluation of load testing (as part of construction quality assurance, QA)
- Observation and evaluation of construction methods
- Observation and evaluation of the installation of production nails

- Observation of drainage and facing construction
- Geotechnical monitoring during construction and/or in the long-term
- Preparation of post-construction engineering reports

Chapter 9 “Construction Inspection and Performance Monitoring” presents guidelines for the main technical activities during and after construction.

Chapter 7: Corrosion Protection

7.1 Introduction

This chapter discusses the main factors that promote or inhibit corrosion in soil nails, presents criteria for assessing soil corrosion potential, and introduces guidelines for selecting levels of corrosion protection. The design and construction of permanent soil nail walls require a thorough consideration of the corrosion potential of tendons and other metallic components of these systems. The tendons discussed in this manual are grouted; therefore, some level of corrosion protection always exists in these systems. However, as it will be discussed, grout alone cannot provide adequate corrosion protection for aggressive ground conditions.

This chapter focuses on corrosion protection for solid bar soil nails. HBSNs are discussed in Chapter 10. The options for corrosion protection of HBSNs are limited as compared to solid bar soil nails due to the methods used to install HBSNs.

7.2 Historical Background

There are no available publications in the literature reporting soil nail wall failures or serviceability issues due to corrosion of the tendons. Therefore, the recommendations presented in this manual do not follow from specific experience in soil nails, but rather experience from other earth retention systems. Publications focusing on corrosion of ground anchors and metallic reinforcement of MSE walls provide relevant information for soil nail protection.

Recommendations and details about corrosion protection for ground anchors are presented in "Tiebacks" (Weatherby 1982), Post-Tensioning Institute "Recommendations for Prestressed Rock and Soil Anchors" (PTI 2004), "Ground Anchors" (Sabatini et al. 1999), "LRFD Bridge Design Specifications" (AASHTO 2014), and "Standard Specifications for Highway Bridges" (AASHTO 2002).

Weatherby (1982) presents a detailed discussion of the effects of corrosion on pre-stressed steel anchors and includes a summary of the state-of-practice for corrosion protection mainly derived from Europe at the time. PTI (2004) includes a decision tree to help users select levels of corrosion protection for ground anchors based on various factors, including soil "aggressiveness" (equivalent to corrosion potential) and potential risk assessment including cost considerations. Sabatini et al. (1999) expands on these concepts and includes a decision tree for selecting corrosion protection levels similar to that of PTI (1996, 2004). AASHTO

(2014) and AASHTO (2002), Section 6, "Ground Anchors" also provide details and recommendations for corrosion protection of ground anchors.

While these publications are specific to prestressed ground anchors, certain aspects of their corrosion protection can be extended to soil nails. For example, the concept of corrosion protection classes can be applied to soil nails. This includes criteria for differentiating between strong and mild soil corrosion potential, and guidelines for selecting increasing levels of corrosion protection based on a decision tree similar to those in PTI (2004) and Sabatini et al. (1999).

7.3 Terminology

Over the years, different terms have been used in relation to corrosion protection of reinforced soil structures. For clarification, the following terms will be considered in this document:

- **Aggressiveness**: The tendency of the soil to promote corrosion of steel at relatively fast rates.
- **Aggressive Soil**: A soil with values of resistivity, pH, sulfate, and chloride contents that exceed established thresholds (Elias et al. 2009) and that promote corrosion of steel at relatively fast rates.
- **Non-Aggressive Soil:** A soil with values of resistivity, pH, sulfate, and chloride contents that do not exceed established thresholds (Elias et al. 2009).
- **Corrosion Potential:** Similar to aggressiveness.
- **Class A Protection:** A level of corrosion protection that involves encapsulation of the soil nail in plastic sheathing.
- **Class B Protection:** A level of corrosion protection that includes epoxy coating or galvanizing the tendon and connection components.
- **Class C Protection:** A level of corrosion protection that consists of a bare tendon with no protective coatings other than the surrounding grout.
- **Passivation:** Refers to the process by which a metal corrodes and forms an outer layer of corrosion, causing it to become less reactive and more “passive.”

The process for selection of corrosion protection type, and the characteristics of Classes A, B, and C in this version of the manual are different than those in previous versions.

7.4 Corrosion Mechanisms

7.4.1 Overview

Corrosion is the process that deteriorates a refined metal by changing its physical and chemical properties. This process may ultimately dissolve the metal due to electrochemical reactions with the surrounding environment. The electrochemical reactions involved in corrosion consist of the movement of ions from a metal surface (anode) to another metal surface (cathode), through a nonmetallic conductor existing in solution (electrolyte) such as dissolved oxygen, chlorine, and hydrogen ions. Electrochemical reactions may arise due to a difference in voltage, oxygen concentration, pH, or electric resistivity between the cathode and the anode. Over time, the anode may be depleted as a result of the metal loss into the electrolyte.

For grouted soil nails, ions can flow between two points along the tendon, as shown schematically in Figure 7.1, or between a nearby metallic object and a nail (such as the plate to the tip of the tendon).

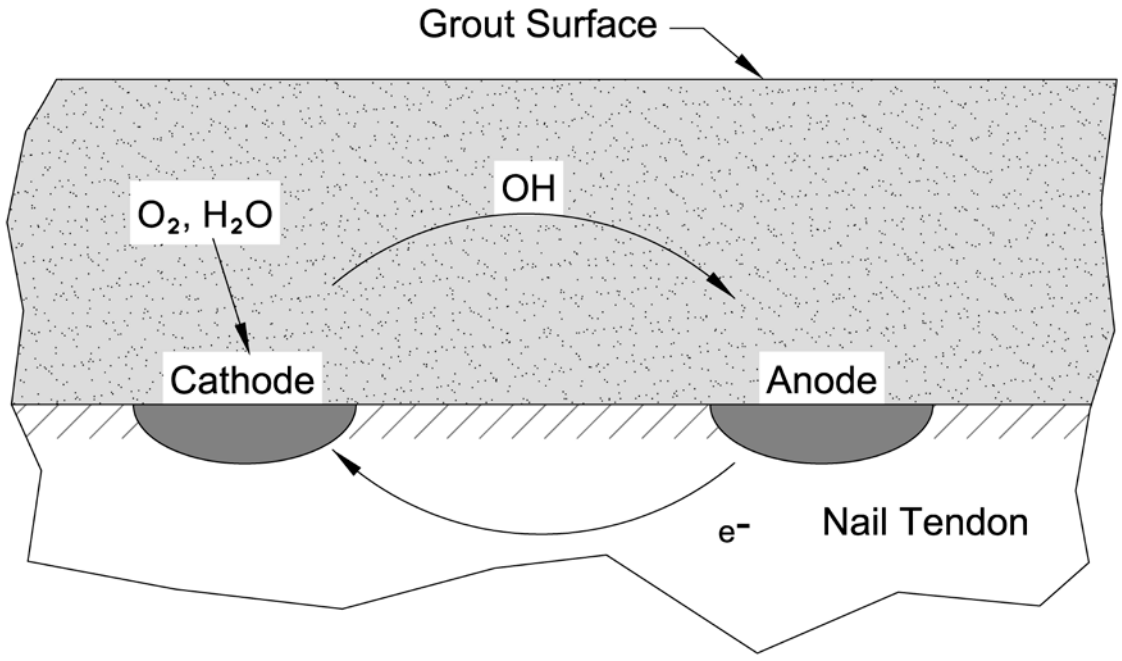


Figure 7.1: Illustration. General corrosion of a tendon. After Hamilton et al. (1995) and Samtani and Nowatzki (2010).

7.4.2 Types of Corrosion

The following types of corrosion can take place in reinforced soil systems (Withiam et al. 1992; Sabatini et al. 1999):

- General corrosion
- Localized corrosion
- Stray-current corrosion
- Stress crack corrosion/hydrogen embrittlement
- Fatigue corrosion
- Bacterial attack
- Galvanic corrosion

The first three types, which are the most significant for soil nails, will be discussed in the following sections. Stress crack corrosion and hydrogen embrittlement corrosion tend to occur typically in prestressing steel bars and strands used as ground anchors, where tensile stresses are much higher than in soil nails (Withiam et al. 1992). Fatigue corrosion is uncommon in soil nails because this mechanism arises in combination with pronounced cyclic loading: a very unlikely event on soil nail projects. Corrosion due to bacterial attack takes place when bacteria existing in the ground reduce sulfate under anaerobic conditions promoting conditions for corrosion. Bacteria living in organic soils or sulfate-bearing clays below the groundwater may promote this attack. Galvanic corrosion is a process when two metals having different electrode potentials are in contact and immersed in an electrolyte, and one of the metals tends to corrode onto the other. This process is not seen in soil nails installed with appropriate materials, but could theoretically occur if, for example, the steel tendon is in contact with another metal with higher anodic index. The next sections discuss general and localized corrosion, as well as stray currents.

7.4.2a General Corrosion

General corrosion takes place on the surface of steel tendons in the shape of a uniformly-distributed, thin layer of rust. This corrosion type may develop when steel is left unprotected on site, is handled improperly during shipping, or is stored inadequately. General corrosion causes an approximately uniform loss of metal of the tendon. Once this very small passive film is formed, the oxidation rate decreases to a small value, effectively stopping further corrosion. This rust layer is thus often favorable because it becomes a protective film. If the conditions that inhibit corrosion are absent, general corrosion may be significant over the typical design of at least 75 years for permanent highway structures.

Bare, non-coated tendons should be inspected for excessive surface corrosion. Wax, oil, or other temporary coatings should not be used in an attempt to protect tendons during stockpiling because these materials can deteriorate the grout-to-nail bond and lead to other problems.

7.4.2b Localized Corrosion

Localized corrosion, also called pitting, develops in small pits or crevices, which promote non-homogeneous concentrations of ions, an increase in electrochemical potential, and further localized corrosion. When the grout covering a tendon is uncracked, it furnishes corrosion protection by: (i) providing a physical barrier to a potentially corrosive substance; and (ii) chemical passivation. As a result of the hydration of the Portland cement and mix water, the sodium, potassium, and calcium hydroxides that are formed provide an environment favoring passivation. This environment has an elevated pH (>13) and contains oxygen, causing the formation of a protective oxide layer over the surface of the tendon.

This protection can be lost when deep cracks form in the grout. Where cracks propagate entirely through the grout and extend into the steel tendon, pitting tends to occur. The extent and depth of pitting will depend on the availability of oxygen to continue the reaction; thus, in a fully submerged environment, the mere existence of cracks may not produce continued pitting. Cracks form in the grout because of tensile stresses caused by loading, or when shrinkage of the grout exceeds the tensile capacity of the grout. Cracks then become pathways that can convey corrosive substances to the tendons, if chlorides and/or sulfates exist abundantly in the ground, and if the crack width is sufficiently large to permit their passage. There is disagreement in the literature about the minimum crack width below which the grout can be considered relatively impermeable; however, a crack width of less than 0.004 in. is generally the lower bound value cited in the literature (Samtani and Nowatzki 2010). Once a pit is created, if sufficient oxygen is available, localized corrosion tends to propagate deeper, the cross-section of the tendon is reduced at that location, and overstressing or eventual failure may occur. Significant surface rust requires further inspection for evidence of pitting, which would be cause for tendon rejection.

If grouting is poorly done, portions of the tendons may be left uncovered, thus promoting localized corrosion.

7.4.2c Stray Current

As mentioned in Chapter 4, stray currents can contribute to the corrosion potential of soils. When there is prolonged exposure to stray currents, they tend to exacerbate the flow of ions between metallic parts and hence accelerate corrosion. Stray currents tend to cause corrosion where the ion flow is strongest, at locations where current leaves the structure and enters the ground or water electrolyte (Withiam et al. 1992).

Soil nail walls using high strength steel (95 to 150 ksi) may be affected by stray currents unless other suitable protective measures are present. Examination of projects where stray

currents existed indicates that the potential for stray current corrosion is very small when the sources are located more than about 100 to 200 ft from the tendons (Elias 2000).

Besides the distance to the stray current source, numerous other factors may affect the rate of corrosion of metals subject to stray currents, including the source voltage, characteristics of the metallic surface (e.g., steel, zinc), the type of electrolyte (e.g., water, water contaminated with other fluids), the presence and concentration of substances in the electrolyte (e.g., salts such as carbonates, chlorides), the nature of the anode/electrolyte interface, and other environmental factors. Presently, there is no published information that provides guidance as to the susceptibility of soil nails to stray currents, and the effect of parameters such as line voltage and soil resistivity on the potential for corrosion of soil nails. Therefore, a corrosion expert must evaluate specific projects where stray currents are suspected to exist.

7.5 Evaluation of Soil Corrosion Potential

In order to select an adequate level of corrosion protection, the soil corrosion potential of the ground around a soil nail system needs to be evaluated. This evaluation must focus on the conditions promoting corrosion, as described below.

7.5.1 Conditions Promoting Corrosion

The conditions promoting corrosion of soil nails include:

- Too low or too high pH of the ground
- Low electric resistivity of the ground
- High concentration of chlorides or sulfides in ground or groundwater
- Saturated or near-saturated soil
- Soil with organic content
- Presence of stray currents
- Availability of oxygen

These factors collectively define ground corrosion potential (or aggressiveness).

The relative level of corrosion potential as affected by soil resistivity has been described (NCHRP 1978) as summarized in Table 7.1.

Table 7.1: Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)

Corrosion Potential/Aggressiveness	Resistivity (ohm-cm)
Very corrosive	< 700
Corrosive	700 - 2,000
Moderately corrosive	2,000 - 5,000
Mildly corrosive	5,000 - 10,000
Non-corrosive	> 10,000

7.5.2 Criteria for Assessing Ground Corrosion Potential

The criteria for assessing ground corrosion potential, which is based on Elias et al. (2009), is presented in Table 7.2. The ground is classified as aggressive if any one of the conditions in Table 7.2 exceeds the listed limits in the third column of the table. According to Elias et al. (2009), the chloride and sulfate requirements may be waved when the resistivity is greater than or equal to 5,000 ohm-cm.

Soils meeting the thresholds shown in the table are considered non-aggressive. Note that the current practice simplifies the use of ranges of resistivity and focuses on a single threshold separating "aggressive" and "non-aggressive" soils.

Table 7.2: Criteria for Assessing Ground Corrosion Potential

Test	Units	Threshold for Non-Aggressive	Test Method
pH	–	5.0 < pH < 10	AASHTO T-289
Resistivity	ohm-cm	Greater than 3,000	AASHTO T-288 (2)
Sulfates	ppm(1)	Less than 200	ASTM D4327
Chlorides	ppm	Less than 100	ASTM D4327
Organics	percentage by weight	Less than 1%	AASHTO T-267

Note: (1) ppm = parts per million.
(2) This method may have limitations for coarse soils that have very little, or no, material finer than the No. 10 sieve. See suggested alternatives in Elias et al. (2009).

Note that the term "non-aggressive" should not be implied as to never cause corrosion. Rather, it implies that the level of corrosion can be tolerated with reasonable confidence

within the design life of the structure (typically between 50 to 75 years), provided that the electro-chemical characteristics of the soils do not change over time.

Table 7.2 includes the recommended test for each property that may affect the corrosion potential. For some of the properties listed in Table 7.2, more than one type of test can be used, occasionally deviating from ASTM or AASHTO. The difference in results might be small in most cases, with some methods being more accurate and reproducible than others. A more detailed discussion about different test methods can be found in Elias et al. (2009). In all cases, the testing method used should be clearly stated.

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. Observations of buried structures lying immediately adjacent to the project may be good indicators of the corrosion potential of the ground.

Classification of the ground as aggressive or non-aggressive should consider future changes during the service life of the soil nail wall that can be reasonably anticipated and which will increase the aggressiveness of the ground (e.g., near road salt sources, mining operations, chemical plants, or chemical storage areas).

For permanent structures supporting roadways exposed to deicing salts, limited data from MSE wall experience indicate that higher corrosion rates may develop in the upper 8 ft of the retained soil as measured from the roadway surface, or to greater depths depending on the gradation of the soil. These higher corrosion rates are not presently well characterized and have not been directly measured for soil nail walls; nevertheless, under these conditions, a 30-mil (minimum) thick geomembrane should be placed below the road base and be tied into the roadway drainage system to avoid the penetration of deicing salts into the ground. The geomembrane should not be tied into the wall drainage system to avoid conveying salts to the wall.

7.6 Corrosion Protection Systems

7.6.1 Introduction

Corrosion protection can be provided by physical or chemical means, or a combination thereof. Electrical means, such as cathodic protection, are not typically used in soil nail projects. Physical protection involves placing a continuous barrier between the tendon, other metallic parts, and the corrosion sources.

Chemical protection consists of the use of a sacrificial material (e.g., galvanized material) or a dielectric material (e.g., epoxy coated bars or electrical isolation pads), which will impede the flow of electric current. Some corrosion protection systems utilize a combination of these mechanisms as discussed below.

7.6.2 Grout Cover

This method of corrosion protection involves fully covering the tendon with neat cement grout. Because grout is always present in soil nails as defined in this manual, this corrosion protection is always provided regardless of whether the ground is aggressive or non-aggressive.

Centralizers are used to ensure that the thickness of the grout cover is uniform along the length of the tendon. For solid tendons, the centralizers should be dimensioned and spaced to achieve a minimum grout cover of 1 in. between the tendon (regardless if this is bare, epoxy-coated, or galvanized) and the soil considering the sag of the bar between centralizers.

Grout cover provides both physical and chemical corrosion protection. When an effective grout cover is in place, the passivation provided by the chemical protection of the grout prevents or delays carbonates and chlorides in the soil, and oxygen and humidity in the air, from reaching the tendon. Grout meeting QC requirements will act as a barrier due to its low permeability and the alkaline environment created around the tendon, and will reduce the corrosion potential.

The protection provided by the grout cover cannot be relied upon in aggressive conditions because some cracking of the grout must be expected to occur. As discussed previously, the grout will crack when the tendon is subject to tensile loads, and cracking will tend to be more intensive at the location of the largest tension forces in the tendons, approximately where the critical slip surface intersects the soil nail. The crack width determines whether corrosion may actually occur or if the grout will remain relatively impermeable; however, there is lack of specific information in the literature about corrosion of soil nails. Therefore, it is a reasonably conservative practice not to rely on the grout cover when the soil nails are installed in aggressive ground, and to include additional methods of corrosion protection for design.

7.6.3 Epoxy Coating

Corrosion protection with epoxy consists of coating the tendon with a fusion-bonded epoxy in accordance with ASTM A775 or A934. This coating is applied by the manufacturer prior to shipment to the construction site. Epoxy coating is dielectric and thus provides physical

and chemical protection. A minimum grout cover of 1 in. between the tendon and the soil is placed around all epoxy-coated tendons.

The coatings used on steel rebar are color-coded in green, gray or purple. The green coating is the most commonly used and meets ASTM A775/A775M. The gray or purple coatings usually meet ASTM A934/A934M. The green epoxy coating is flexible and is suitable for rebar that will be bent in different shapes. The gray and purple epoxy coatings are less flexible than the green epoxy coating, but they have greater chemical resistance. The purple epoxy coating is better suited for marine or harsh environments. The long-term resilience of the various colors of epoxy coating has not yet been studied for comparison for soil nail walls; therefore, with respect to longevity, no type of epoxy coating is favored over another in this manual.

Epoxy coatings can be provided for aggressive or non-aggressive conditions. Without development of site-specific data demonstrating effectiveness in aggressive conditions, protection must not rely solely on epoxy coatings. While undamaged epoxy coatings can provide protection for significantly long periods because of their dielectric nature, epoxy coatings may be damaged during transporting and handling. Although ASTM standards allow for some discontinuities in epoxy coatings (referred to as holidays in corrosion protection), damage to the epoxy coating should be repaired when practically possible. Occasionally, field-repairs are made to cover small chips or nicks in the epoxy coating applied over solid bars using kits provided by bar manufacturers. However, some of the damage may not be detected.

The required thickness of the epoxy coating must be 7 to 12 mils as specified in ASTM A775 or A934 for the tendon sizes typically used as soil nails. However, a reasonable range of epoxy coating thickness should be specified, as opposed to one single value. Establishing a range of thickness for epoxy coating of tendons in a project specification, different from that specified by ASTM, must be done with care and with understanding of the consequences of having excess coating. The process of epoxy application has an inherent thickness tolerance that may vary from manufacturer to manufacturer. Epoxy coaters attempt to apply a larger thickness than specified so that the minimum thickness specification is met everywhere along the tendon. This will necessarily result in the existence of zones where the tendon will have excess thickness of epoxy. If the minimum thickness specification is too large, it may create a significant problem during installation as couplers and nuts may not fit on tendons with excess epoxy thickness. This may result in field personnel resorting to filing the excess epoxy, thus compromising the integrity of the coating, especially at the nail head. The threads of the couplers and nuts cannot be excessively oversized to accommodate the excess epoxy thickness without losing performance.

As mentioned, regardless of the type of epoxy encapsulation that is used, the minimum grout cover over the tendon must be 1 in.

7.6.4 Zinc Coating (Galvanization)

A 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 tendons and other metallic pieces in a zinc bath. The protection provided by galvanized coating is both physical and chemical, as this process forms a protective layer of sacrificial zinc oxide. Cement grout is placed around all galvanized tendons.

Bars should be coated according to AASHTO (2014) or ASTM A123. ASTM A767/A767M can also be used as a reference. A minimum galvanization coating of 2.0 oz/ft^2 or 3.4-mil thickness is required per Article 11.10.6.4.2a (AASHTO 2014). ASTM A123 also requires a 3.4-mil minimum average for all tendons on a project, and a 3.1-mil minimum for an individual tendon. Galvanization must be applied in accordance with ASTM A153 for nuts, plates, and other hardware. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of corrosion that would occur in non-galvanized or bare tendons. After the zinc is consumed by oxidation, corrosion of the base metal starts.

Galvanization can also be damaged during handling and construction by abrasion, scratching, notching, and cracking. Care must be taken during handling and construction to avoid damage. Construction equipment should not travel directly on reinforcing elements and elements should not be dragged, excessively bent, or field cut. Galvanized reinforcement should be well supported during lifting and handling to prevent excessive bending. Any damaged section should be field repaired by coating the damaged area with a field grade, zinc-rich paint.

Again, regardless if galvanization is used over the tendons, the minimum grout cover over the tendons must remain 1 in.

7.6.5 Encapsulation

Encapsulation is common practice in permanent soil nail walls constructed in an aggressive or unknown environment. It consists of fitting the tendon with a corrugated sheathing of PVC with a nominal 0.04-in. thickness, or of HDPE with a nominal 0.06-in. thickness. The annular space between the tendon and the sheathing is grouted in the shop and the whole assembly transported to the project site. Cement grout is placed around all encapsulated tendons.

The sheathing is corrugated to promote transfer of the nail load to the surrounding grout. It must be sufficiently strong to resist transportation, handling, and installation; and must be non-reactive with concrete, chemically stable, ultra-violet-light resistant, and impermeable. Certain sheathing techniques may be proprietary.

The grout inside the sheathing must completely fill the annular space around the tendon and provide a minimum internal cover of 0.2 in. over the tendon. 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 drill hole wall must be 1 in. Coupling of encapsulated tendons, if required, should consist of heat shrink over wax tape and should be done in accordance with PTI recommendations (2007) to avoid development of a discontinuity in the permanent soil nail.

The portion of a soil nail tendon immediately behind the facing tends to be more susceptible to corrosion at similar ground conditions. Sheathings only 3-ft long extending behind the facing have been employed in some projects (see Figure 3.9).

7.6.6 Sacrificial Steel

Sacrificial steel is used in metallic reinforcement, such as that used in MSE walls. Fishman and Withiam (2011) identified that non-prestressed elements are often designed by including sacrificial steel to account for metal loss due to corrosion, and that prestressed elements, such as ground anchors, do not include sacrificial steel. Their approach is specifically applicable to MSE wall reinforcements, where the quality of the fill can be controlled when compared to soil nails.

Sacrificial steel may be used as a means to provide corrosion protection for solid bar soil nails depending on the stipulations of the Owner and any jurisdictional agency. Because coatings and encapsulation are inexpensive, more effective, and appear to offer less uncertainty, they are preferred over the use of sacrificial steel in solid bar soil nails.

Sacrificial steel is computed using the following equation for plain steel elements (Fishman and Withiam 2011):

$$X(\mu m) = 80\,(\mu m/yr) \times t_f^{0.8}$$

Equation 7.1: Thickness of sacrificial steel.

Where X is loss of steel in μm, and t_f is the service life in years. Steel loss must be considered as taken twice from the diameter of the bar:

$$d_{loss} = d_{bar} - 2X$$

Equation 7.2: Remaining bar diameter.

Where d_{loss} is the bar diameter after sacrificial steel is considered and d_{bar} is the nominal bar diameter.

Models incorporating galvanization over sacrificial steel can be used.

7.6.7 Nail-Head Corrosion Protection

The bearing plates, headed studs, washers, and nuts constitute the nail head. While the nail head is not in direct contact with the ground, it is connected to the soil nail and, as a result, it requires corrosion protection. The minimum concrete or shotcrete cover is 2 in. on the outside face and 3 in. on the soil side. All components of the nail head should be epoxy coated or galvanized when the tendon is encapsulated, epoxy coated, or galvanized. However, if the total cover (initial and final facing) over the head components is 9 in. or more, the nail head components may not need to be encapsulated, epoxy coated, or galvanized if the wall is constructed in a non-marine environment. Epoxy coating of the nail head can be applied when a galvanized tendon is used and vice-versa.

The thickness of the final facing should be increased from what is structurally necessary so as to meet the minimum requirements for concrete or shotcrete cover over the headed studs and other nail head elements. Alternatively, the thickness of the final facing can be kept as structurally needed, and the layout and/or features of the headed studs can be modified so that the headed studs are shortened but the original facing resistance is maintained.

7.6.8 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 tendon. The isolation can occur along the path or at the nail. Encapsulation can be used for corrosion protection to provide isolation. Effectiveness of electrical isolation must be field-tested and verified by personnel qualified for this task after nail installation and before grouting.

7.7 Methodology for Selecting Corrosion Protection Levels

7.7.1 Introduction

This section presents a method for selecting the corrosion protection level suited for the project, soil, and soil nail tendon types. There are many factors to consider in the selection of a corrosion protection system. Some of the more significant factors include the perceived

consequences of failure, the relative cost of different levels of protection, varying performance of protection methods based on tendon type, and the effect of tendon installation on some protection systems. A decision tree is presented to appropriately account for the above listed factors and others, along with a step-by-step procedure.

7.7.2 Selection Procedure

7.7.2a Overview

The corrosion protection selection methodology presented herein for solid soil nail bars (Figure 7.2) is an adaptation of the decision tree presented in PTI (2004). The methodology deviates from previous FHWA guidance for soil nail walls to better emphasize the different levels of protection between encapsulated and epoxy coated or galvanized bars.

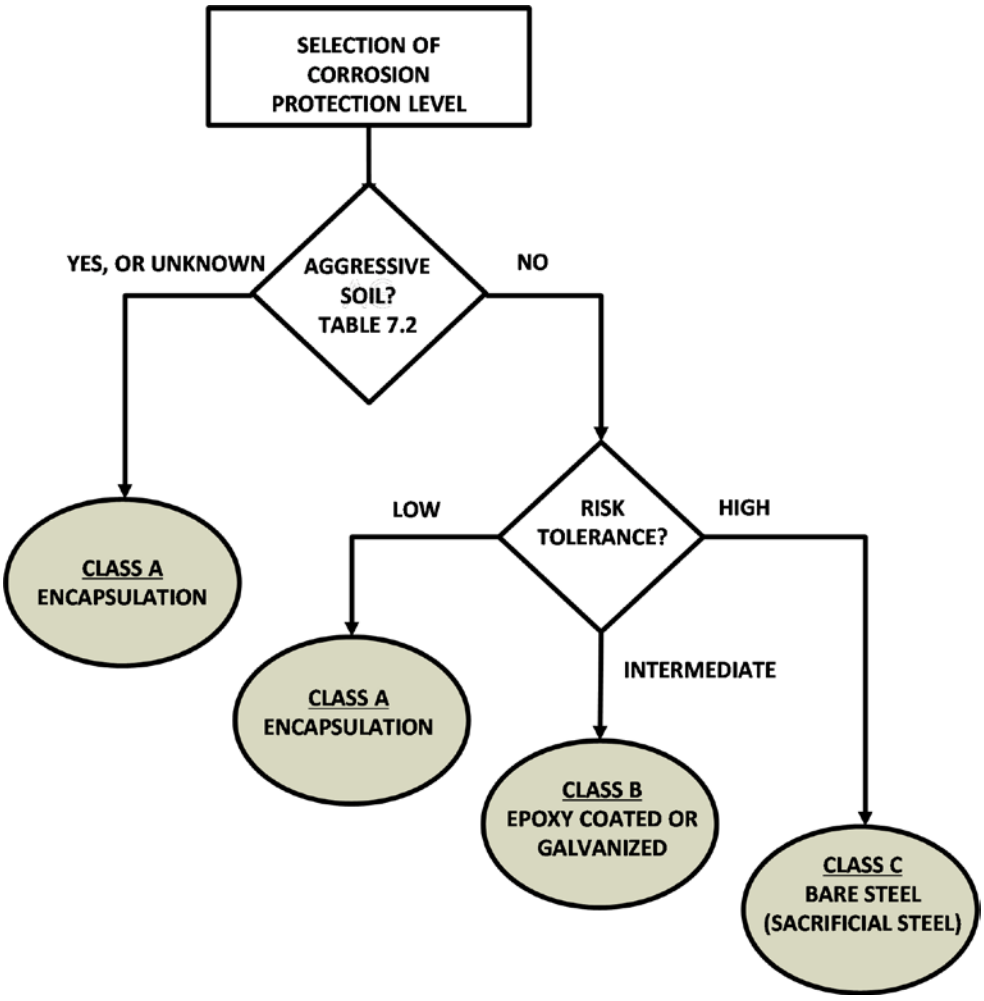


Figure 7.2: Illustration. Procedure for selecting soil nail corrosion protection level.

The corrosion classes shown in the figure represent the minimum level to be provided in permanent soil nails. Owners may elect to adopt a higher corrosion class level for conservatism, depending on their tolerance for risk (see next section).

One change with respect to PTI (2004) consists of omitting the consideration of the service life because this document only addresses permanent elements. The first item in the tree is to denote the soil as aggressive or non-aggressive according to the criteria presented in Table 7.2. Aggressive soils or those with unknown corrosive conditions should receive Class A protection as defined in Table 7.3. Projects planned in non-aggressive soils should then be evaluated further to consider the risk tolerance. Risk tolerance is a term meant to convey the assessment of the remaining factors including the perceived risks and failure consequences, and added costs to provide a higher level of protection. In general, the subdivisions of low, intermediate, or high risk tolerance are based on a balanced selection to compensate risks and costs, but also on the principle that in case of uncertainty, a conservative level of corrosion protection is provided.

Table 7.3: Corrosion Protection Levels for Soil Nails

Class Protection	Protection Methods Used(1)	Conditions/Remarks
A	Encapsulation	• Aggressive soil, or unknown corrosion potential • Non-aggressive soil conditions with low risk tolerance • This is the highest level used in practice; however, in extreme situations, encapsulation can be combined with epoxy coating or galvanization
B	Epoxy Coating or Galvanization	• Non-aggressive soil conditions with intermediate or high risk tolerance
C	Bare Steel Tendon (Sacrificial Steel)	• Non-aggressive soil conditions with high risk tolerance

(1) All soil nail bars are assumed to be grouted and include the grout protection in each class protection level listed herein.

7.7.2b Risk Tolerance

Risk tolerance must be assessed by the Owner for each individual project considering a multitude of factors. These factors are too many to list and to provide specific guidelines for

their evaluation. Therefore, two simple, hypothetical examples are described to illustrate the evaluation of risk tolerance by the Owner.

In a hypothetical case where the soil is determined to be non-aggressive based on comprehensive testing, failure of the soil nail wall due to the unlikely corrosion of the soil nails would happen only after a significant time after construction. If such failure would produce only limited material losses, that the Owner could tolerate and afford to repair, then the Owner may opt to assess the risk tolerance as "high." The Owner may still opt to request a Class B or Class A protection if the Owner deems that the additional cost is not significant.

Conversely, if testing indicates that the soils are non-aggressive but, in the judgment of the Owner, a failure would produce costly material losses that the Owner may or may not be prepared to sustain, the Owner may opt to assess the risk tolerance as "intermediate" or "low."

In a case where there is only limited testing that indicates that the soils are non-aggressive, no evidence of corrosion of nearby structures, but where failure may produce injury or loss of life, the Owner should assess the risk tolerance as "low."

Ultimately, the Owner must decide the level of risk tolerance in the specific project based on the particular circumstances that surround the project, the level of confidence in the investigation, the aptitude of the Owner to inspect and maintain existing structures, and the level of funding available, among others.

7.7.2c Corrosion Protection Levels

Three levels of corrosion protection are presented in this document:

- Class A Corrosion Protection: consists of encapsulation (Figure 7.3)
- Class B Corrosion Protection: consists of epoxy coating or galvanizing (Figure 7.4)
- Class C Corrosion Protection: consists of a bare steel tendon with no protective coatings other than grout cover (Figure 7.5)

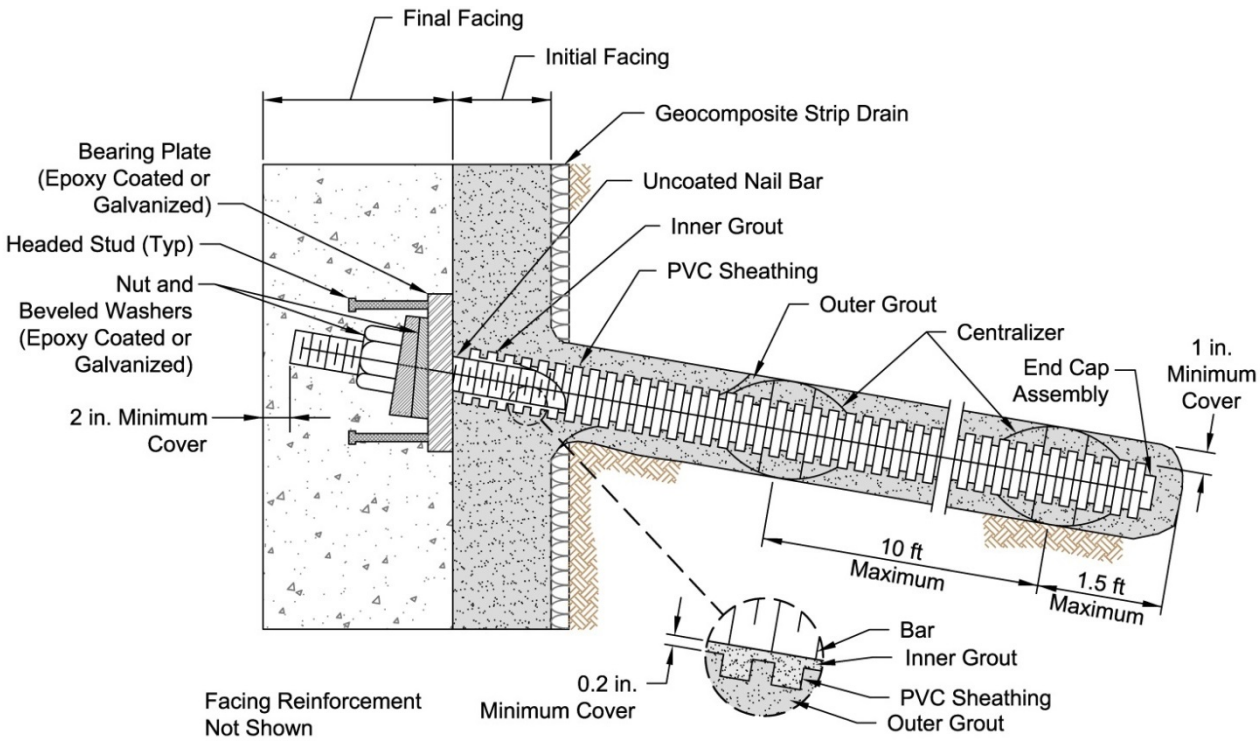


Figure 7.3: Illustration. Class A corrosion protection: encapsulation. Modified from Byrne et al. (1998).

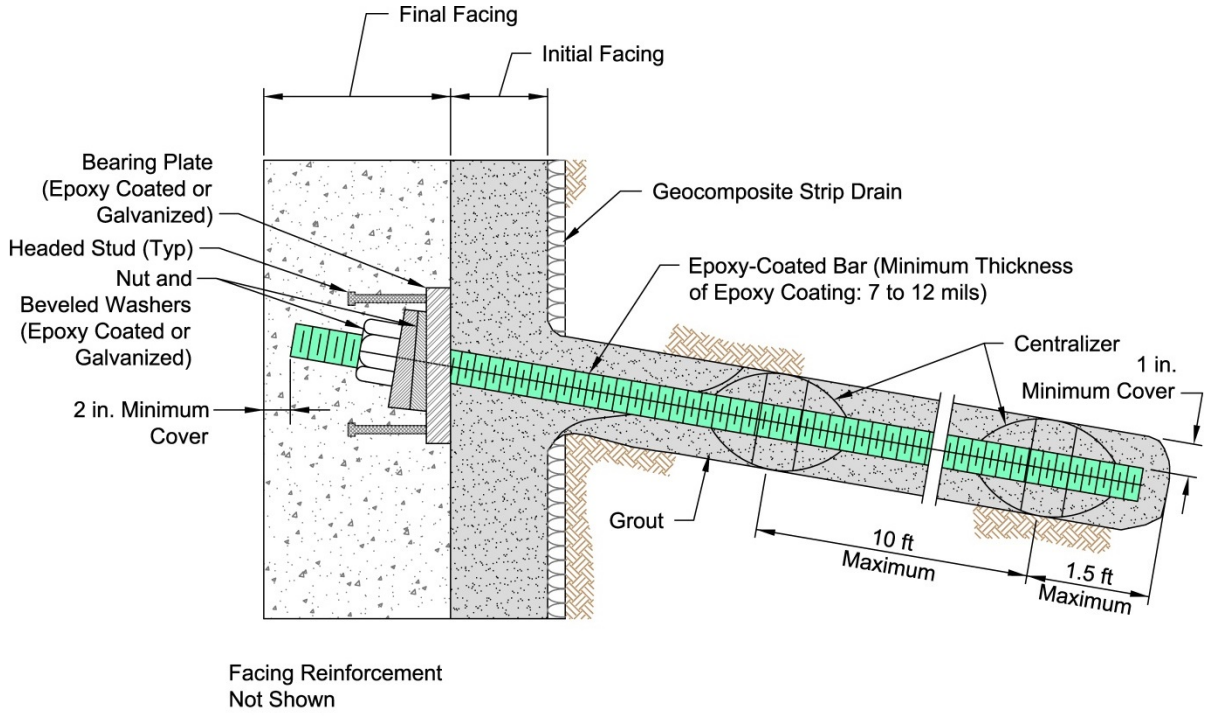


Figure 7.4: Illustration. Class B corrosion protection: galvanizing or epoxy-coating. Modified from Byrne et al. (1998).

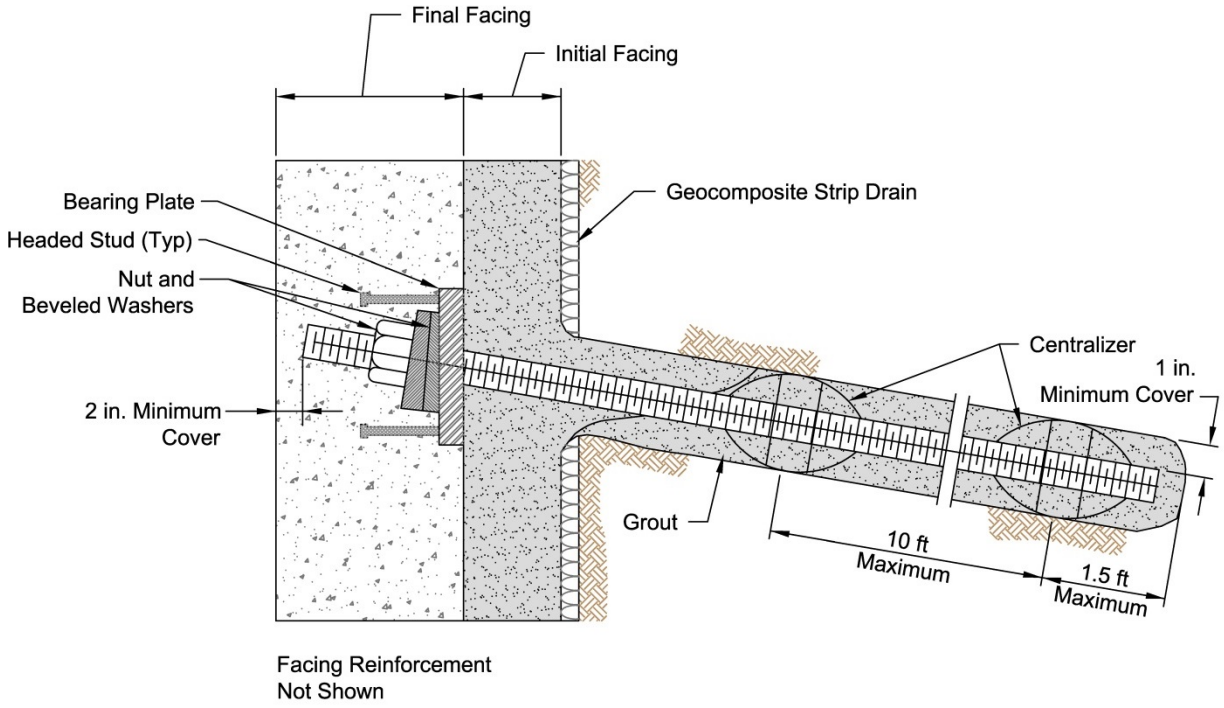


Figure 7.5: Illustration. Class C corrosion protection: bare steel tendon. Modified from Byrne et al. (1998).

7.8 Design of Corrosion Protection

7.8.1 Design Steps for Corrosion Protection of Soil Nails

The selection of soil nail corrosion protection is performed by the Owner considering the following steps.

(1) Use Table 7.2 to classify soil as non-aggressive or aggressive, based on results of corrosion testing as described in Chapter 4 and in this chapter.

(2) Use Figure 7.2 to determine the appropriate corrosion protection level with regard to aggressiveness, risk tolerance, and other miscellaneous considerations detailed in this chapter.

(3) In conjunction with determining the corrosion protection level, select the appropriate tendon type to meet the project needs.

(4) If Class C is selected:
   a. Obtain the nominal tendon diameter
   b. Select the design life of the soil nail wall (typically 50 to 75 years)
   c. Calculate sacrificial steel using Eq. 7-1

d. Calculate the new diameter with steel loss
e. Use the new diameter to calculate the area of the tendon with steel loss
f. The area with steel loss should then be used as the cross-sectional area of the tendon, A_t, in Eq. 6-5 to calculate the nominal resistance of the tendon
g. Proceed with the design according to Chapter 6

(5) If Class A or B is selected, complete the design according to Chapter 6.

7.9 Unusual Conditions

Structures exposed to a marine or other chloride-rich environment represent unusual conditions that require further consideration. For marine structures exposed to saltwater, carbon steel losses on the order of 3.2 mils per side or radius should be anticipated in the first few years, reducing to a range from 0.67 to 0.7 mils thereafter. Zinc losses are likely to be quite rapid as compared to losses in retained soils meeting the minimum electrochemical criteria. Total loss of zinc of approximately 3.4 mils should be anticipated in the first year of exposure.

As mentioned earlier, the potential of higher corrosion rates occurring under road and bridge abutments exposed to de-icing can be addressed by installing a 30-mil (minimum) thick geomembrane below the road base and tied into the roadway drainage system to avoid the penetration of de-icing salts into the ground.

Structures exposed to acidic water emanating from mine waste, abandoned coal mines, or pyrite-rich soil and rock strata are also special conditions that may require the input of a corrosion specialist.

Conditions such as these may not be suitable for soil nailing applications. In these unusual conditions, the corrosion potential of the soil will likely be the strongest factor for consideration during the feasibility study.

Chapter 8: Contracting Approaches and Specifications

8.1 Introduction

Two types of contracting methods can be used to develop contract drawings and specifications for permanent soil nail wall systems. Some Owners prefer one approach over the other, or a combination of the two approaches. The contracting method is selected based on: the criticality and complexity of the project, the experience of the Owner and their engineering consultants, and the availability of specialty contractors. Both contracting approaches are valid if properly implemented, and each has advantages and potential disadvantages. These contracting methods include:

- *Procedural or Method Approach.* All details of design, construction materials, and methods are specified in the contract documents. An example specification is presented in Appendix E.
- *Performance or End-Result Approach.* The contract documents specify the criteria for the successful outcome or result of the soil nail wall. A full design and detailing of the soil nail wall, and construction methods, are not prescribed. With this approach, a detailed design submittal occurs in conjunction with the submittal of construction drawings. An example specification is presented in Appendix F.

Because specialty contractors often introduce innovative, cost-competitive solutions, contract documents for soil nail wall projects should be structured to allow specialty contractors to make use of the latest available construction techniques. Therefore, whether using the procedural or performance approach, specifying the drilling and grouting methods or other Contractor's means and methods is not recommended. However, restrictions can be placed to avoid undesirable drilling and grouting methods in various subsurface conditions. In this section, contracting procedures and guideline information are summarized. The example specifications presented in Appendices E and F can be readily modified for a specific project and can be referenced, where applicable, to the Owner's standard specifications. To facilitate tailoring the guideline specifications to specific projects, comments have been added throughout Appendices E and F, and are shown in italics.

8.2 Geotechnical Analyses Required by Specifications

Whether the procedural or performance approach is followed, there are three essential analyses that must be conducted as part of the feasibility assessment of a soil nail wall: global stability, deformation, and soil corrosivity/corrosion protection class determination. These analyses may be performed as part of the geotechnical report and included in the contract documents for use by the Contractor, performed by the Owner, or be required submittals

from the Contractor. If required of the Contractor, ample time must be supplied by bid documents to permit the Contractor to conduct their own subsurface investigation and analyses besides those performed by the Owner for feasibility. The three analyses are discussed further below.

The Owner may perform their own global stability analyses and soil nail wall design before the preparation of bid documents. This approach is recommended for Owners who have sufficient geotechnical engineering expertise and local soil nail design experience, and where significant design modifications by the Contractor are not anticipated.

If the Owner performs their own global stability analyses but leaves the soil nail wall design to the Contractor, results from global stability analyses should be provided by the Owner as reference documents in bid packages. All potential issues identified during the global stability analysis performed by the Owner should be disclosed in bid documents. However, contract documents should require that the Contractor verify stability concerns during the final design of the soil nail wall.

Alternatively, the Owner may decide to charge the Contractor to perform global stability analyses in addition to conducting the soil nail wall design. In this case, the requirement by the Contractor to perform the stability analyses must be clearly indicated in specifications included in bid documents. In addition, the Owner should provide specific information for the global stability analysis or verification by the Contractor in the bid documents. This information should include soil shear strength parameters, pore pressure magnitude, etc. The objective of providing this information is to reduce the level of effort by the Contractor and reduce or eliminate the needs for the Contractor to perform their own exploration. The Owner will also need to plan in the procurement schedule sufficient time during the bid for the Contractor to complete an evaluation of the Owner-provided information and to conduct their global stability analyses.

For projects where the soil nail wall deformation is not a critical factor as affecting adjacent structures, the Owner may elect to perform preliminary deformation analyses during the planning stages of the project. These evaluations may be simple calculations or more complex numerical analyses. For projects where the wall deformation is a critical factor, the Owner should always conduct deformation evaluations, and should indicate in the performance specifications a minimum set of considerations that the Contractor's designer must include in their own deformation analysis. If the Owner requires specific numerical methods, the performance specifications should also include these methods.

The Owner should indicate in the specifications the minimum acceptable methods of monitoring to assess movements of the soil nail wall and the surrounding structures.

Before the preparation of bid documents, the Owner must perform corrosivity testing on representative soil samples, as described in Chapter 4, to determine the class of corrosion protection required for the soil nails. The corrosion class and design life are generally specified by the Owner. The Owner will then include the corrosion class required on design plans.

The Owner should always provide the results of corrosivity testing to the Contractor. The Contractor may also elect to provide a higher corrosion class than specified as a substitution. If the selection of a high corrosion class requires special handling and preparation of the various soil nail elements, the Owner should plan accordingly to allow the Contractor to secure these special elements.

8.3 Experience Requirements for Contractors

The practices used by the Contractor for soil nail wall construction need to provide a high quality product and avoid undesirable construction conditions that could jeopardize the structural integrity of the wall and represent a hazard to workers. Barring the potential for differing conditions, the likelihood of obtaining a satisfactory soil nail installation increases when an experienced and qualified soil nail Contractor with experienced personnel and good QC procedures is selected to construct the wall, regardless of the construction method being considered. The prospect of success further increases when comprehensive construction quality assurance procedures are developed and enforced by the Owner or the inspection personnel. Owners with no previous soil nailing experience should seek a competent third-party quality assurance provider.

Contract documents for soil nailing projects should clearly define: (i) the Contractor experience requirements, (ii) submittal procedures for this information, and (iii) the means by which the Owner will enforce these requirements. Such experience requirements are used by many transportation Owners for other specialty construction techniques, such as permanent ground anchor walls, micropiles, drilled shaft foundations, and shotcreting. However, for some public Owners, it is prohibited by law to "pre-qualify" contractors, and any "pre-qualifications" of experienced contractors must be disregarded. However, this does not preclude specification requirements for bidders to demonstrate that they meet specific soil nail experience criteria as discussed above.

Contractor experience requirements must extend to their specialized personnel (e.g., driller, nozzlemen, etc.). The requirements must include: (i) a minimum duration of experience with similar type of work, and/or (ii) a minimum number of projects where the Contractor has successfully constructed similar soil nail walls. Typical requirements for soil nail contractor experience are included in the guideline specifications presented in Appendices E and F.

8.4 Procedural Specifications

The procedural contracting approach includes the development of a detailed set of plans and specifications to be provided in the bidding documents. In this approach, complete design details and specifications are developed so that each contractor submitting a bid has a defined product to price, making it more straightforward for the Owner to compare pricing. This approach empowers the Owner's engineers to examine options that may be available during design, but requires engineering staff trained in soil nail wall design and construction methods. The Owner's design staff then also becomes a valuable asset during construction, when questions and/or design modifications are required.

Under this contracting procedure, the Owner 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, and drainage) in strict accordance with the contract documents. The Owner assumes all design risks and is responsible for design modifications and costs that may be required based upon actual field conditions. The Owner should provide existing geotechnical reports as reference documents.

The use of a procedural or method specification is recommended only for Owners who have developed sufficient in-house expertise and consider soil nail wall design and construction control as a conventional or standard method for earth retention.

8.5 Performance-Based Specifications

This approach is often called "line and grade," "conceptual plans," or "Design/Build." Under this approach, the Owner: (i) prepares drawings defining the geometric and aesthetic requirements for the structure, and material specifications for the components, (ii) defines performance requirements including LRFD resistance factors, corrosion protection levels, and deformation limits, and (iii) indicates the range of acceptable design and construction methods.

This approach offers several benefits, when coupled with sound specifications and experience requirements of contractors. Design of the structure should be performed by an experienced designer: in-house or retained by the Contractor. This design is then more suited to the Contractor's equipment and methods along with material components successfully and routinely used in the past. Also, the performance specification approach lessens preconstruction engineering costs and manpower requirements for the Owner, transferring some of the project design costs to construction. The disadvantage is that the Owner's engineers must have adequate expertise in soil nail technology to perform a design review and approve construction modifications, or must engage a consultant with demonstrated proficiency in this technology.

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 foot of wall face.

Special provisions may be included in performance-based specifications. These need to include the basis for special designs, a list of related submittals, as well as the schedules for these submittals and for review and approval of these submittals. Submittal items to be contained in special provisions may include: (i) calculations and design drawings to be conducted by Contractor, (ii) shop drawings and construction processes, and (iii) material information. In addition, special provisions may also include the basis for construction control and monitoring requirements.

Information furnished by the Owner as part of the contract documents should include the geometric, geotechnical, and design-specific information listed below:

- Plan and evaluate the areas to be retained, including beginning and end stations, top of wall, bottom of wall, and original ground line.
- Typical cross-section(s) indicating, as a minimum, original and final grade, face batter, pay limits, surface and subsurface drainage requirements, and excavation limits.
- Elevation view of each structure showing original ground line, minimum foundation level, finished grade at wall base, 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 structures, bridges, overhead signs and lights, screening walls, and traffic surcharges.
- Limits and requirements of drainage features beneath, behind, above, or through the structure.
- Tolerable deformations of wall during earthquakes, if applicable.
- Return period of design ground motions, if different than those published in AASHTO (2014), if applicable.
- Reference to specific governing sections of an Owner design manual (materials, structural, hydraulic, and geotechnical), construction specifications, special provisions, and performance criteria for the soil nail wall. The Owner may specify, as part of the special provisions, acceptable design methods by referencing one or more of the methods outlined in this manual.

- Results of all geotechnical investigations and laboratory testing on site. As an alternative, a baseline geotechnical report may be developed by the Owner and included as part of the contract documents.
- Maximum resistance factors and limitations on material properties for design.
- 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.
- Wall construction monitoring requirements.

If the Owner elects to provide the geotechnical information as not-to-be-relied upon, then the Owner shall give Contractor ample time to perform their own investigation to be used for design.

Performance specifications are recommended for Owners with limited experience with soil nail technology or for complicated projects where a specialty Contractor's specific or local knowledge can be maximized.

Chapter 9: Construction Inspection and Performance Monitoring

9.1 Introduction

Inspection activities, when appropriately conducted, play an integral role in the production of a high-quality soil nail wall. This chapter focuses on the most common aspects of soil nail wall inspection and monitoring. The discussion presented herein is applicable regardless of the contracting approach followed in a given project and what party is assigned inspection duties. Additional inspection measures may be necessary depending on specific project requirements and constraints.

Proper construction inspection procedures are a vital component of constructing the soil nail wall in accordance with the intent of the project plans and specifications. Inspection involves evaluation of the following: (i) conformance of system components to material specifications; (ii) conformance of construction methods to execution specifications, if any; (iii) identification of site or soil conditions that require adjustment to the installation procedures or to the original design; (iv) conformance to short-term performance specifications (i.e., load testing); and (v) 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 are conducted to assess the performance of the soil nail wall and its components, and may include short-term and long-term measurements. Short-term monitoring is usually limited to measurements during and possibly shortly after construction. Depending on the project, it may include measurements of vertical and lateral wall deflection, measurement of ground movement in the vicinity of the wall, and load testing of soil nails. Long-term monitoring of the soil nail wall is less typical and usually includes a continuation of measurements from short-term monitoring. Short-term monitoring of wall performance can be performed by the Owner or the Contractor depending on the contracting approach used. Long-term performance monitoring is usually conducted by the Owner.

The inspection personnel is responsible for ensuring that all required construction activities and testing have been completed in accordance with the contract specifications and plans. The inspector must understand all relevant aspects of the construction of soil nail walls, and must be intimately familiar with the design drawings and specifications of the project. The inspector must be equipped with the necessary safety gear, measurement instruments and tools to perform the inspection activities, with exception of gear and instruments that are to be provided by the Contractor. The inspector must verify that the gear provided is in working order and suitable for inspection work. The inspector is responsible for carrying and

maintaining soil nail installation logs, a daily log and field book, and for recording all relevant aspects of soil nail wall construction and related activities.

9.2 Inspection of Construction Materials

9.2.1 Materials Inspection

The quality of all materials used is controlled 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 design and specification requirements
- Laboratory testing of representative samples from materials delivered to the site or approved storage area

Steel components such as tendons, bearing plates, nuts, washers, welded wire mesh (WWM), and reinforcing bars are normally accepted based on mill certifications. Cement is accepted based on certification by the cement producer pending the results of unconfined compressive testing on grout and observation of soil nail installation. Other materials such as centralizers, coatings, grout additives, sheathing, geocomposite drainage material, and piping are normally accepted based on certificates provided by the manufacturer. In addition to the manufacturer's certifications, visual inspection of the materials is necessary to establish whether they have suffered damage during transport or storage.

Water used in the preparation of grout or shotcrete must be free from chemical or organic material that may negatively affect its performance. Potable water is generally suitable for preparation of grout or shotcrete without the need for testing.

9.2.2 Storage

Inspection must include verification of proper storage of the materials that will form part of the soil nail wall. All materials must be kept in a suitable location and protected as needed to prevent deterioration by the elements and construction activities. Cement is typically supplied either in bagged or bulk form, and must be kept dry and elevated from the ground during storage. The Contractor should avoid stacking the bags too high to prevent over compaction of the cement.

Tendons must be placed on supports to prevent direct contact with the ground and moisture in the ground. Support must be provided with appropriate spacing to prevent excessive bending of the bars. If excessive, the bending may compromise the tolerances or the

minimum cover. Similar care must be taken with other steel components and drainage materials, which should be stored in suitable containers and protected from damage.

9.2.3 Inspection of Corrosion Protection

The corrosion protection systems of all tendons must be inspected at the project site, prior to acceptance and use. Tendons with damaged corrosion protection must be repaired, if possible, or replaced. Inspection measures should also be performed for all other steel elements that are corrosion protected.

Encapsulated tendons are usually delivered to the site assembled. The annular space between the corrugated sheath and the tendon is grouted at the production facility. Inspection must include visual examination of the encapsulation to detect surface damage such as cracks or scores. Surface damage to the encapsulation may be repaired; however, any repairs should follow procedures approved by the manufacturer and are subject to approval of the Owner. A representative number of encapsulated tendons should be checked for voids by lightly tapping the encapsulation with a steel rod and listening for hollow sounds indicating the presence of voids. Generally, tendons with voids within the encapsulation should be replaced instead of attempting their repair.

Epoxy-coated tendons are susceptible to damage due to impact and scarring. Construction equipment should not travel directly on reinforcing elements, and elements should not be dragged, excessively bent, or field cut. Occasionally, tendons may be incompletely coated at the factory with visible areas of bare steel or areas with significant color variations. The epoxy coating should be visually examined for damage and inconsistencies. Nicks and dents may be repaired using a repair kit provided by the manufacturer. However, tendons showing areas with bare steel or coating thickness outside the range described in Chapter 7 should be replaced.

Galvanized tendons are also susceptible to damage during transport and storage, and must be inspected following similar procedures as epoxy coated tendons. Galvanized reinforcement should be well supported during lifting and handling to prevent excessive bending. Repairs to areas of limited damage of the protective layer must strictly follow manufacturer's recommendations.

9.3 Inspection of Construction Activities

9.3.1 Introduction

Inspection during construction of a soil nail wall should be carried out by qualified individuals. Depending on the particulars of the project, inspection may be entirely carried

out by the Geotechnical Engineer, by the wall Designer or Specialty Contractor, by a third party, or by a combination thereof. In this chapter, "inspector" refers to the person or entity performing the inspection of a given item. It must be noted that, in some cases, a third-party inspection firm may not be qualified to perform certain specialized inspection tasks identified in this chapter. In those cases, the geotechnical engineer or the wall designer must be engaged to complete those tasks as appropriate. This chapter does not differentiate between inspection entities and only focuses on the necessary inspection tasks. This differentiation must be established by the Owner or the wall designer as needed.

A discussion of inspection issues for each of the major construction steps is provided below. Note that Porterfield et al. (1994) provide considerably greater detail on inspection methods, nail testing, inspection forms, and handling of difficult ground conditions during construction.

9.3.2 Excavation

The two types of excavation that generally occur during construction of a soil nail wall are:

(1) Mass excavation, which is conducted to provide equipment access and general site grading
(2) Bench excavations required for construction of the soil nail wall (i.e., excavation to the finished wall line shown on the plans, or "neat line")

During mass excavation, the inspection personnel 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 with the plans within the specified tolerance. The following items should be reported on or noted by the inspector: [*Additional notes to the inspector are included in italics.*]

- The type, nature, and moisture condition of excavated materials and those present at the excavation face [*Particular attention should be placed on the presence of moisture and seepage, soft soil zones, sloughing and overbreaks, boulders, manmade features or obstructions, and, in general, any other observations relevant to the construction and performance of the soil nail wall. The inspector should prepare sketches of the encountered features with indication of dimensions, plan location to known references, and elevation.*]
- Unexpected soft zones, seepage areas, or fill [*These features should be specifically reported to the Owner's or the Contractor's designer for further action as needed because they have the potential to degrade the performance of the soil nail wall unless properly addressed during design and construction. Unexpected seepage may*

be addressed by installation of additional drainage features as needed. Zones of soft soil or fill that were not considered during design may represent a serious issue. In this case, a revision of the design may be warranted.]

- Recurrent raveling or sloughing of the excavation face [*This should be reported to the Owner as it may result in excessive ground surface movements above the wall, as well as be detrimental to wall performance. 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.*]
- The condition of the excavation face at the start and end of each work day, and which portions of the face have been left unprotected, if any [*Generally, but not always, the excavation face should not be left unprotected overnight.*]
- The stability of the retained soils, the area behind or upslope of the wall, and the bottom of the excavation, at least twice a day [*The inspector should examine, if safe to do so, the ground surface above and in the vicinity of the wall for cracks, openings, and any evidence of movement and/or seepage. The inspector must pay close attention to utilities, manholes, pavements, and any elements that may crack. The condition of the ground surface and any structures or utilities near the wall must be noted during the first day of construction, so that future observations can be evaluated. Ground movements may occur at a distance behind the wall equal to the wall height. Inspection beyond this width is usually unnecessary. The requirements for visual inspection may be adjusted depending on whether the project includes instrumentation and monitoring, and its intensity, frequency, and reliability.*]

9.3.3 Drilling

The drill holes should be located and oriented as shown on the plans and within the tolerances in the specifications. Any deviation in location or orientation beyond the specified tolerances must be noted and communicated to the Owner. Generally, the angle of the drill mast is measured using an angle tool placed directly on the drill mast. Care must be taken that the drill mast angle is representative of the tendon inclination as it enters the excavation face. The following items should be reported on or noted by the inspector: [*Additional notes to the inspector are included in italics.*]

- The equipment and installation method used by the Contractor [*The inspector must become familiar with the Contractor's drilling systems, as generally described in Chapter 3, and understand the potential effects on the borehole and retained ground.*]
- The pressure, flow rate, and volume of drill fluid (or grout for HBSNs) injected into the drill hole [*It is very important that the driller controls the pressure and volume to maintain adequate borehole progress, clearing of cuttings at the bit face, and to minimize infiltration into the formation. However, it is just as critical that the*

inspector note whether return flush becomes clogged, there is connection with other drill holes or the ground surface, or if heaving is resulting from pressurization from the drilling fluid.]

- Subsidence of ground above the drilling location or large quantities of soil removal with little or no advancement of the drill head [*In such cases, the Contractor will have to modify drilling procedures.*]
- The type of drilling method and tools used, including type of drill bit, casing diameter, final length of casing, length of drill string, and type of drilling fluid [*The inspector must also observe the cuttings, if possible, and note the type of material being drilled throughout the length of the hole. In addition, loss of drilling fluid must also be noted. The inspector must note zones of difficult and easy drilling. All of this requires continual communication between the driller and the inspector. An efficient tool to quantify drilling difficulty in a project, is noting the effective drilling time, discounting stoppages and time lags for coupling each section of drill string or per given drilling lengths, throughout advancement of the hole. Although this information cannot be directly correlated to ground properties or extrapolated to other project sites, it aids the inspector in learning to discern significant material changes during drilling within the site.*]
- The length of the drill bit, drill string sections and casing sections before the start of drilling, and the final length of the drill hole after the hole is flushed and before tendon insertion [*This observation will help to assess the drill length and inserted casing length at any time during installation. The inspector must also measure the diameter of the drill bit as the bit diameter should be equivalent to the nominal diameter of the drill hole.*]
- The length of time the drill hole is left open before the tendon is inserted [*Drill holes in soil should only be kept open for short periods of time to limit the potential for seepage, caving, and/or degradation of the walls of the hole. In general, a tendon should be inserted and grouted immediately after drilling of the hole. Exceptions to this practice must be established on a project-by-project basis according to the characteristics of the materials being drilled and job-site conditions, and provided that the Contractor takes adequate precautions to flush the hole as needed before tendon installation.*]

Special considerations are required for hollow bar installation. The inspector should record the following parameters:

- The drilling rate and grout pressure [*Generally, if using grout as the drilling fluid, it is extremely difficult to see the type of cuttings exiting the hole. Recording the drilling rate thus becomes particularly important during inspection of HBSN installation.*

This must be accompanied by noting the grout pressure measured at the surface and consistent return of grout to the face during drilling.]

- The type of drill bit and the number and size of grout exit ports [*Usually, this observation needs to be made only once in a project; however, in some cases, the Contractor may opt to switch drill bit type or size, or to modify the number or size of grout ports in the drill bit*.]
- The type of soil cuttings returning to the surface [*The inspector may, from time to time, attempt to sieve-wash the outflow grout to see the type of soil cuttings flushed to the surface. This may be especially useful if drilling through formations with widely different grain sizes or when drilling through organic soils*.]

9.3.4 Tendon Installation

The inspection personnel must verify that each tendon meets the length, diameter, steel grade, centralizer, and corrosion protection requirements in the plans and specifications. The tendon must be inserted into the hole to the minimum specified length. An inability to do so indicates an unacceptable condition caused by caving or sloughing of the hole. Insufficient drilled length can quickly be eliminated as potential cause of difficulty during installation of the tendon by consulting the drilling log maintained by the inspector for each soil nail.

Tendons must be handled carefully to avoid damage, with adequate support provided to those lifted by drill rigs or cranes. Centralizers should be stiff and large enough to provide space for the minimum specified grout cover. Centralizers should be spaced as shown in the construction drawings or as required by the specifications, but should not impede the free flow of tremied grout into the hole, or in the case of HBSNs, impede the drilling grout and flushed cuttings. The Contractor must use suitable devices during handling and lifting of the tendons to prevent damage to their corrosion protection.

9.3.5 Grouting

The entire length of the nail should be grouted without voids, intrusion of soil, or gaps in the grouted column. Generally this aspect of grouting cannot be directly verified by the inspector. However, the inspector can indirectly verify the integrity of the grout by inspecting the drill hole before grout placement, and by recording the volume of grout injected in the hole. Inspection of the drill hole was discussed previously and includes measurement of the drilled length.

The inspector must verify that grout is injected by tremie pipe inserted to the bottom of the drill hole in a continuous process. If the tremie pipe is withdrawn as grouting proceeds, the withdrawal rate should be controlled to ensure that the end of the tremie pipe is always below the grout surface. Grouting may be performed before or after insertion of the tendon. If

performed after insertion, the volume of the tendon must be considered in the theoretical volume calculation. Also, the inspector must verify that the tremie hose is properly attached to the tendon so as not to coil during tendon insertion, and so that it can be pulled from the hole after grouting without detriment to the tendon and its corrosion protection. Grouting can also be performed within a temporary casing followed by nail insertion and casing extraction.

When hollow-stem auger drilling 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.

The inspector must maintain a record of the volume of grout placed in the hole. Measurement of the volume of injected grout requires volume calibration of the holding tank or the use of a flowmeter, starts when final grout is injected in the hole, and ends when final grout exits the hole free of impurities. The volume of injected grout must be at least equal to the theoretical volume of the hole, although it is often 10 to 30 percent larger than the theoretical volume. If the volume of grout injected is less than 100 percent or greater than 130 percent of theoretical, the Contractor and the Owner must be notified, and remedial measures must be undertaken, if needed.

Another important aspect of grouting inspection is to measure the grout properties: namely specific gravity and strength. For production soil nails, specific gravity of the newly prepared, fluid grout should be the primary quality control mechanism and should be preferred to compression testing of grout specimens. In a neat cement grout, there is a direct relationship between the specific gravity of the newly prepared grout and its water-cement ratio. Because there is also a direct relationship between the water-cement ratio and the grout strength for a given type of cement, mixing method and mixing energy, and water source, the specific gravity becomes an indirect indicator of grout strength that can be obtained during nail installation rather than relying on testing days or weeks later.

Because the properties of cement and water will vary from location to location and may vary over time, grout cubes should be prepared using grout of known specific gravity and the project mixing equipment. These can then be tested in compression at the start of the project for each mix design to be used as a confirmation of the material properties. If the specimens meet the required specification strength, then the corresponding specific gravity can be used as a QC tool. For projects of extended duration or with a relatively large number of nails, or where there is a change in the cement source, water source, admixtures, grout batching, or grout mixing equipment, additional comparison cube strength tests should be performed.

Typically, the specific gravity is measured using an API specific gravity balance (i.e., Baroid mud balance), where grout is placed in a cup of known volume, capped, and then weighed in

accordance with ASTM D4380 or API RP 13B (API, 2009). The target specific gravity is typically in the range of 1.8 to 1.9 to produce fluid, pumpable grout with compressive strengths over the commonly used 4,000 psi. Grout cubes are prepared in the field and tested in a laboratory according to ASTM C109. Project specifications must clearly establish the type and minimum frequency of grout testing. Regular specific gravity testing conducted as the grout is mixed is commonly required. Molding of compressive test samples is often limited to daily or periodic samples depending on the Owner's preference.

For HBSNs, drilling grout may have a specific gravity of 1.4 to 1.6. After the bar is installed to the desired depth, a heavier final grout mixture (1.8 to 1.9) is pumped through the hollow bar. The nail is considered complete when the heavier mixture returns to the excavation face, signaling that the lighter drilling grout was flushed from the hole and that all drilling spoils have also been removed.

9.3.6 Geocomposite Strip Drains

Once the final wall line excavation and nail installation have been completed for each lift, the geocomposite strip drains are placed vertically, at specified intervals. Strip drains must be continuous from the top to the bottom of the wall. The inspector should verify that strip drains are installed in accordance with the construction drawings and specifications. Often, strip drains are fitted with an impermeable, protective liner on one side. The inspector must verify that this liner is not placed in contact with the soil, but rather positioned to receive shotcrete.

At the base of the soil nail wall, strip drains are connected either to a footing drain below the finished grade, or to weep holes that penetrate the finished wall. Weep holes should be located and spaced as shown on the plans, coinciding with the strip drain locations. A filter fabric is placed against weep holes behind the wall to prevent clogging. Alternatively, if drainage aggregate is used, it must be wrapped in 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. The perforated pipe should not be wrapped in filter fabric; rather, the aggregate surrounding the pipe may be wrapped in filter fabric as indicated in the project plans.

9.3.7 Wall Facing

After the geocomposite strip drains are installed, the reinforcing steel is placed and shotcrete is applied to the lines and grades specified. The WWM and 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), or as the

Contractor's preferred construction sequence, shotcrete facing may be placed before nails are drilled and installed.

Conventional shotcrete procedures as described in the specifications (see Appendices E and F) are applicable. 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 underlying lift of shotcrete. Acceptable methods for cleaning the shotcrete joint include washing with a combination of injected water and compressed air, or blowing with compressed air. 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 shotcrete. The specifications may require the nozzleman to provide proof of certification by the ACI or the American Shotcrete Association (ASA), and to complete a pre-qualification test panel prior to beginning production work.

The inspector should be aware of the following basic recommended practices for applying shotcrete:

- The nozzle should be held perpendicular to the exposed excavated surface, except when shooting around reinforcing bars
- Optimum nozzle distance from the surface for shotcrete application is: 2 to 5 ft for wet-mix, 3 to 6 ft for dry-mix
- Placement of shotcrete should start at the bottom
- Voids must not be allowed to form behind bars, plates, or steel mesh
- Where sharp edges and accurate lines are required, these should be set out by screen boards, guide wires, and/or depth spacers

The inspector must check that the shotcrete lines have been properly established and provide the minimum thickness within the tolerance established in the specifications. The inspector must also verify that the surface of the shotcrete is being properly finished according to the specifications and to the inclination established in the project plans. Also, the inspector must observe the installation of bearing plates with appropriate headed studs as required, and verify that they are positioned while the shotcrete is wet and without voids underneath. In the case of initial facing before placement of the final facing, the inspector must verify that the bearing plates are fitted with the proper number and diameter of headed studs as per the project plans.

Construction of a final cast-in-place facing must follow typical inspection procedures for cast-in-place concrete. As part of these procedures, the inspector must verify that the reinforcing steel is firmly placed at the locations and to the dimensions specified in plans, the

reinforced concrete and shotcrete thicknesses are as shown on plans, and that no deleterious material is present over the reinforcing steel before concrete pumping.

9.4 Load Testing

9.4.1 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 standard practice to complete testing in each row of nails prior to progressing to the next lower level. This requirement of completing all testing in the row above, before proceeding, may be relaxed, at the direction of the Engineer. This most often occurs for walls of significant lateral extent to facilitate the construction sequence.

If test results indicate faulty construction practices or if the resistance of soil nails is less than that required, the Contractor should be required to alter nail installation and/or construction methods. Testing procedures and nail acceptance criteria must be included in the specifications.

The inspector must observe and log the results of the load tests performed and must ensure that the load testing schedule follows the specifications and approved testing submittals provided by the Contractor. The inspector may also provide input during selection of proof test nails based on observations during construction.

Load testing consists of verification tests and proof tests. A bonded length must be created for verification and proof test soil nails in the specific layer where bond is to be measured and an unbonded length must be proved above. In proof tests, the unbonded length is established as the minimum stressing length necessary for the test. The Contractor may achieve the formation of the unbonded length (referred to as "debonding") by any method as long as it can be shown that the debonding is effective. Grout should not be allowed to be pumped above the assumed bond length because the load would be transferred upward through the grout in the unbonded zone (i.e., the "donut" effect described in Cadden et al. 2010) and would result in artificially high calculated bond stresses.

Typical methods for establishing the unbonded length include placing a painted centralizer, or a piece of survey ribbon, on the nail at the desired point of grout termination (top of bonded zone), prior to insertion of the nail into the drill hole. 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. Both grout stages are subject to the same inspection procedures described previously.

For HBSNs, creating the unbonded length presents some challenges. Several methods have been employed to effectively create a length of nail that is free to move with respect to the soil. Since hollow bars are drilled with grout, the end product of installation is a bar fully grouted along its length with a complete column of grout. Following installation, the grout can then be removed by flushing using a tremie pipe and water. The unbonded zone may also be created through isolation by using a pre-installed smooth-casing bond breaker. However, Cadden et al. (2010) state that the estimated bond strengths using this method were artificially high. No universally established and accepted installation procedures exist to allow consistent creation of bonded and unbonded zones within a test HBSN for selective bond strength verification (Cadden et al. 2010). Consequently, reported values of bond strength from different contractors can vary greatly.

9.4.1a Verification Load Tests

Verification load tests are conducted to verify the compliance of the pullout capacity resulting from the Contractor's installation methods with those values of pullout capacity and bond strengths used in design. Verification tests should be conducted to a maximum load that induces bond stresses at least equal (or greater if desired) than the nominal bond resistance used in design for specific strata. When verification tests are intended to be taken to complete pullout, it may be necessary to use an oversized tendon to ensure that the ground-grout bond is achieved before the tendon is overstressed.

The number of verification load tests must be defined by the Engineer based on the ground conditions and wall geometry. The number of verification tests must consider the number of relevant soil strata to which production nails are to be bonded. A number of tests per strata or per area of wall can be specified. Verification test locations should be shown on the project documents. Test locations are commonly selected such that two verification tests are conducted in each anticipated major soil strata. Alternatively, the designer may specify the number and location of verification tests using the design nail length. In this way, the tests will verify the bond strength in any combination of strata that the production nails will engage. If the Contractor makes substantive changes in the drilling or soil nail installation operation, or if significant variability of the ground conditions are observed, additional verification tests may be required. Verification tests are performed on "sacrificial" test nails, which are not incorporated into the permanent work. Additional details of verification testing are provided below.

9.4.1b Proof Tests

Proof tests are conducted during construction on a minimum of 5 percent of the total production nails that are installed. Proof tests are intended to verify that there are no significant variations in soil nail performance throughout the wall. When selecting nails for proof testing, the installation records of the nails prepared by the inspector should be considered. Proof testing should be conducted where issues were observed during installation of adjacent production nails, such as irregular drilling progress, lower than typical grout take, or those that had to be redrilled. Verification tests should not be counted towards the minimum of 5 percent of the total production nails.

9.4.1c Creep Tests

Creep tests are performed as part of 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, requiring a sustained load hold period of 10 to 60 minutes. This test is performed to ensure that the nail can safely carry the intended load throughout the structure service life with low or predictable creep displacements under constant loads.

9.4.2 Equipment for Testing

A center-hole hydraulic jack and hydraulic pump are used to apply a test load to a nail tendon. 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 reaction block must be sized appropriately to avoid overstressing the shotcrete or bearing surface. The jacking frame should not react directly against the nail grout column during testing. Once the jack is centered and aligned, an alignment load (AL) should be applied to the jack to secure the equipment and minimize the slack in the setup. AL should not exceed 2.5 percent of the maximum test load, as defined subsequently. The hydraulic jack must be appropriately sized so that the anticipated test load is not greater than 80% of the rated jack capacity. Figures 9.1 and 9.2 show typical 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 setup and wall. The use of two dial gauges provides: (i) an average reading in case the loading is slightly eccentric due to imperfect alignment of the jack and the nail tendon, and (ii) 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 0.001 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 1 in., or at least 2 in.



Figure 9.1: Photo. Soil nail load testing setup. Source: Cadden et al. (2010).



Figure 9.2: Photo. Soil nail load testing setup against shotcrete facing. Photograph courtesy of Schnabel Engineering.

A hydraulic jack is used to apply load to the nail tendon while a pressure gauge is used to correlate to the applied load. At the Owner's discretion, 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 should be used as a means to monitor a constant applied load while the hydraulic jack pump is incrementally adjusted. Over extended periods of time, small load loss in the jack will not be reflected with sufficient accuracy using a pressure gauge due to its low resolution. Recent calibration data for the jack, pressure gauge, and load cell must be obtained from the Contractor prior to testing. Figure 9.3 is a schematic of a typical hydraulic jack used in soil nail applications. The figure depicts a jack support that is adjustable for inclination which, in typical practice, is not necessary (see Figure 9.2).

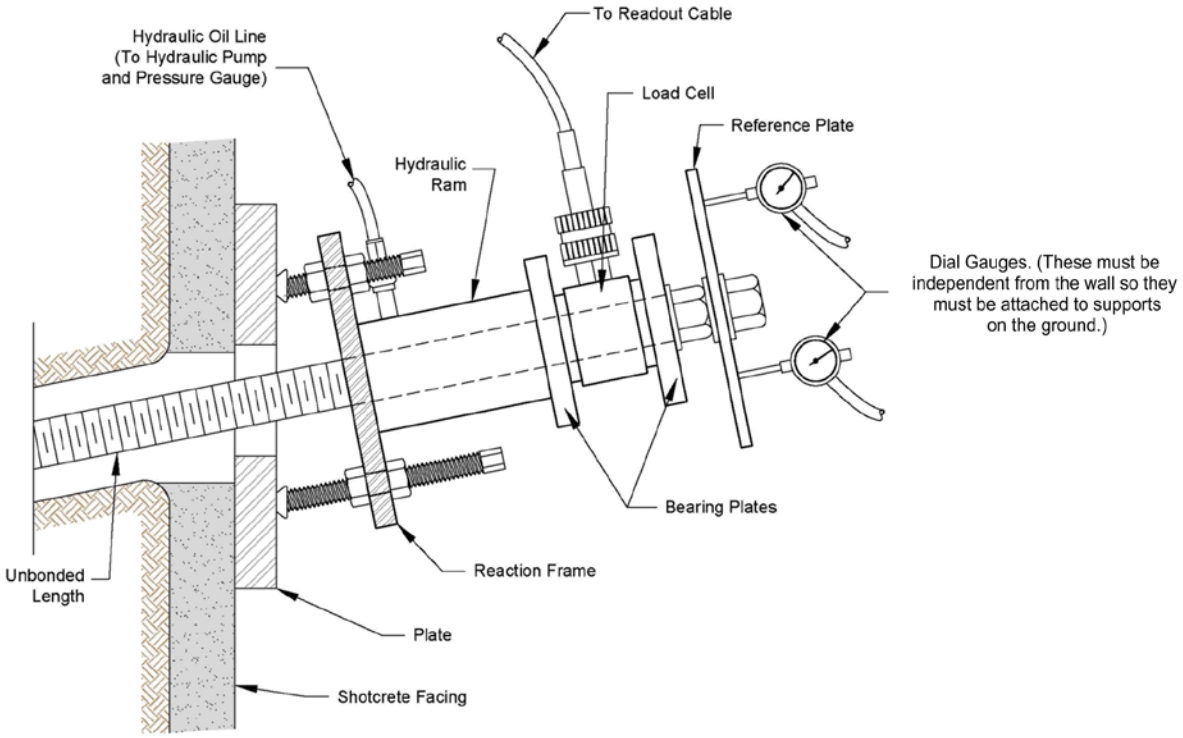


Figure 9.3: Illustration. Hydraulic jack used for soil nail load testing. Source: Porterfield et al. (1994).

9.4.3 Details of Verification Testing

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. The verification tests must be conducted on nails as considered in design and constructed with the same construction methods to be used on production nails. It would be optimal for verification tests to reach pullout, which is defined as the test load at which attempts to further increase the test load increments result in continued movement of the tested nail, when the maximum test load is applied. However, this may not be possible in some cases. Pullout, may take place before test load is achieved, or pullout may not be achieved for the selected maximum test load.

Verification tests provide the following information:

- Determination of the actual bond resistance if carried to pullout
- Verification that the actual bond resistance is larger than the nominal bond resistance used in design
- Potential for creep of the soil nail

At a minimum, verification test loading must be carried out to a load defined by the nominal bond strength. Test loads in excess of this minimum, and preferably to pullout, are desirable, as they provide considerably more information and may lead to more economical soil nails.

Verification test nails must have a bonded (grouted) length and an unbonded length. Fully grouted soil nails must not be used. The unbonded length must extend from the back of the bearing plate to the top of the bonded length. The unbonded length must be at least 3 ft within the drill hole to avoid transferring added stresses originating under the bearing plate during the load test to the upper bond zone: an effect that might artificially increase the bond strengths. Besides the minimum 3 ft of unbonded length within the drill hole, extra bar length must be considered outside the drill hole to allow connecting the bar with the load test assembly. These extra bar lengths include the jack length and the "tail" length.

The bonded length of test soil nails during verification tests, $L_{B\,VT}$, must be selected such that a bond limit state is achieved before a tensile limit state is achieved, as indicated below:

(1) Select $L_{B\,VT\,max}$

For Grades 60 and 75 and other mild steel in accordance with ASTM A615, the maximum length, $L_{B\,VT\,max}$, is defined as:

$$L_{B\,VT\,max} = \frac{A_t \times f_y \times C_{RTY}}{r_{PO}}$$

Equation 9.1: Maximum bond length in verification tests (Grades 60 and 75).

For Grade 150 and other high-strength steel in accordance with ASTM A722, the maximum length is defined as:

$$L_{B\,VT\,max} = \frac{A_t \times f_u \times C_{RTU}}{r_{PO}}$$

Equation 9.2: Maximum bond length in verification tests (Grade 150).

Where:

C_{RTY} = reduction coefficient for mild-grade steel = 0.9

C_{RTU} = reduction coefficient for high-strength steel = 0.8

A_t = cross-sectional area of the test tendon

f_y = yield strength of test tendon (mild steel)

f_u = ultimate tensile strength of test tendon (high-strength steel)

r_{PO} = nominal pullout resistance (per unit length) of soil nail = $\pi \times q_u \times D_{DH}$

q_u = bond strength

D_{DH} = drill hole diameter

(2) If $L_{B\ VT\ max} > 10$ ft

Select $L_{B\ VT}$ to be 10 ft $\leq L_{B\ VT} \leq L_{B\ VT\ max}$.

(3) If $L_{B\ VT\ max} < 10$

To avoid tensile breakage, select $L_{B\ VT} = 10$ ft and increase the test tendon size as needed, and recalculate $L_{B\ VT\ max}$ until $L_{B\ VT\ max} > 10$ ft.

The maximum load during the verification test is defined as the Verification Test Load (VTL) and is calculated as follows:

$$VTL = L_{B\ VT} \times r_{PO}$$

Equation 9.3: Maximum load in verification tests.

VTL is defined in terms of a maximum, "ultimate" load, not a "design" load, as was done in the previous version of this manual. To accommodate the definition of VTL without changing the load testing practice, the factors multiplying VTL in the load schedule of Table 9.1 are different from those presented in the previous edition of this manual. Also note that VTL is not affected by a safety factor for pullout as was the case with the previous version of this manual, where the maximum testing load was 2.0 times the "design" load in the verification test. The definition of VTL does not depend on a pullout resistance factor either.

If pullout is not achieved up to VTL, it is optional to apply test loads larger than VTL to attempt to reach pullout. Unload/reload cycles at intermediate loads are possible, but these are rarely conducted.

Verification tests are performed according to the schedule in Table 9.1.

Table 9.1: Load Schedule in Verification Tests

Load	Hold Time (minutes)[2]
AL[1]	1
0.13 VTL	10 (recorded at 1, 2, 4, 5, 10)
0.25 VTL	10 (recorded at 1, 2, 4, 5, 10)
0.38 VTL	10 (recorded at 1, 2, 4, 5, 10)
0.50 VTL	10 (recorded at 1, 2, 4, 5, 10)
0.63 VTL	10 (recorded at 1, 2, 4, 5, 10)
0.75 VTL (Creep Test)	60 (recorded at 1, 2, 4, 5, 6, 10, 20, 30, 50, 60)
0.88 VTL	10
1.00 VTL	10
AL	1[3]

Notes: (1) AL = alignment load, which is commonly less than or equal to 0.025 VTL.
(2) Soil movement must be measured after each load increment has been achieved and at each time step.
(3) Permanent soil nail movement must also be recorded.

The Contractor must record soil nail movements at each load increment. As noted above, each load increment is held for at least 10 minutes. Creep tests are performed at 0.75 VTL. Test loads in excess of this minimum, and preferably to pullout failure, are recommended.

Intermediate load steps during unloading, which should be optional, may be added to create a stepped sequence. Stepped unloading can be used to obtain the shape of the unloading curve with better definition. This can be helpful when only one strain gauge (see below) is used and the jack movement cannot be monitored accurately. Stepped unloading can help provide an improved estimate of the permanent soil movement into elastic and non-recoverable components. Stepped unloading can include one to seven additional intermediate load steps. Each additional load step in stepped unloading should be held until the readings in the gauges are stable.

Test acceptance criteria require that:

- Pullout does not occur at loads less than 1.00 VTL.
- The total movement (Δ_{VTL}) measured at VTL must exceed 80 percent of the theoretical elastic elongation of the unbonded length (L_{UB}), as defined below.
- The creep movement does not exceed the criteria presented in Section 9.4.5.

$$\Delta_{VTL} > 0.8 \frac{VTL\, L_{UB}}{E\, A_t}$$

Equation 9.4: Theoretical elastic elongation of the unbonded length in verification tests.

Where:

E = Young's modulus of steel (29,000 ksi)

This last item 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 also contained in the specifications (Appendices E and F).

Figure 9.4 shows a data log sheet that can be used as a model for load testing of soil nails. Figure 9.5 presents an example of data reduction of soil nail load testing to calculate the elastic movement.

Project:	Example	Client/Owner:	City of Anwhere, USA
Project No. :	1234	Test Date:	1/2/2014
Location:	Anywhere, USA	Performed by:	Perfect Contractor Co.
Recorded by:	XYZ	Checked by:	ABC
Nail No.:	A-1	Primary Grouted Date:	1/3/2015
Drilled Date:	1/1/2015	Post-Grouted Date:	N/A

Total Bar Length:	25 ft	Yield Strength of Steel:	75 ksi
Tail Length:	4.5 ft	Design Bond Strength:	10 kip/ft
Jack Length:	4.0 ft	Cross-Sectional Area of Steel:	29000 in^2
Unbonded Length:	6.5 ft	Yield Load:	300 kip
Bond Length:	10 ft	Steel Modulus of Elasticity:	4.0 ksi

Load			Elapsed Time (minutes)	Movement			Comments
Load (% VTL)	Applied Pressure (1) (psi)	Applied Load (kip)		Dial Gauge 1 (in.)	Dial Gauge 2 (in.)	Nail Movement (2) (in.)	
3	150	5.1		0.000	0.000	0.000	Alignment Load
13	825	25.4	0:01	0.040	0.024	0.032	
...	...	...	...	...	...	...	
13	825	25.4	0:10	0.040	0.024	0.032	
25	1650	50.2	0:01	0.096	0.074	0.085	
...	...	...	...	...	...	...	
25	1650	50.2	0:10	0.110	0.083	0.097	
38	2475	75.1	0:01	0.137	0.103	0.120	
...	...	...	...	...	...	...	
38	2475	75.1	0:10	0.137	0.103	0.120	
50	3325	100.6	0:01	0.152	0.128	0.140	
...	...	...	...	...	...	...	
50	3325	100.6	0:10	0.165	0.135	0.150	
75	4975	150.3	0:01	0.255	0.225	0.240	Start of Creep Test
75	4975	150.3	0:02	0.260	0.23	0.245	
75	4975	150.3	0:04	0.265	0.235	0.250	
75	4975	150.3	0:05	0.270	0.24	0.255	
75	4975	150.3	0:06	0.265	0.235	0.250	
75	4975	150.3	0:10	0.273	0.242	0.258	
75	4975	150.3	0:20	0.278	0.245	0.262	
75	4975	150.3	0:30	0.287	0.249	0.268	
75	4975	150.3	0:50	0.290	0.258	0.274	
75	4975	150.3	1:00	0.297	0.263	0.280	
75	5800	175.1	0:01	0.330	0.310	0.320	
...	...	...	...	...	...	...	
88	5800	175.1	0:10	0.340	0.32	0.330	
100	6650	200.7	0:01	0.418	0.382	0.400	Max Load Test, VTL
...	...	...	...	...	...	...	
100	6650	200.7	0:10	0.421	0.389	0.405	
...	...	...	...	...	...	...	Intermediate Unloading Steps Not Shown
3	150	5.1	0:01	0.080	0.040	0.060	

Notes:(1) Jack calibration chart provided seperately

(2) Nail movement is calculated as the average of dial gauge readings after they have been "zeroed" relative to initial dial gauge readings.

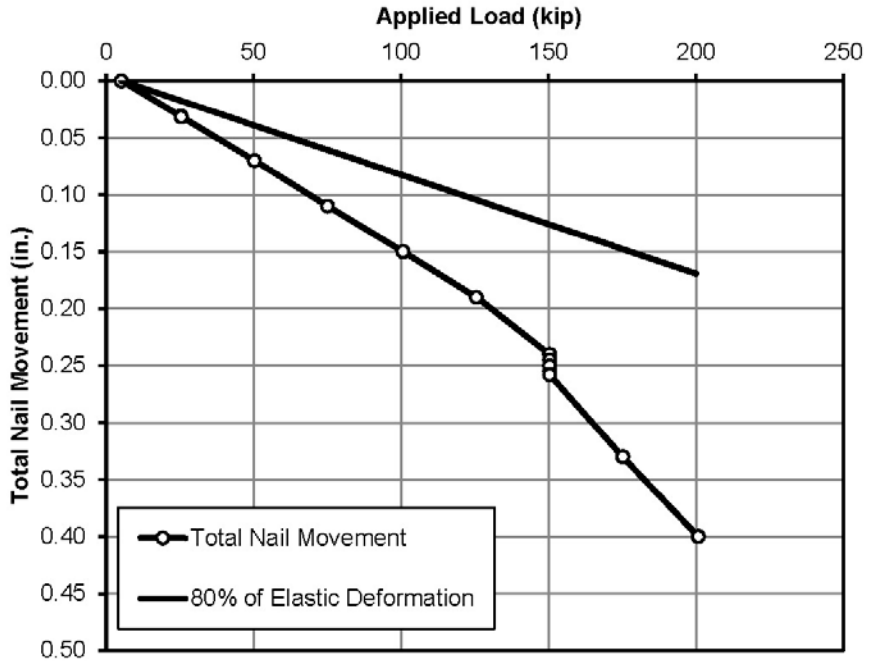
Figure 9.4: Chart. Typical data sheet for soil nail load testing.

Project:	Example	Client/Owner:	City of Anwhere, USA
Project No. :	1234	Test Date:	1/2/2014
Location:	Anywhere, USA	Performed by:	Perfect Contractor Co.
Recorded by:	XYZ	Checked by:	ABC
Nail No.:	A-1	Primary Grouted Date:	1/3/2015
Drilled Date:	1/1/2015	Post-Grouted Date:	N/A

Total Bar Length:	25 ft	Yield Steel of Steel:	75 ksi
Tail Length:	4.5 ft	Design Bond Strength:	10 kip/ft
Jack Length:	4.0 ft	Cross-Sectional Area of Steel:	29000 in^2
Unbonded Length:	6.5 ft	Yield Load:	300 kip
Bond Length:	10 ft	Steel Modulus of Elasticity:	4.0 ksi

Verification Test Load Acceptance Criteria

Total Nail Movement at Verification Test Load	0.405 in.	Pass
80% of Elastic Deformation at VTL	0.17 in.	



Pullout before VTL?	No	
80% of Elastic Deformation at VTL > Total Nail Deflection?	No	PASS
Creep movement (see separate sheet)?	No	

Figure 9.5: Chart. Example of data reduction from soil nail load testing.

9.4.4 Details of Proof Tests

Proof tests are single-cycle tests in which the load is applied in increments to a maximum proof test load (PTL).

Proof test nails must also have a bonded length and a temporary unbonded length. Similarly to the verification test, the temporary unbonded length must extend from the back of the bearing plate to the top of the grouted length. The temporary unbonded length must be at least 3 ft within the drill hole. In addition, extra bar length (including the jack length and the "tail" length) must be considered outside the drill hole to connect the bar with the load test assembly.

Similarly to the verification test, the bonded length of test soil nails during proof tests, $L_{B\ PT}$, must be selected such that a bond limit is achieved before a tensile limit state is achieved, as indicated below.

(1) Select $L_{B\ PT\ max}$

For Grade 75 and other mild steel in accordance with ASTM A615, the maximum length, $L_{B\ PT\ max}$, is defined as:

$$L_{B\ PT\ max} = \frac{A_t \times f_y \times C_{RTY}}{r_{PO} \times 0.75}$$

Equation 9.5: Maximum bond length in proof tests (Grades 60 and 75).

For Grade 150 and other high-strength steel in accordance with ASTM A722, the maximum length, $L_{B\ PT\ max}$, is defined as:

$$L_{B\ PT\ max} = \frac{A_t \times f_u \times C_{RTU}}{r_{PO} \times 0.75}$$

Equation 9.6: Maximum bond length in proof tests (Grade 150).

(2) Select $L_{B\ PT\ max}$ to be 10 ft or $L_{B\ PT\ max}$, whichever is smaller, to avoid tensile breakage.

Production proof test nails that are shorter than 12 ft may be tested with less than the minimum 10-ft bond length. Fully grouted test nails must not be proof tested.

The maximum load in the proof test (PTL) is as follows:

$$PTL = L_{B\,PT} \times r_{PO} \times 0.75$$

Equation 9.7: Maximum load in proof tests.

Proof tests are conducted according to the loading schedule of Table 9.2. Each load increment is held for at least 10 minutes. The Contractor must record soil nail movements at each load increment and the time intervals shown in the table for each load step. Creep tests are performed at 1.00 PTL.

The acceptance criterion requires that:

- No pullout occurs at loads less than 1.0 PTL.
- The total soil nail movement (Δ_{PTL}) measured at PTL is greater than 80 percent of the theoretical elastic elongation of the unbonded length, as defined below.
- The creep movement does not exceed the criteria presented in Section 9.4.5.

$$\Delta_{PTL} > 0.8 \frac{PTL\, L_{UB}}{E\, A_t}$$

Equation 9.8: Theoretical elastic elongation of the unbonded length in proof tests.

Proof tests are also subject to the creep test criteria described in Section 9.4.5.

For similar considerations made for VTL, the load schedule in proof tests of Table 9.2 is different than that presented in the previous edition of this manual because of the definition of PTL in terms of a maximum load, not as a multiple of a “design” load.

Table 9.2: Load Schedule in Proof Tests

Load	Hold Time (minutes)[3]
AL[1]	1
0.17 PTL	Until Movement Stabilizes [4]
0.33 PTL	Until Movement Stabilizes
0.50 PTL	Until Movement Stabilizes
0.67 PTL	Until Movement Stabilizes
0.83 PTL	Until Movement Stabilizes
1.0 PTL (Creep Test)[2]	10 recorded at 1, 2, 4, 5, 6, and 10
AL	1

Notes: (1) AL = alignment load, which should be AL≤ 0.025 PTL.
(2) If the nail movement measured between 1 and 10 minutes exceeds 0.04 in., PTL must be maintained for 50 additional minutes and movements must be recorded at 20, 30, 50, and 60 minutes. The permanent soil movement must also be recorded.
(3) Times are measured after the target load has been achieved in each increment.
(4) If the soils reinforced with nails are relatively susceptible to deformation of creep, it is recommended to hold each load increment for 10 minutes and to record the soil nail movement at 1, 2, 5, and 10 minutes.

9.4.5 Details of Creep Tests

Creep tests, which are part of verification and proof tests, are 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 for the creep movement require that:

Verification Tests

- The creep movement between the 1- and 10-minute readings at 0.75 VTL is less than 0.04 in.
- The creep movement between the 6- and 60-minute readings at 0.75 VTL is less than 0.08 in.
- The creep rate is linear or decreasing throughout the creep test load-hold period.

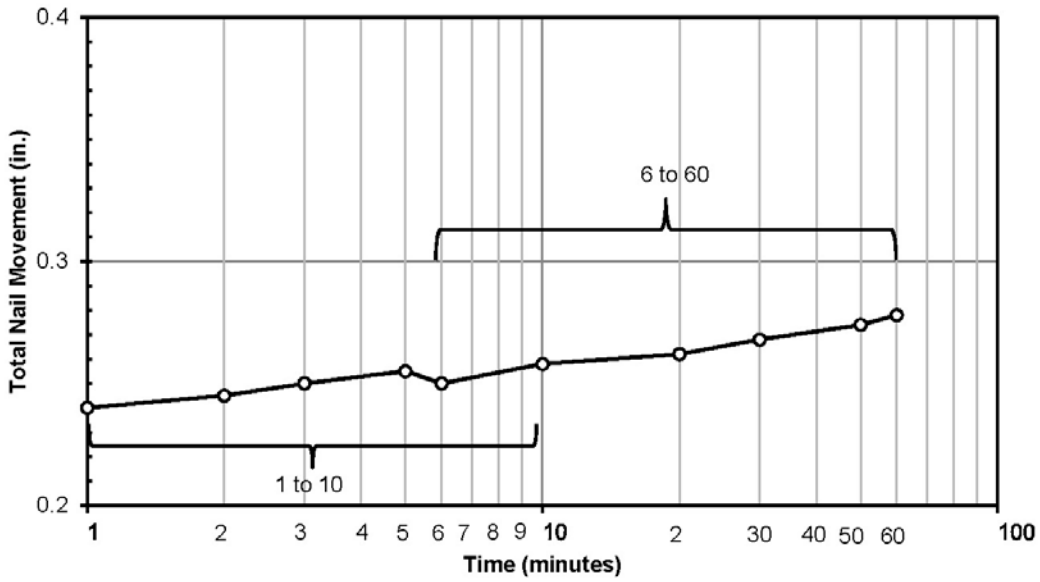
Proof Tests

- The creep movement is less than 0.04 in. between the 1- and 10-minute readings.
- If this movement is exceeded, PTL must be maintained for an additional 50 minutes with readings recorded at 20, 30, 50, and 60 minutes.
- If the creep test is extended, the creep movement between the 6- and 60-minute readings is less than 0.08 in.

The creep criteria described above are based on experience and current practice with ground anchors and have been established to ensure that nail design loads can be safely carried throughout the service life of the structure. Figure 9.6 presents an example of data reduction of soil nail load testing to calculate the creep movement between the 1- and 10-minute readings.

Project:	Example	Client/Owner:	City of Anwhere, USA
Project No. :	1234	Test Date:	1/2/2014
Location:	Anywhere, USA	Performed by:	Perfect Contractor Co.
Recorded by:	XYZ	Checked by:	ABC
Nail No.:	A-1	Primary Grouted Date:	1/3/2015
Drilled Date:	1/1/2015	Post-Grouted Date:	N/A

Total Bar Length:	25 ft	Yield Strength of Steel:	75 ksi
Tail Length:	4.5 ft	Design Bond Strength:	10 kip/ft
Jack Length:	4.0 ft	Cross-Sectional Area of Steel:	29000 in^2
Unbonded Length:	6.5 ft	Yield Load:	300 kip
Bond Length:	10 ft	Steel Modulus of Elasticity:	4.0 ksi



Creep Test Acceptance			
Time Range (min.)	Criteria (in.)	Data (in.)	Pass/Fail
1 to 10	0.04	0.018	Pass
6 to 60	0.08	0.03	Pass

Figure 9.6: Chart. Example of data reduction from a soil nail creep test.

9.4.6 Strain Gauges

Strain gauges can be used to measure the axial strain of the tendon during testing. Given Young's modulus of elasticity of steel (typically 29,000 ksi), the axial load in the tendon may be calculated at the strain gauge location. The placement of several spot-weldable strain gauges along the tendon permits the determination of the variation of axial loads along the length of the nail. Therefore, the change in axial load and the average mobilized bond resistance between two consecutive strain gauges under any applied load may be estimated. In addition, the load increase in a soil nail over time may also be assessed. It is often

possible to ascertain when the bond resistance is fully mobilized along a portion of the tendon before geotechnical pullout occurs.

The grout is typically assumed to crack during tension loading of the tendon, and its strength and stiffness are disregarded. However, this practice may result in significant errors in the interpreted axial load magnitude at low loads and, thus, may result in the overestimation of mobilized bond. The grout modulus should generally be accounted for if the measured strain is less than the limit tensile strain of the grout.

A significant advantage of using strain gauges is that they allow determining the actual bond resistance in tests where pullout does not occur, and allow the determination of variations in mobilized bond resistance in a test nail that may be bonded into two or more strata. In situations when obtaining correct strain gauge readings is critical, redundant strain gauges must be installed.

9.5 Instrumentation and Monitoring

9.5.1 Introduction

Performance monitoring should be included in any critical or unusual soil nail wall installation. Examples of critical or unusual installation are walls constructed for road widening projects under existing bridges, and walls with high surcharge loading. In addition, performance monitoring should be considered when a permanent soil nail wall is constructed by an Owner for the first time.

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. 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 1 year 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.

9.5.2 Monitored Parameters

The most significant indicator 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 for the sections where they are installed. Monitoring of optical

survey points at the top of the wall and, where practical, on the wall face provide information on actual face deformations that can be correlated with inclinometer data at specific sections.

9.5.3 Monitoring and Action Plan

Monitoring during wall construction should be performed to obtain data on the overall wall performance. At a minimum, a performance-monitoring plan should typically include measurement of the following quantities:

- Horizontal movements of the wall face using surface markers on the facing with optical surveying methods, as well as inclinometer casings installed 3 to 8 ft behind the facing, but generally not farther than 30 percent of the wall height if only one row of inclinometers is used.
- 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 across the cracks.
- Local movements and/or deterioration of the facing using visual inspections and instruments such as crack gauges.
- Drainage behavior of the structure, especially if groundwater is observed during construction, by visual observation of outflow points or through standpipe piezometers installed behind the facing.

A more comprehensive monitoring plan soil nail wall performance- may be established for a number of reasons. In the case of walls constructed through and over fine-grained soils, it may also be important to monitor the ground surface at the toe and in front of the wall for signs of cracking and bulging that would suggest basal heave or global instability. Where soil nail walls are used as part of a stabilization system in soils undergoing movement, additional instrumentation may be necessary as determined by geotechnical data needs. Additional monitoring or instrumentation may be required for monitoring specific safety aspects during construction: to validate design changes or value engineering; to control excavation rates or other aspects of construction that may impact stability in certain cases; and to enhance 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 and possible redistribution of load in creep-susceptible soils. Ideally, strain gauges are attached to the nail tendon in pairs, are

mounted top to bottom at 5-ft spacing, and diametrically opposed to address bending effects. The end of the tendon should be inscribed so that the final orientation of the strain gauge can be verified.

- Inclinometer arrays to measure horizontal movements of the structure at various distances behind the wall.

A typical instrumentation layout for a comprehensive monitoring plan is shown in Figure 9.7.

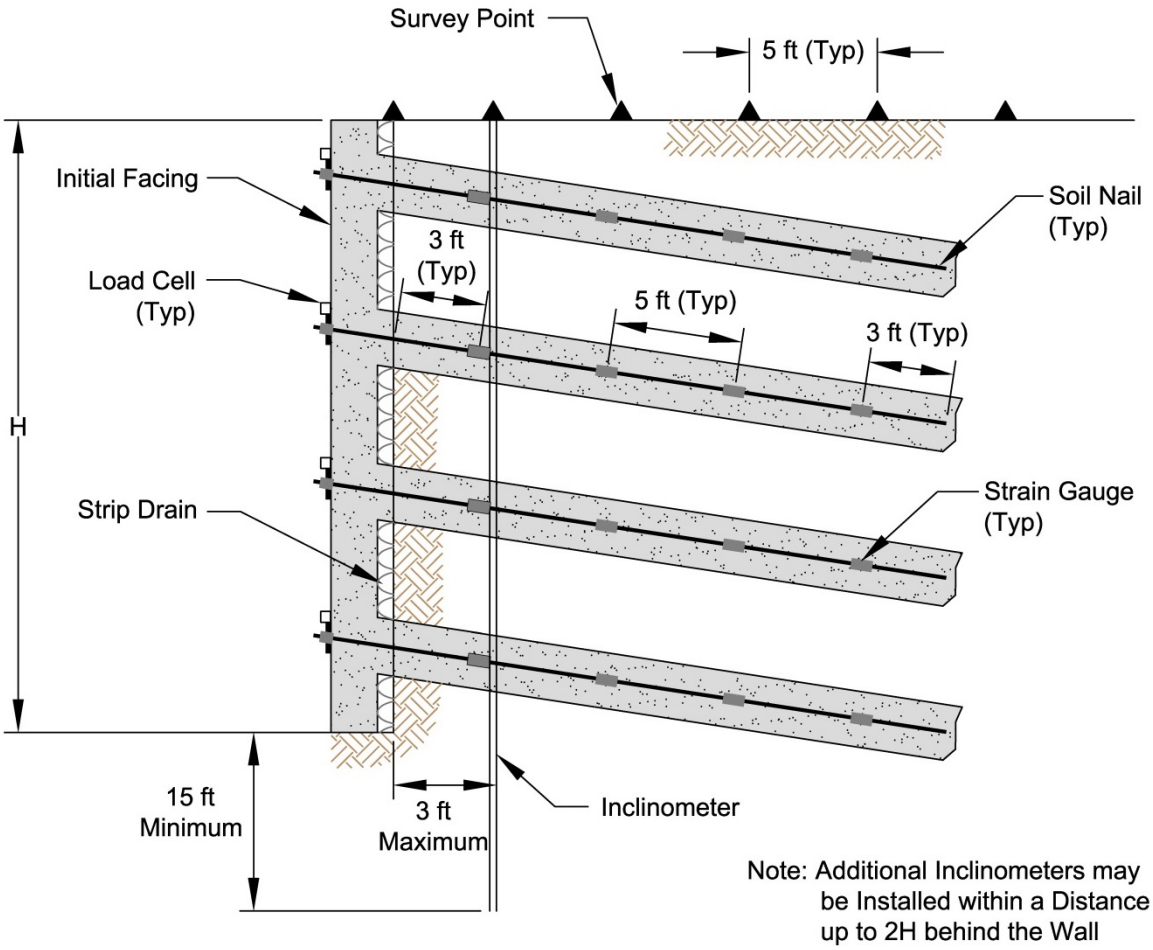


Figure 9.7: Illustration. Typical comprehensive monitoring instrumentation. Source: Byrne et al. (1998).

A monitoring program desired by the Owner must be well defined in the design documents regardless of its purpose, characteristics, and duration. The monitoring plan must indicate type and location of the instruments, frequency of measurements and visual observations, and frequency of reporting. The plan must also clearly define minimum requirements for reporting the data, and must identify the reporting lines and parties involved.

Establishment of movement threshold criteria and an action plan are essential elements of a monitoring plan. The action plan is typically developed by the Contractor, although minimum action requirements may be established in the project specifications. The action plan addresses the response steps to be taken for certain occurrences that can be reasonably anticipated. The action plan must identify steps that can be reasonably carried out within the context of the project and within a reasonable time frame.

9.5.4 Monitoring Instruments

Instruments for monitoring of soil nail wall performance 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.

9.5.4a Inclinometers

Inclinometers provide the most comprehensive data on wall deformations. Inclinometers are a well-established technology and are commercially available from several manufacturers. In recent years, in-place inclinometers have been used extensively. They allow rapid measurements at points throughout the full depth of the inclinometer without the need to physically introduce and remove a probe into the inclinometer casing to take readings at every depth. However, in-place inclinometer readings should be verified with manual readings using a standard probe with certain frequency.

9.5.4b Survey Points

Typically, reflector prisms are attached to the wall face or installed on stable bases on the ground surface and read directly by optical surveying methods or indirectly with electronic distance measuring (EDM) equipment. Optical or laser survey systems are typically capable of measuring horizontal and vertical displacements to an accuracy of 0.1 in. or better within a 300-ft range.

In recent years, automated total stations have been used more frequently for optical surveying of retaining structures during construction. One advantage of an automated total station system is a reduction in reading errors by using an instrument that is semi-permanently installed. Also, the higher reading frequency that can be accomplished with such systems permits discerning periodic, environment-associated movements from gravity-induced wall and ground movements. In projects of long duration, automated systems may result in savings compared with traditional surveys.

9.5.4c 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.

Vibrating wire strain gauges are the most commonly used in soil nail testing applications in the U.S. A strain gauge is used with "sister bars" where pieces of rebar are attached on each end of the strain gauge (Figure 9.8). The installation consists of attaching both sister bars to the nail tendon before installation and grouting. As the nail tendon elongates during testing, the sister bars also deform and act on the strain gauge, which provides a frequency reading that is read by the datalogger box and is transformed into a microstrain reading (strain = microstrain reading x 10^{-6}).

Another common type of strain gauge is the spot-weldable vibrating wire strain gauge, which may be used in lieu of the strain gauges with sister bars. Other types of strain gauges, such as optical grating/optical fiber strain gauges, and resistance strain gauges, may also be used in soil nail applications.

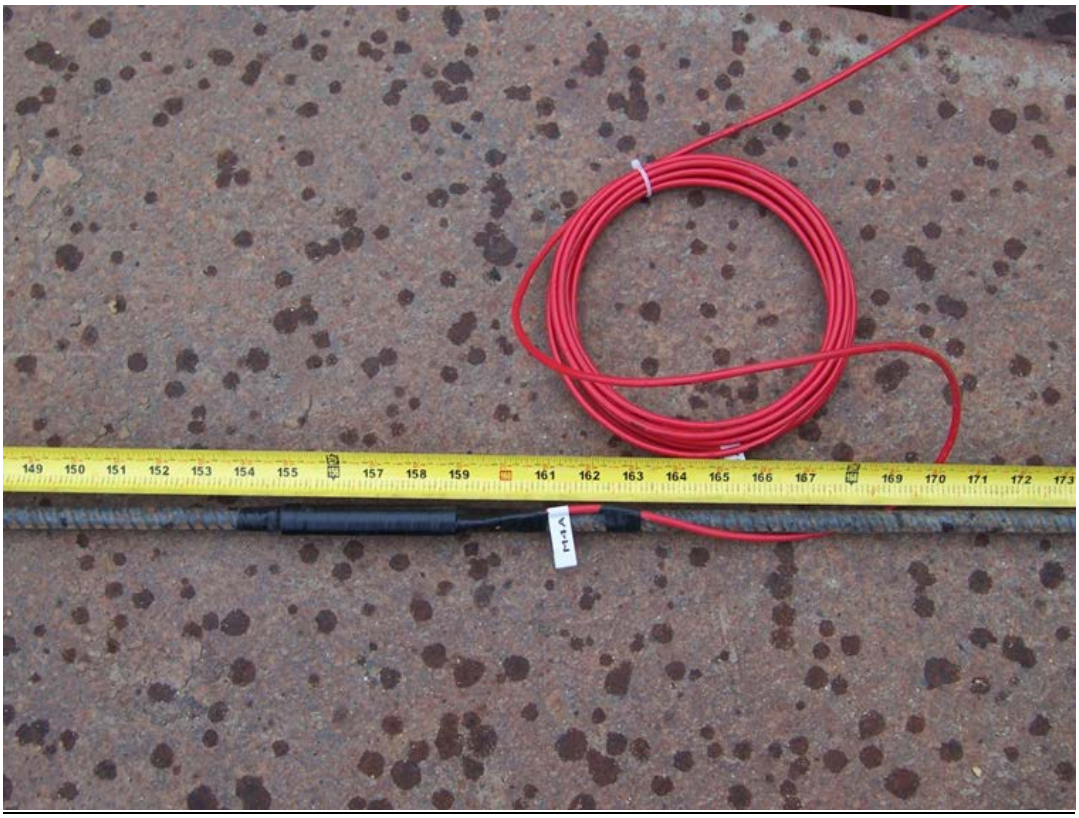


Figure 9.8: Photo. Strain gauge attached to a sister bar. Photograph courtesy of Schnabel Engineering.

9.5.4d Load Cells at the Nail Head

Load cells installed at the soil nail head can be used to provide information on the actual loads that are developed at the facing and how these vary over time. However, care must be taken when interpreting results because it is known that load cells occasionally provide inaccurate readings.

Load cells typically consist of Wheatstone-resistance bridges attached to the wall of an element with a known modulus. Measurement of the resistance of the load cell is correlated to the strain in the element and hence the load. Other load cells are available and may present some advantages over traditional systems. These include load cells using vibrating wire and other measurement systems.

Chapter 10: Considerations for Hollow Bar Soil Nails

10.1 Introduction

HBSNs have been increasing in popularity and usage since the technology was introduced about 20 years ago (Cadden et al. 2010). HBSNs are advanced through their full design length with a sacrificial bit using grout as the drilling fluid; thus drilling, grouting, and tendon installation are all accomplished as part of one single process. Two publications by the Central Federal Lands Highway Division provide guidance on critical aspects of HBSN: "Hollow Bar Soil Nails Pullout Test Program," Cadden et al. (2010); and "Hollow Bar Soil Nails – Review of Corrosion Factors and Mitigation Practice," Samtani and Nowatzki (2010).

HBSNs are particularly advantageous in collapsible soils where the simultaneous injection of grout through the tendon during drilling keeps the drill hole from collapsing. Due to their installation method, the procedures for HBSN installation, design, and corrosion protection require special considerations which are addressed in this chapter. Information included here is meant to supplement the preceding chapters and will not repeat introductory information on topics that are not specific to HBSNs.

10.2 Installation

Each HBSN is fitted with an oversized, sacrificial drill bit containing exit port holes for the grout. The grout is pumped from the surface through the hollow core of the tendon and exits through the drill bit as the tendon, acting as the rotary drill string, is advanced. The grout is continuously pumped during advancement, provides temporary support against drill hole instability, and prevents overheating of the drill bit. The grout return must be maintained at the face during drilling because this contributes to an adequate bond between soil and grout. The grout inside the tendon is subject to significant pressure, and exits the drill bit at relatively high velocity and low pressure, which helps to undercut the soils around the bit, and transports the soil cuttings to the ground surface. Water should not be used as drilling fluid in a first stage, not even after switching to grout in a second stage and after the drill depth is reached. Experience has shown that the use of water as drilling fluid can cause erratic and poor bond strengths.

Grout for HBSNs must have a specific gravity ranging between 1.8 and 1.9. The minimum compressive strength for grout should be 1,500 psi at 3 days, and 3,000 to 4,000 psi at 28 days, and should meet the requirements of AASHTO T106/ASTM C109. The sample for testing must be taken from the grout returning from the drill hole. When working with longer nails, a lighter drilling grout with specific gravity of 1.4 to 1.6 is often used for

economy to advance the bar and carry spoils to the surface. After the tendon is installed to the desired depth, a heavier final grout mixture is pumped through the hollow bar and the nail is considered complete when the heavier mixture returns to the excavation face, signaling that the lighter drilling grout was flushed from the drill hole and that all drilling spoils have also been removed.

10.3 Bond Strength

Thirty seven HBSNs were installed in four different sites across the U.S. and tested as part of an investigation co-sponsored by FHWA and the International Association of Foundation Drilling (ADSC) (Cadden et al. 2010). The goals of this investigation were to establish appropriate de-bonding procedures for proof-testing HBSNs and to create a database of bond strengths in different ground conditions. Gravity-grouted, solid bar soil nails were also installed and tested for comparison at each of the sites. The results of the tests are summarized in Table 10.1. The bond strength values provided in the table were calculated using the nominal drill bit diameter, not the actual grout body diameter of the HBSNs. Some of the tests did not reach failure; therefore, the range of resistance values for the HBSNs may be somewhat conservative. Note that presumptive values of bond strength, for pressure-grouted micropiles (Sabatini et al. 2005), are provided for similar soil types as a comparison.

Table 10.1: Nominal Bond Strength of Soil Nails in Granular Soils (after Cadden et al. 2010)

Soil Type	Gravity-Grouted Solid Bar Soil Nail (psi)	Hollow Bar Soil Nail (psi)	Pressure-grouted Micropiles (Sabatini et al. 2005) (psi)
Silty Sand (SM)	12.8-17.1	18.7-24.4	Sand (some silt) 10-27.5
Poorly Graded Sand (SP)	13.2	71.2-79.6	Sand (some Gravel) 17.5-52
SP with Gravel	50.5-82.2	-	Sand (some Gravel) 17.5-52
Poorly Graded Gravel (GP) with Sand	53.2-64.5	62.4-156.3	Gravel (some Sand) 17.5-52

Note: Bond strength values from tests are calculated using the nominal drill bit diameter.

The tests showed that HBSNs tend to develop larger bond strength values in granular soils than traditional, gravity-grouted solid bar nails. This is due to the larger drill hole diameter,

and thereby grout body that is created by undercutting and permeation of the soils by the grout, and to more pronounced roughness features in the grout-soil interface. Consequently, the presumptive resistance values given for gravity grouted nails in Table 10.1 are likely very conservative for HBSNs in most cases. The bond strength is more sensitive to craftsmanship and details of the installation techniques used by each Contractor for HBSNs than for solid bar soil nails.

The use of presumptive bond strength values for pressure-grouted micropiles given by Sabatini et al. (2005) may be appropriate for hollow bar soil nails installed in SP and GP soils as evidenced in Table 10.1. In granular soils with a significant fines content (SM), the calculated average bond strength values for HBSNs were similar to those for gravity-grouted nails. This suggests that undercutting and permeation of these soils by the grout are not as significant as in more predominantly granular soils. In these soils, as well as in predominantly fine-grained soils, the use of the presumptive values in Table 10.1 may be appropriate for preliminary design.

10.4 Corrosion Protection of HBSNs

10.4.1 Introduction

One of the most controversial topics related to HBSNs is the effectiveness of corrosion protection measures. This is due to the abrasion and friction that the tendon is subject to during installation. Samtani and Nowatzki (2010) evaluated various parameters related to corrosion of HBSNs, summarized the practice of protecting HBSNs against corrosion at the time, and presented corrosion guidance. Under the same subsurface conditions, HBSNs have higher corrosion risks than solid bars because the methods used to install HBSNs tend to deteriorate the protection furnished to these bars. Damage to epoxy and zinc coatings is caused by contact with the soil mass into which the HBSN is being installed, by erosion from the outgoing soil cuttings, and by impact with centralizers during installation. Therefore, the effective service life of coatings may be severely shortened.

Fishman and Withiam (2011) evaluated existing computational models for predicting metal loss in soil reinforcement used in retaining structures (mainly MSE walls), developed a methodology based on improved models to predict the performance of metal-reinforced systems over their service life, and suggested changes to the current AASHTO LRFD specifications to incorporate these models and methodology. In addition, they introduced modified LRFD resistance factors to account for corrosion exposure. Although this document focused on corrosion of MSE walls, the numerical models of Fishman and Withiam (2011) can be used to estimate the time-rate of metal loss, and assess future performance of HBSNs.

Because of the damage that is inflicted on the bar coatings during installation, it cannot be routinely demonstrated that any of the bar coatings described in Chapter 7 can reliably protect the tendons. Therefore, the only corrosion protection method currently available for HBSNs is the use of a bare bar with sacrificial steel. These issues will be discussed further in this chapter.

10.4.2 Sacrificial Steel

In the absence of thorough studies specific to corrosion of soil nails, this manual uses the metal loss models developed by Fishman and Withiam (2011) as the basis for evaluation of sacrificial steel thickness of HBSNs. The focus of the work by Fishman and Withiam (2011) was MSE walls, where the aggressiveness of the fill can be controlled by setting acceptance criteria for fill quality. They developed a metal loss model based on extensive studies on the rate of corrosion of uncoated mild steel wire or steel strips buried in different types of soils in reinforced zones of MSE walls. Fishman and Withiam (2011) indicated that their study was intended to be inclusive of passive reinforcement elements embedded in the ground. Post-tensioned elements were also evaluated separately in their study. The study included a review of the variability of the steel mechanical properties, and of the applicability and accuracy of the metal loss models. They expressed their findings in terms of reduction resistance factors for tension to account for the variability of material properties and the accuracy of the metal loss models.

As presented in Chapter 7, sacrificial steel is computed using the following equation for plain steel elements (Fishman and Withiam 2011):

$$X(\mu m) = 80\ (\mu m/yr) \times t_f^{0.8}$$

Equation 7.1: Thickness of sacrificial steel.

Where X is loss of steel in μm, and t_f is service life in years. Steel loss must be considered as taken twice from the diameter of the bar:

$$d_{loss} = d_{bar} - 2X$$

Equation 7.2: Remaining bar diameter.

Where d_{loss} is the bar diameter after sacrificial steel is considered and d_{bar} is the nominal bar diameter.

These equations can be used to estimate the diameter reduction for the design life of the structure, in the absence of product-specific data. The diameter reduction is used to calculate

the steel area for bar design in Chapter 6. The resistance factors for steel in tension in Chapters 5 and 6 should be used for calculation of the tendon tensile resistance.

10.4.3 Design Steps for Corrosion Protection of HBSNs

The selection of soil nail corrosion protection for HBSNs is performed by using the following steps.

(1) Select an HBSN trial size.

(2) Use Table 7.2 to classify soil as non-aggressive or aggressive, based on results of corrosion testing as described in Chapters 4 and 7. If the soil is classified as aggressive, the use of HBSNs is not recommended.

(3) If the soil is classified as non-aggressive, Class C protection (sacrificial steel) may be used.

(4) Select Class C:
   a. Obtain the nominal tendon diameter
   b. Select the design life of the soil nail wall (typically 50 to 75 years)
   c. Calculate sacrificial steel using Eq. 7-1
   d. Calculate the new diameter with steel loss
   e. Use the new diameter to calculate the area of the tendon with steel loss
   f. The area with steel loss should then be used as the cross-sectional area of the tendon, A_t, in Eq. 6-5 to calculate the nominal resistance of the tendon
   g. Proceed with the design according to Chapter 6

(5) Calculate the factored resistance of the trial HBSN by applying the resistance factor to the nominal tensile resistance (see Chapters 5 and 6) calculated using the reduced steel area.

(6) Compare the nominal resistance to the tensile forces calculated from a computer-based design software (see Chapter 6) and repeat process as needed.

10.5 Corrosion Protection Monitoring

Considering that HBSNs often provide significant cost savings over solid bars, Owners should consider implementing corrosion monitoring programs to evaluate the long-term performance of HBSNs, obtain specific data that could support future developments in corrosion protection, and hence eliminate any excessive conservatism that may exist in the formulation presented in this manual.

The main goals of corrosion monitoring programs are to: (i) assess corrosion rates over relatively extended periods of time (years to decades); (ii) confirm or rectify assumptions made during design in relation to metal loss rates; (iii) investigate the performance of metallic reinforcement under environmental conditions that possibly change over time (e.g., higher corrosion potential due to migration of corrosive substances); and (iv) furnish data to assess the integrity of permanent structures built with steel reinforcements.

Monitoring can also help detect undesired performance and prevent impending or near-term failures in cases where the estimates of service life were too optimistic. On the other hand, corrosion monitoring can help establish some evidence that a structure can extend its service life. This information can be very valuable to Owners in managing the infrastructure assets and prioritizing maintenance, rehabilitation, retrofit, or replacement (Elias et al. 2009).

Monitoring programs are uncommon in solid bar soil nail projects but are becoming more prevalent with other systems using metallic reinforcement, such as MSE walls, and can be possibly extended to both hollow and solid bar soil nail walls. Elias et al. (2009) present a complete description of corrosion monitoring of metallic reinforcement as applied to MSE walls, and discuss monitoring programs developed by several state Departments of Transportation (DOTs) across the U.S.

The two main testing techniques involve: (i) assessment of the state of coupons that had been buried strategically behind the nail-supported wall during construction for monitoring purposes; and (ii) remote electrochemical testing methods. The retrieval of coupons is destructive and requires accessing and extracting samples through the wall facing each time. Accessing coupons placed behind the concrete/shotcrete facing of a soil nail wall might be challenging. Special construction detailing must be developed during the planning phase of the monitoring to allow easy access with lockable openings. The retrieval of samples has limitations in that the number of measurements is restricted to the number of buried coupons. Testing might not be totally representative because the corrosion at the coupon is localized and might not reflect the variability of corrosion along the entire tendon.

Remote electrochemical methods include measurements of potential and polarization resistance (PR) to indirectly determine instantaneous, average, in situ corrosion rates of reinforcement elements in service. This method is nondestructive and permits measurements to be taken at any time. Closer monitoring is possible with PR testing. The use of remote electrochemical methods is recommended on at least some coupons buried for retrieval to confirm results and help calibrate the measurements.

Buried coupons should: (i) be placed in locations where corrosion rates are expected to be faster; (ii) also be placed under “normal” conditions to establish a benchmark for future

reference; and (iii) ideally include different types of tendons (carbon steel, zinc, and galvanized steel) to allow for better comparison.

The number of monitoring locations and the frequency of monitoring must be carefully planned. Each project might present unique conditions that help dictate the monitoring locations and frequency. The number of monitoring locations must be a function of the total wall length so that the anticipated spatial variability of site conditions can be evaluated. For MSE walls, Elias et al. (2009) recommend test locations spaced 200 ft maximum. Since soil nail walls are built in natural soils having a wider variability of conditions, a smaller spacing should be specified for soil nail walls. An increased density of coupons is also recommended in areas where higher corrosion potential is expected. Testing should be performed at different depths (a minimum of two is recommended for MSE walls) and at an increasing distance from the wall face (a minimum of two locations is also recommended for MSE walls).

Monitoring should take place at the time of installation to establish initial values, then monthly for the first 3 months, bi-monthly for the next 9 months, and annually thereafter. Ideally, monitoring should extend over a reasonably long period of time that would allow the observation of: (i) an initial, slow corrosion rate resulting from the protection of coatings; (ii) the start of a transition period when the coating is being depleted; and (iii) finally the initiation of faster, long-term corrosion of the basal carbon steel.

Ultimately, results of monitoring efforts in soil nail walls will help refine the values of resistance factors and the metal loss models for HBSNs. They will also help in assessing the effectiveness of protective coatings in preventing or delaying corrosion of soil nails.

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Glossary

Bond stress

Shear stresses mobilized along the grout-ground interface.

Corrosion potential

The tendency of a soil to produce corrosion to metallic elements that are in contact with it. Corrosion potential is assessed by field testing and/or by laboratory testing of soil samples obtained in field explorations. Corrosion potential is evaluated based on: (a) pH (potential of hydrogen); (b) electrical resistivity; (c) chloride content; (d) sulfate content; and (e) organic content.

Corrosion protection

The means provided for protecting soil nail bars from corrosion. These can include physical or chemical protection, or a combination thereof. Physical protection involves the use of a continuous barrier between the nail bar, other metallic parts, and the corrosion sources. Chemical protection consists of the use of a sacrificial material (e.g., galvanized material) or a dielectric material (e.g., epoxy coated bars), which will impede the flow of electric current. Some of the corrosion protection systems currently in use utilize a combination of these mechanisms.

Drainage

In the context of soil nail walls, drainage mainly involves vertical geocomposite strip drains, which are installed behind the initial facing and adjacent to the excavation face. The objective of the strip drains is to limit the development of water pressures behind the wall facing.

Facing strength limit states

A condition achieved by the initial or final facing of a soil nail wall due to bending, punching shear, or tension of the headed studs.

Final facing

The final facing is constructed over the temporary facing and provides structural continuity throughout the design life. It may include a visually pleasing finish. Final facing may consist of cast-in-place reinforced concrete, reinforced shotcrete, and precast concrete panels.

Grout

Cementitious material is used to cover the bars in the drill holes, where soil nails are installed. Grout usually consists of a Portland cement and water. The grout transfers stresses between the ground and the tendons and provides some level of corrosion protection to the bars.

Hollow bar soil nail (HBSN)

Hollow core and self-drilling bars that are advanced with grout being injected through the bar. The grout also serves as the drilling medium. The grout, which is applied under pressure, exits at ports at the drill bit and at significant velocity to help remove the soil undercutting.

Initial facing

Initial facing is applied to each excavation lift on the exposed face soon after excavation to provide temporary stability and protection. The initial facing receives the bearing plate and consists of reinforced shotcrete.

Launched soil nail

Bare bars installed into the soil using a firing mechanism involving compressed air, which can launch the bar at high speeds approaching 200 miles per hour. No corrosion protection is provided. Used for temporary slope stabilization.

Limit state

A condition in which a structure as a whole, or one of its components, has achieved a level of stress, deformation, or displacement that may affect its performance.

Load proof tests

Load tests conducted during construction on a minimum of 5 percent of the total production nails to verify that no significant variations exist in soil nail performance throughout the wall.

Load verification tests

Load tests conducted before or during construction to verify the compliance of the pullout capacity resulting from the Contractor's installation methods with those values of bond strengths used in design.

Overall stability

Limit state involving potential slip surfaces extending through the soil and/or soil nails. If the contribution to stability comes from both soil nails and soil, the case refers to internal stability. If the contribution to stability comes from the soil only (slip surfaces not intersecting soil nails), the case refers to general global stability. Particular cases of global stability may arise, including basal heave and sliding stability, for special subsurface conditions.

Pullout resistance

The axial resistance developed at a soil nail by the mobilization of bond (shear) stresses along its grout-soil interface.

Screw-in nail

A helical soil nail that can stabilize retained soils. These soil nails are installed using drilling equipment with sufficient torque output, and without removal of the native soils. These nails typically consist of 1.5-inch square solid steel shafts on which steel helices are welded at regular intervals.

Service limit state

A condition achieved by a structural element in which no limit stress is achieved but other performance aspects are affected. Service limit states for soil nail walls involve lateral displacement and settlement of the wall.

Shotcrete

Pneumatically fresh concrete/mortar that is projected fresh through a hose at high velocity onto the excavated face or the initial facing of a soil nail.

Soil nail

Reinforcing, passive element drilled and grouted sub-horizontally in the ground to support excavations in soil and intermediate geomaterials. Soil nails are considered reinforcing elements that: (a) contribute to the stability of earth-resisting systems mainly through tension as a result of the deformation of the retained soil or weathered rock mass; (b) transfer tensile loads to the surrounding ground through shear stresses (i.e., bond stresses) along the grout-ground interface; (c) develop resistances that can be estimated with established design procedures; (d) have long-term, demonstrable corrosion protection to ensure adequate, long-term performance of the system; (e) interact structurally with the facing of the excavation; (f) are load-tested according to

prescribed methods; and (g) are routinely subject to construction QC/QA according to established procedures.

Solid bar soil nail (SBSN)

Commonly used bars in soil nails walls. Solid bars are placed in drill holes in a first operation and grouted under gravity in a second stage.

Strength limit state

Limit states related to strength or nominal resistances of the structure or any of its components. Strength limit states in soil nail walls include geotechnical limit states (nail pullout and, for some subsurface conditions, lateral sliding stability), structural limit states (soil nail in tension, facing in bending, facing in punching shear, and final facing headed stud in tension).

Tendon or bar

The reinforcing component of a soil nail.

Tensile limit state

A condition achieved by the tendon of a soil nail, the reinforcement in the facing, or the headed studs in the soil nail head.

Appendix A: Reinforcement Characteristics

Note: Products are occasionally discontinued or modified by manufacturers. Some properties/sizes originally considered in a design might not be available for construction. Check availability with manufacturer. In cases where the sizes originally planned were discontinued, adopt the next available larger size.

The following tables are included in this appendix:

Table A.1a: Properties of Solid-Threaded Bars Grade 60

Table A.1b: Properties of Solid-Threaded Bars Grade 75

Table A.1c: Properties of Solid-Threaded Bars Grade 150

Table A.2: Properties of Hollow Bars CTS/Titan Bar Type

Table A.3: Properties of Hollow Bars MAI Bar Type

Table A.4: Properties of Hollow Bars GEO-Drill Bar Type

Table A.5: Welded-Wire Mesh

Table A.6: Reinforcing Bar

Table A.7: Headed-Stud

Table A.1a: Properties of Solid-Threaded Bars - Grade 60

Bar Designation	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Yield Stress	Yield Load
Conventional	inch	$inch^2$	lb/ft	Conventional	ksi	kip
#6	0.86	0.44	1.50	60	60	26
#7	0.99	0.60	2.04	60	60	36
#8	1.12	0.79	2.67	60	60	47
#9	1.26	1.00	3.40	60	60	60
#10	1.43	1.27	4.30	60	60	76
#11	1.61	1.56	5.31	60	60	93
#14	1.86	2.25	7.65	60	60	135

Table A.1b: Properties of Solid-Threaded Bars - Grade 75

Bar Designation	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Yield Stress	Yield Load
Conventional	inch	$inch^2$	lb/ft	Conventional	ksi	kip
#6	0.86	0.44	1.50	75	75	33
#7	0.99	0.60	2.04	75	75	45
#8	1.12	0.79	2.67	75	75	59
#9	1.26	1.00	3.40	75	75	75
#10	1.43	1.27	4.30	75	75	95
#11	1.61	1.56	5.31	75	75	117
#14	1.86	2.25	7.65	75	75	168

Sources: Dywidag, Williams and Contech

Table A.1c: Properties of Solid-Threaded Bars - Grade 150

Bar Designation (inch)	Maximum Diameter (w/ threads)	Minimum Cross-Sectional Area	Unit Weight	ASTM Grade	Ultimate Load (1)
Conventional	inch	$inch^2$	lb/ft	Conventional	kip
¾	0.75	0.37	1.32	150	57
1	1.00	0.85	3.01	150	128
1¼	1.25	1.25	4.39	150	188
1⅜	1.375	1.58	5.56	150	237
1¾	1.75	2.58	9.22	150	400

Sources: Dywidag, Williams and Stressteel

Note: (1) Based on ultimate strength.

Table A.2: Properties of Hollow Bars

CTS/Titan Bar Type

Bar Designation	Effective Outer Diameter	Cross-Sectional Area	Unit Weight	Yield Load (1)	Ultimate Load
Conventional	**inch**	**inch^2**	**lb/ft**	**kip**	**kip**
30/16	1.02	0.53	1.8	42.7	55.1
30/14	1.03	0.58	1.9	49.5	61.8
30/11	1.03	0.64	2.2	58.5	72.0
40/20	1.42	1.13	3.8	95.6	121.4
40/16	1.42	1.40	4.8	118.1	148.4
52/26	1.92	1.94	6.7	164.2	208.0
73/56	2.76	2.11	7.3	186.6	232.7
73/53	2.76	2.50	8.9	218.1	260.9
73/45	2.76	3.50	12.0	285.6	356.4
73/35	2.76	4.20	14.2	321.6	419.4

Source: Contech

Note: (1) These bars are alloy steel and have nominal yield strength of 87 ksi.

Table A.3: Properties of Hollow Bars

MAI Bar Type

Bar Designation	Outer Diameter	Cross-Sectional Area	Weight	Average Yield Stress (1)	Average Ultimate Tensile Stress (1)	Yield Load	Ultimate Load
Conventional	**inch**	**inch^2**	**lb/ft**	**ksi**	**ksi**	**kip**	**kip**
R25N	1.00	0.41	1.41	83	111	34	45
R32N	1.26	0.56	1.88	92	112	52	63
R32S	1.26	0.72	2.49	87	112	63	81
R38N	1.50	1.00	3.43	89	112	90	112
R51L	1.50	1.17	3.97	86	106	101	124
T40N	1.57	1.32	4.50	89	112	118	148
R51N	2.00	1.61	5.44	88	112	142	180
T76N	3.00	3.32	11.29	81	108	270	360
T76S	3.00	3.88	13.24	87	110	337	427

Source: Dywidag

Note: (1) The yield strength and ultimate tensile stresses of these bars vary as indicated above, per manufacturer's information.

Table A.4: Properties of Hollow Bars

GEO-Drill Bar Type

Bar Designation	Nominal Outer Diameter	Cross-Sectional Area	Weight	Yield Load (1)	Ultimate Load (1)
Conventional	**inch**	**inch^2**	**lb/ft**	**kip**	**kip**
B7X1-32	1.25	0.556	2.10	47.2	58.4
B7X1-32X	1.25	0.776	2.70	66.0	81.5
B7X1-38	1.50	1.067	3.76	90.7	112.0
B7X1-51	2.00	1.795	6.26	152.0	188.0
B7X1-76	3.00	3.880	13.79	329.0	407.0

Source: Williams

Note: (1) These bars have a yield strength of 85 ksi and an ultimate strength of 105 ksi.

Table A.5: Welded-Wire Mesh [1]

Mesh Designation[2] [3] [4]	Cross-Sectional Area per Unit Width [5]	Weight per Unit Area
in. × in. – $in.^2/100$ × $in.^2/100$	$in.^2/ft$	lb/cft^2 [6]
4x4 - W 1.4 x W 1.4	0.042	31
4x4 – W 2.0 x W 2.0	0.060	44
4x4 – W 2.9 x W 2.9	0.087	62
4x4 – W 4.0 x W 4.0	0.120	88
6x6 – W 1.4 x W 1.4	0.028	21
6x6 – W 2.0 x W 2.0	0.040	30
6x6 – W 2.9 x W 2.9	0.058	42
6x6 – W 4.0 x W 4.0	0.080	58

Source: WRI (2006)

Notes:
(1) These properties are applicable for WWM in Grades 60 or 65. Consult WRI (2006) for WWM properties for Grades 70, 75, and 80.
(2) The first pair of numbers (e.g., 4x4) indicate the mesh opening size (or wire spacing) in the longitudinal and transverse direction, respectively.
(3) Prefix "W" indicates plain wire. A prefix "D" would indicate pre-deformed wire.
(4) The second pair of numbers (e.g., 1.4x1.4) indicates the cross-sectional area of wires (in square inches x 100) in the longitudinal and transverse direction, respectively.
(5) The area per unit width is obtained by dividing the wire cross-sectional area by 100 and by the mesh opening size (or wire spacing, in feet). If the longitudinal and transverse spacings and wire sizes are different, the cross-sectional area per unit width will be different in each direction. Consult WRI (2006) for additional information.
(6) CSF = 100 square feet.

Table A.6: Reinforcing Bars

Bar Designation	Nominal Diameter	Nominal Area
#	inch	$inch^2$
3	0.375	0.11
4	0.500	0.20
5	0.625	0.31
6	0.750	0.44
7	0.875	0.60
8	1.000	0.79
9	1.128	1.00
10	1.270	1.27
11	1.410	1.56
14	1.693	2.25
18	2.257	4.00

Source: ACI 318-11

Table A.7: Headed-Studs

Headed Stud Size	Nominal Length (1) (2)	Head Diameter	Shaft Diameter	Head Thickness	Head Area / Shaft Area	Head Thickness / (Head Diameter- Shaft Diameter)
-	L_S	D_{SH}	D_{SC}	t_{SH}	A_{SH} / A_{SC}	$t_{SH} / (D_{SH} - D_{SC})$
-	inch	inch	inch	inch	-	-
¼ x 4⅛	4⅛	0.50	0.25	0.19	4.0	0.75
⅜ x 4⅛	4⅛	0.75	0.38	0.28	4.0	0.75
⅜ x 6⅛	6⅛	0.75	0.38	0.28	4.0	0.75
½ x 4⅛	4⅛	1.00	0.50	0.28	4.0	0.56
½ x $5^5/_{16}$	$5^5/_{16}$	1.00	0.50	0.28	4.0	0.56
½ x 6⅛	6⅛	1.00	0.50	0.28	4.0	0.56
⅝ x $6^9/_{16}$	$6^9/_{16}$	1.25	0.63	0.31	4.0	0.50
¾ x $3^{11}/_{16}$	$3^{11}/_{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $4^3/_{16}$	$4^3/_{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $5^3/_{16}$	$5^3/_{16}$	1.25	0.75	0.38	2.8	0.75
¾ x $6^3/_{16}$	$6^3/_{16}$	1.25	0.75	0.38	2.8	0.75
⅞ x $4^3/_{16}$	$4^3/_{16}$	1.40	0.86	0.38	2.5	0.73
⅞ x $5^3/_{16}$	$5^3/_{16}$	1.40	0.86	0.38	2.5	0.73
⅞ x $6^3/_{16}$	$6^3/_{16}$	1.40	0.86	0.38	2.5	0.73

Source: Stud Welding Associates, Inc.

Notes: (1) Nominal length indicated is before welding.
(2) After-welding lengths to be considered as follows: For $D_S \leq$ ½", L_S is approximately ⅛" shorter after welding. For $D_S >$ ⅝"L_S is approximately 3/16" shorter after welding.

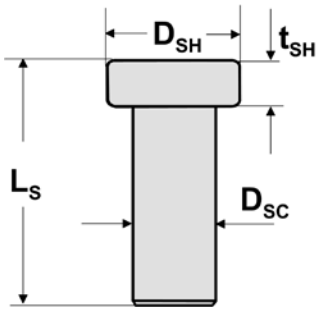


Figure A.1: Illustration. Geometry of headed studs.

Appendix B: Charts for Preliminary Design

Design charts were developed using the computer program SNAIL to help estimate lengths and maximum tensile forces of soil nails for typical conditions in design. The charts were developed for ranges of the angle of backslope (β) and the effective friction angle (φ_s) of the retained soils. The bond strength and other parameters need to be selected before using the charts. The charts are expected to provide reasonable estimates for the conditions and assumptions indicated. However, the charts should not replace the use of soil nail design computer programs for more complex conditions and for other drastically different conditions.

The following assumptions were made:

- face batter, $\alpha = 0$
- nail inclination, i = 15
- normalized cohesion, $c^* = c'/\gamma_s H = 0.02$
- factor of safety for overall stability, $FS_{OS} = 1.35$
- drill hole diameter, $D_{DH} = 4$ in.

1. Select the following parameters (Figure B.1) using consistent units before using the charts:

   - wall height (H)
   - angle of backslope (β)
   - vertical and horizontal spacing of soil nails (S_V and S_H)

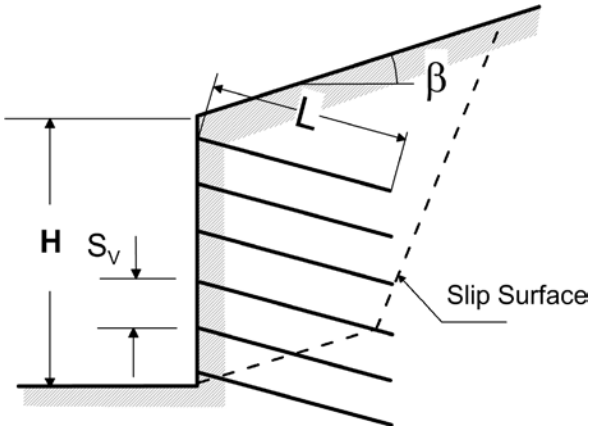


Figure B.1: Illustration. Conditions for calculations with charts.

2. Select the following properties and parameters for the retained soil using consistent units:

- unit weight (γ_s)
- effective friction angle (φ_s)
- cohesion, (c)
- bond strength (q_U)
- Factor of safety for pullout (FS_{PO}, usually 2.0).

3. Select the appropriate chart corresponding to β = 0, 10, or 30 degrees (Figures B.2 through B.4).

4. Calculate the normalized bond resistance (μ) as defined below and enter μ in the horizontal axis of the chart for both parts (a) and (b).

$$\mu = \frac{q_U \, D_{DH}}{FS_{PO} \, \gamma_s \, S_H \, S_V}$$

Equation B.1: Normalized bond resistance.

5. Select the appropriate curve for φ_s. Interpolate between φ_s = 27, 31, 35 or 39 degrees as needed.

6. Obtain the normalized soil nail length (L/H) required to achieve FS_{OS} = 1.35 from part (a), and the normalized maximum tensile force (t_{max}) from part (b). Interpolate between Figures B.2 through B.4 if needed. Note that only sufficiently high values of φ_s will achieve the selected FS_{OS} for relatively large values of β.

7. Calculate the soil nail length (L) by multiplying the normalized soil nail length (L/H) read from the chart by H (in feet).

8. Calculate the soil nail maximum force (T_{max}) by multiplying t_{max} read from the chart by γ_s, H, S_V and S_H.

9. If necessary, correct L/H for values of FS_{OS}, c*, and D_{DH} (in inches) other than those assumed.

$$\frac{L}{H} \text{ (corrected)} = C_{1L} \times C_{2L} \times C_{3L} \times \frac{L}{H} \text{ (from charts)}$$

Equation B.2: Corrected normalized length.

Where:

$C_{1L} = 1.50 - 0.15 \, D_{DH} + 0.0065 \, D_{DH}^2$

Equation B.3: Length correction factor for drill hole diameter.

C_{2L}= -4.0 c* + 1.09 $\geq$ 0.85

Equation B.4: Length correction correction factor for cohesion.

C_{3L}= 0.52 FS_{OS} + 0.30 $\geq$ 1.0

Equation B.5: Length correction factor for factor of safety.

10. If necessary, correct t_{max} for values of FS_{OS}, c*, and D_{DH} (in inches) other than those assumed.

$$t_{max}\ (\text{corrected}) = C_{1F} \times C_{2F} \times t_{max}\ (\text{from charts})$$

Equation B.6: Corrected normalized maximum force.

C_{1F}= -0.3 + 0.4 D_{DH} -0.017 D_{DH}^2

Equation B.7: Load correction for drill hole diameter

C_{2F}= -4.0 c* + 1.09 $\geq$ 0.85

Equation B.8: Load correction for cohesion.

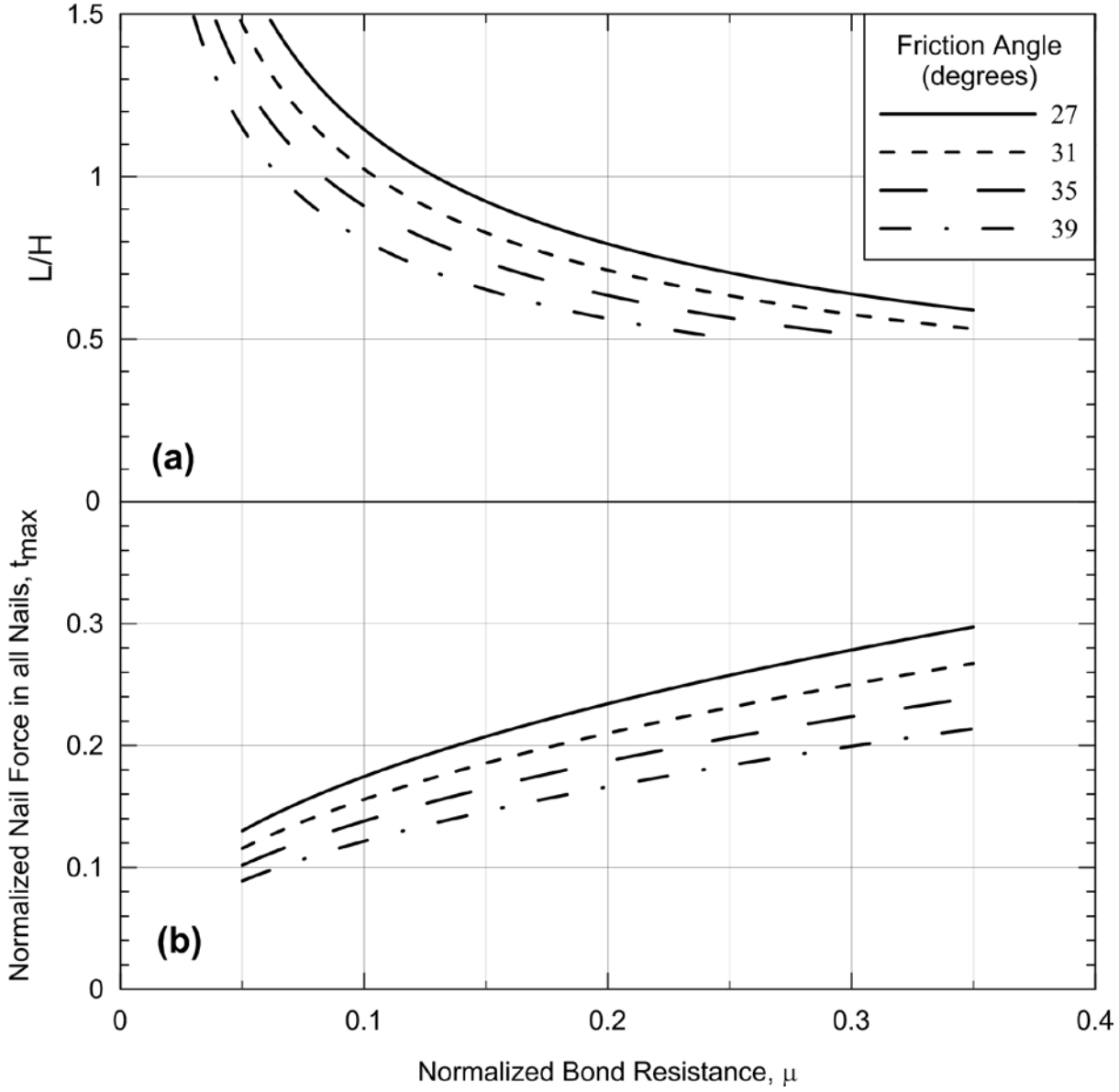


Figure B.2: Chart. Batter = 0 deg. - Backslope 0 deg.

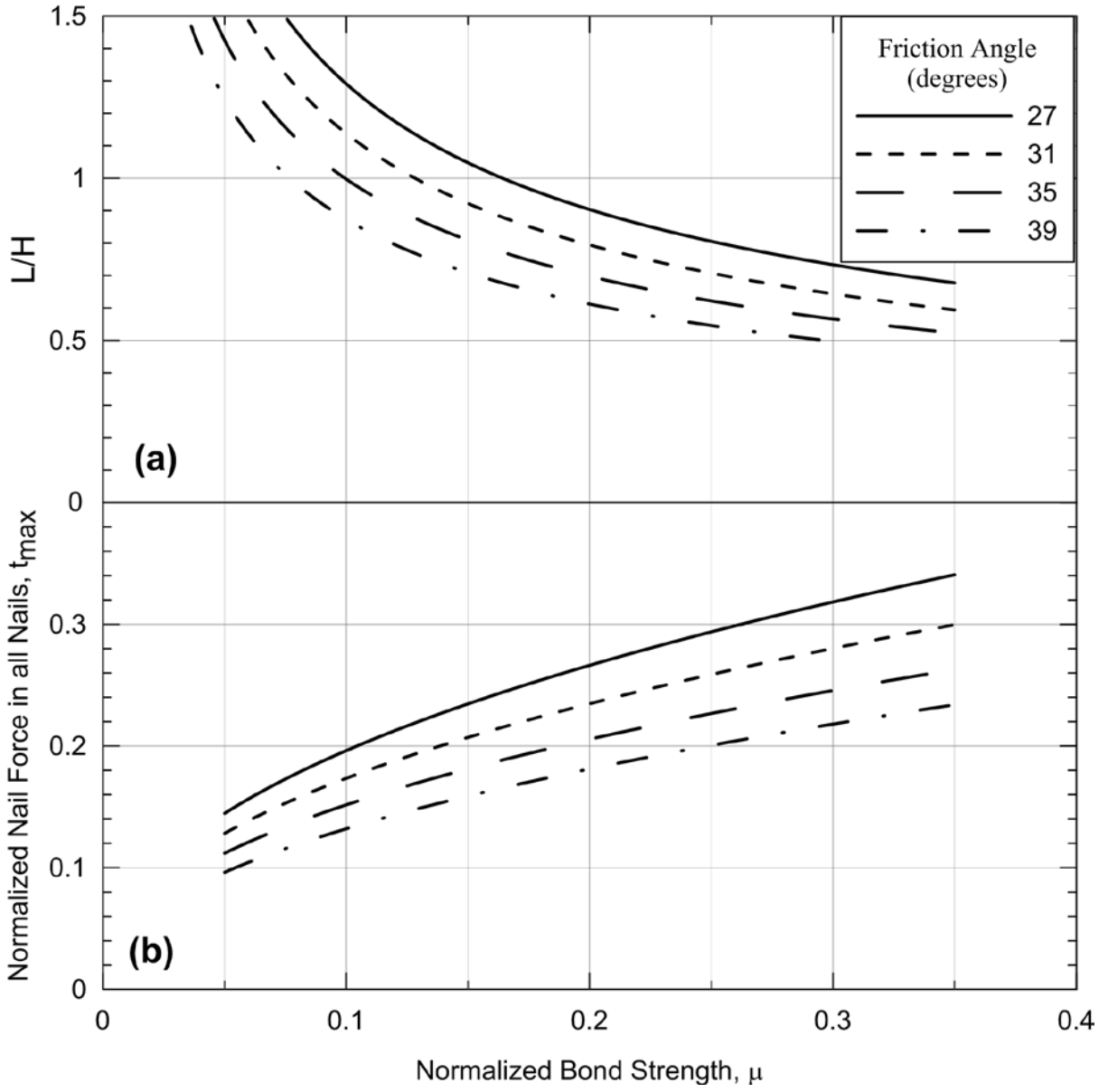


Figure B.3: Chart. Batter = 0 deg. - Backslope 10 deg.

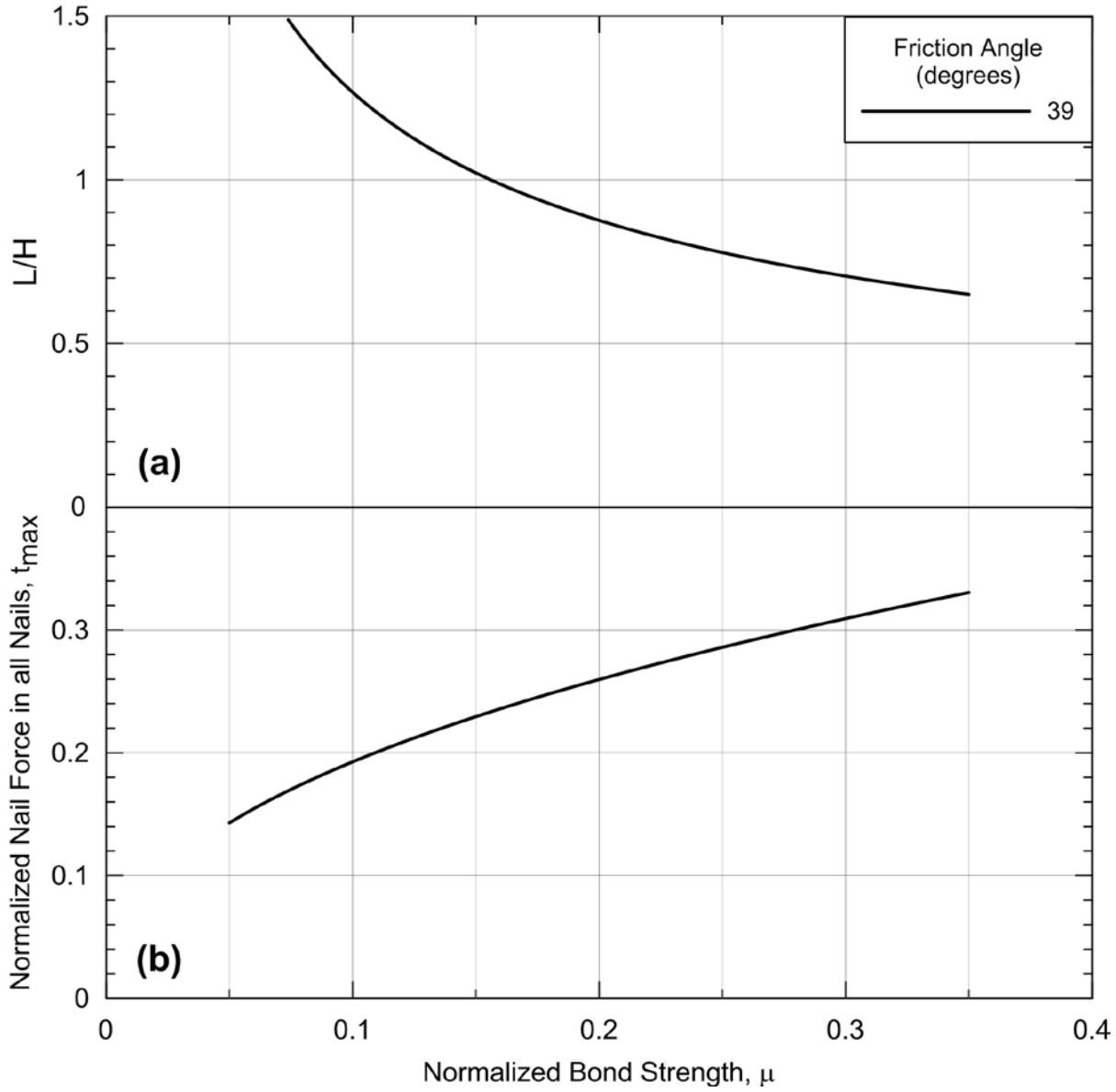


Figure B.4: Chart. Batter = 0 deg. - Backslope 30 deg.

Appendix C: Design Example

Overview

The purpose of this example is to illustrate the main steps of the design of soil nail walls by matching the steps listed in Chapter 6. Several assumptions were made to simplify the example, including: development of geotechnical parameters, selection of a critical cross-section for design, evaluation of loads, and feasibility supporting the selection of a soil nail wall as an earth-retaining system. This example omits Step 9 (seismic design), Step 10 (drainage), and Step 11 (other design considerations). The calculations in the example were obtained using the program SNAP-2 (Siel 2014). Differences between the nomenclature used in this manual and that of SNAP-2 are noted where necessary. The example also includes LRFD verifications as described in Chapter 5.

Step 1: Project Requirements

A 25-ft high soil nail wall, 200 ft in length, is planned for construction as part of a roadway project. Figure C.1 shows an idealized cross-section of the wall. The constraints and requirements of the project from the Owner require a permanent wall with cast-in-place (CIP) concrete as final facing to address aesthetic concerns. In addition, the wall is considered a critical structure. No underground utilities or permanent structures exist behind the wall. It is also assumed that standards have been selected and that the project performance requirements are similar to those recommended in this manual.

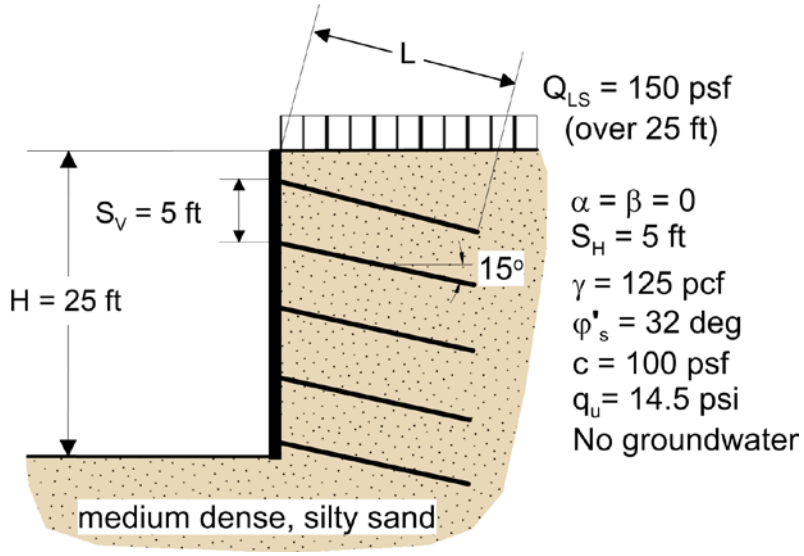


Figure C.1: Illustration. Cross-section of wall.

Step 2: Subsurface Exploration and Development of Parameters for Design

It is assumed that a geotechnical exploration and a geotechnical laboratory testing program have been completed. The geotechnical exploration revealed a uniform stratigraphy consisting of medium dense, silty sand and groundwater occurring several feet below the planned toe of the wall. The geotechnical exploration also revealed that sliding and basal heave would not occur at this site. Results from laboratory testing indicated that the corrosion potential of the in situ soils is low. It is assumed that the Owner's engineer followed the procedure included in Section 7.7 for selecting an adequate level of corrosion protection for soil nails. The minimum protection level selected for this soil nail wall would be Class C. Based on results from the geotechnical exploration and laboratory testing, the following geotechnical parameters have been selected for design:

- soil unit weight, γ_s = 125 pcf
- effective friction angle, φ_s' = 32 degrees
- cohesion, c = 100 psf
- bond strength, q_U = 14.5 psi

The soil profile developed for design is that shown in Figure C.1.

It is assumed that the wall is in a zone with very low seismic hazard; therefore, there is no need to develop seismic parameters for design.

It is assumed that the Owner conducted a risk evaluation and perceives some risks associated with long-term corrosion. As the Owner considers that the cost differential to provide a higher level of protection is marginal, the Owner prefers to specify a Class B corrosion protection, consisting of epoxy coating or galvanizing over steel bars.

Step 3: Load Definition

3.a Define Unfactored, Service Loads

The permanent load acting on the wall includes the weight of the soil behind the wall. The live loads include those loads from equipment construction operating behind and above the wall. It is assumed that the live load consists of a uniform load, Q_{LS} equal to 150 psf, extending from the wall to 25 ft behind the wall.

3.b Select Load Combinations and Load Factors

The load combinations for this example only include Strength I and Service limit states. Load factors for overall stability are 1.0 for use with SNAP-2. Load factors to be used in

LRFD verifications are presented subsequently for each case, according to Section 5.5 "Load Combinations and Load Factors."

Step 4: Soil-Nail Configuration and Material Selection

4.a/b/c/d Develop Wall Layout, Cross-Sections, Nail Pattern, and Splaying

Given the wall height (H = 25 ft) and face batter ($\alpha = 0$), the following layout is selected for trial runs in SNAP-2.

Vertical and Horizontal Spacing of Soil Nails

- Adopt $S_H = S_V = 5$ ft Check: $S_H \times S_V = 25$ ft$^2 \leq 36$ to 42 ft^2
- This vertical spacing results in five rows of soil nails.

Vertical Spacing at the Top and Bottom of the Wall

The spacing between the first row and the top of the wall (S_{V0}) is selected as:

- $S_{V0} = 3$ ft ≤ 3.5 ft ok

The spacing between the deepest row and the bottom of the wall (S_{VN}) is:

- $S_{VN} = 2$ ft ≤ 2 to 3 ft ok

Soil Nail Inclination

Because no utilities or obstructions exist behind the wall, the soil nail inclination is selected as:

- i = 15 degrees for all nails ok (between 10 and 15 degrees)

Soil Nail Length

The maximum soil nail length is selected for a trial run at:

- L = 0.7 H = 17.5 ft ok (L between 0.6 H and 1.2H)

Distribution of Soil Nail Length in Elevation

A uniform pattern of soil nail lengths is first selected for simplification. Therefore, all five soil nails have L = 17.5 ft.

However, the reader is reminded that the lowermost rows of nails should be ideally shorter than the upper rows to estimate a smaller maximum load nail T_{max}, obtain a better distribution of loads among all rows, and help reduce the wall deflection. If the soil nails are uniform in length, the lower nails develop larger loads than the upper rows.

No sensitive structures exist immediately behind the wall; hence no special considerations for controlling wall deflections are needed. If this was required, wall deformation could be partly controlled by installing longer soil nails in the upper rows (for the same safety factor), selecting tendons of larger sizes (to mobilize strength with less strains in the bars), or even by using ground anchors.

Soil Nail Pattern on Wall Face

Both "square" and "staggered" soil nail patterns are considered feasible. A staggered pattern would tend to result in smaller effective nail spacings, and therefore fewer nails than the square pattern.

Soil Nail Splaying

Special considerations for the layout of soil nails in plan are not required because wall corners and obstructions are omitted in this example.

4.e Detail Corrosion Protection

It was assumed that the Owner specified a Class B level of corrosion protection for soil nails to accommodate their tolerance for risk and because the cost differential for providing a higher corrosion protection was estimated to be low.

The Contractor should select the drill hole diameter not only for constructability and to achieve the design bond strength and soil nail pullout resistance, but also to accommodate tendons, centralizers, and couplings (if applicable), while providing the minimum grout cover required for corrosion protection.

It is assumed that the drill hole diameter will be D_{DH} = 6 in. and that the tendons will consist of #7 threaded bars. The nominal diameter for #7 bars, including threads, is 0.99 in. (see Table A.1b).

It is assumed that the corrosion protection will consist of epoxy coating, with a thickness of up to 10 mils on each side of the bar. This thickness of epoxy lies within the recommended range presented in Chapter 7. Therefore, the available space for grout cover is $0.5 \times (6.0 - 0.99) = 2.50$ in., not accounting for couplers. However, even if the trial tendon length needs to be modified if indicated by stability analysis, the expected soil nail lengths

are within the range of normally available tendon lengths; hence, couplers are not needed. Therefore, the available space is greater than the minimum cover of 1 in. recommended in Chapter 7.

4.f Select Soil Nail Type and Mechanical Properties

Solid bar soil nails are selected. The steel is selected as Grade 75, with f_y = 75 ksi.

Step 5: Selection of Resistance Factors

Safety factors selected for this example are presented in Table C.1 for ASD calculations with SNAP-2. Resistance factors for LRFD verifications are adopted from Tables 5.4-5.7 and 5.10-5.12, and are summarized in Table C.2.

Table C.1: Summary of Factors of Safety for Use with the ASD Method

Limit State	Condition	Symbol per this Manual	SNAP-2 Symbol	Minimum Recommended Factors of Safety
Overall Stability	Internal	FS_{OS}	FS	1.5
Strength – Geotechnical	Pullout Resistance	FS_{PO}	FoS_p	2.0
Strength – Structural	Tendon Tensile Strength	FS_T	FoS_y	1.8
Strength – Facing	Flexural	FS_{FF}	TF FoS	1.5
Strength – Facing	Punching Shear	FS_{FP}	TF FoS	1.5
Strength – Facing	A307 steel bolt (assumed)	FS_{FH}	HT-FoS	2.0

Table C.2: Summary of Resistance Factors for LRFD Verifications

Strength Limit State	Condition/Case	Symbol	Resistance Factor
Overall Stability	Wall is permanent and critical	ϕ_{OS}	0.65
Nail Pullout	Static	ϕ_{PO}	0.65
Nail in Tension	Grade 75	ϕ_T	0.75
Facing Flexure	Both initial and final facing	ϕ_{FF}	0.90
Facing Punching Shear	Both initial and final facing	ϕ_{FP}	0.90
Facing Headed Stud Tensile	A307 steel bolt	ϕ_{FH}	0.70

Step 6: Overall Stability

6.a Verify Internal Stability

Before going into the design sub-steps, a succinct overview of stability analysis with SNAP-2, data entry, and results is presented.

Overview of Stability Analysis with SNAP-2

A SNAP-2 user can perform a full design of a soil nail wall by selecting trial designs and repeating the analysis until strengths and capacities are verified. After the first run, the user must modify one or more of the design parameters (e.g., increase in the tendon length and/or diameter, decrease tendon spacing, thickening the facings, etc.), and conduct new trials until stability requirements are met. In addition, SNAP-2 allows the user to search for various slip circles and select one with the lowest safety factor.

Data Entry – Wall Geometry and Stratigraphy

To model the wall in SNAP-2, the user must enter the points defining the initial, intermediate, and final configuration of the grades; the top surface of each soil layer; and the location of groundwater. The location of each point is defined by the horizontal coordinate, X, and the vertical coordinate, Y. In this example, the base of the wall is at (0,0). The final configuration of the design cross-section is shown on Figure C.2.

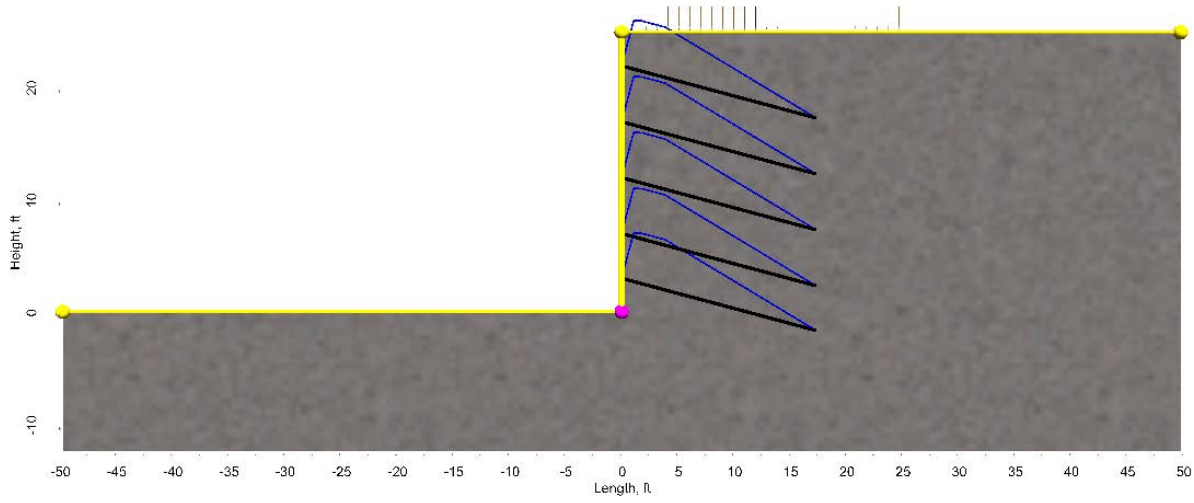


Figure C.2: Illustration Design cross section.

Data Entry – Material Properties

The user must enter properties for soil layers, as well as dimensions and material properties of the soil nails, facing components, shotcrete/concrete, welded-wire mesh, rebar, bearing plate, and headed studs. Mean values of the soil resistance should be considered.

The wall-soil interface friction is estimated to be 2/3 φ_s' in SNAP-2. This parameter, among others, is considered in SNAP-2 to perform checks for eccentricity (overturning), lateral sliding and bearing capacity. Because lateral sliding and basal heave are unlikely scenarios for this wall, these limit states are not further considered. Overturning of soil nail walls is not considered a realistic limit state in this manual.

A summary of properties of soil nail wall components is presented in Table C.3.

Table C.3: Summary of Properties of Soil Nail Components

Parameter	Main Features	Additional Descriptions
Nail features	5 solid bars, #7 Grade 75	bore hole diameter, D_{DH} = 6 in.
Facing thickness/type	initial: h_i = 4 in. final: h_f = 8 in.	initial: shotcrete f'_c = 4,000 psi final: CIP concrete f'_c = 4,000 psi
Primary reinforcement grade/type	initial: Grade 60 WWM 6×6 - W2.9×W2.9	final: Grade 60 rebar; No. 4 @ 12 in. (ea. way)
Added reinforcement in facing	initial: rebar 2 × #5 (ea. way)	final: none
Bearing plate	Square dimension: L_P = 9 in. thickness: t_P = 1 in.	Grade 50, f_y = 50 ksi
Headed stud type	4 × ($^1/_2$ × $4^1/_8$)	Grade A307 steel
Headed stud dimensions	stud length: L_S = 4.125 in. head diameter: D_H = 1 in. head thickness: t_{SH} = 0.31 in.	shaft diameter: D_{SC} = 0.5 in. stud spacing: S_{SH} = 6 in.

Other Input

Other parameters must be entered when applicable. Additional parameters may include seismic coefficients and tolerable wall deflections. Also, the excavation sequence can be defined. Attachment 1 lists the input parameters used in SNAP-2. Note that the symbols used in SNAP-2 do not generally coincide with those defined in this manual.

Results

After a few trial runs, the length of the nails was modified to L = 18 ft until the calculated factor of safety for overall stability was at or slightly above the minimum value. The output was also inspected until all the internal checks conducted by SNAP-2 indicated that the minimum requirements were exceeded.

The results of overall stability are summarized in Table C.4 for the various construction stages. Results for the scenario where the deepest nail has been installed are shown in Figure C.3.

Table C.4: Summary of Stability Analysis from SNAP-2

Excavation Stage	Calculated Safety Factor, FS_{OS}	Condition	Remarks
0	1.98	Initial	Existing sloping grades
1	1.99	Temporary	1st unsupported cut
2	2.11	Temporary	1st row installed, 2nd lift exposed
3	1.81	Temporary	2 upper rows installed, 3rd lift exposed
4	1.50	Temporary	3 upper rows installed, 4th lift exposed
5	1.24	Temporary	4 upper rows installed, 5th lift exposed
6	1.62	Final	5 rows installed, no surcharge
7	1.51	Final	5 rows installed, with surcharge

The calculated FS_{OS} is directly used in LRFD verifications. Considering that the load factor in ASD-based overall stability is $\gamma = 1.0$:

$\phi_{OS} = 1/ FS_{OS} = 1/1.51 \sim 0.60$ ok (matches value of Table C.2 for static loads)

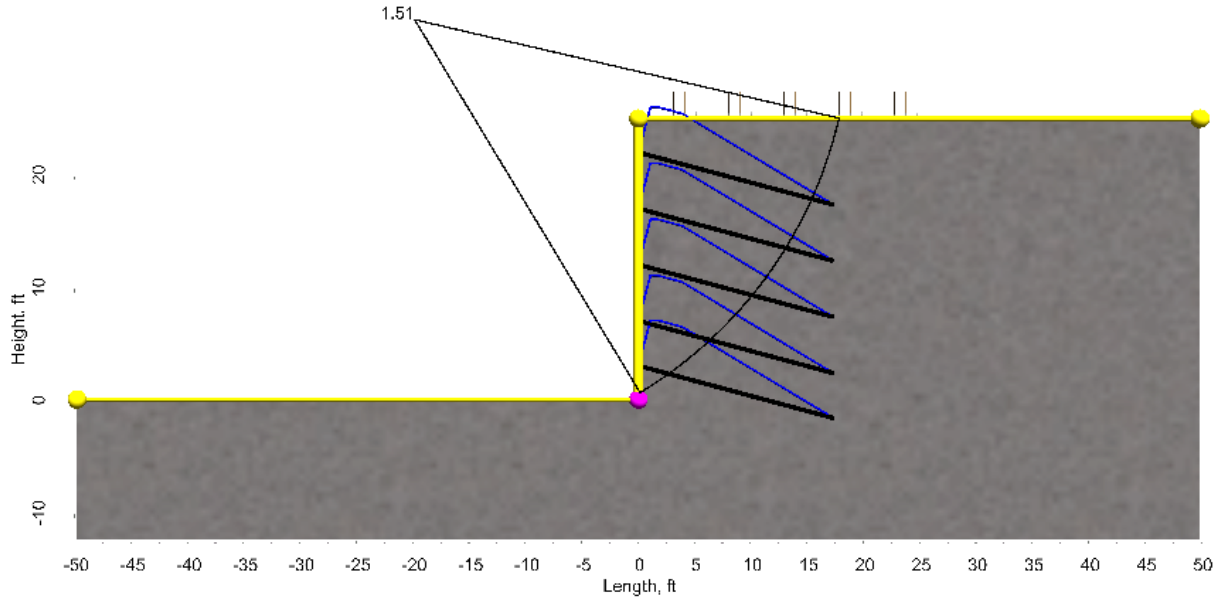


Figure C.3: Illustration. Results of overall stability.

The LRFD verification of strength limit states includes the maximum tensile force of all soil nails, T_{max}, as calculated in ASD-based stability analyses. T_{max} is limited by pullout resistance, tensile resistance, or any of the facing resistances.

T_{max}, L_P, and L_{fail} are summarized in Table C.5 for all nails for the final excavation stage. The pullout development length, L_P, which can be calculated from L and L_{fail}, is also calculated.

Table C.5: Summary of Nail Loads from SNAP-2

Nail No.	Maximum Force, T_{max_i} (kip)	Controlling Limit State	L_{fail} (ft)	L_p (ft)
1	3.2	Pullout	16.1	1.9
2	7.2	Pullout	13.6	4.4
3	12.5	Pullout	10.4	7.6
4	19.0	Pullout	6.4	11.6
5	24.7	Tensile	2.4	15.6

T_{max} = 24.7 kip is at the lowermost nail, as expected for this shape of a slip surface and uniform-length nails. The force distribution among nails calculated by SNAP-2 would suggest that the upper nails are not as effectively utilized as the lowest nail in the final wall

configuration. However, this is an artifact of the limit-equilibrium analysis. The nail head force at this nail is estimated as $T_o = 0.7 \times 24.7 = 17.3$ kip.

SNAP-2 includes an option to automatically shorten the rows of nails in the lower half of the wall following a procedure by Byrne et al. (1996), which is similar to the method described in Chapter 6. This option was not used in this design example.

As an alternative to the reduction of soil nail lengths in the lower half of the wall, a non-uniform soil length pattern was selected manually in SNAP-2 for all rows. The objective was to obtain the same or comparable FS_{OS}. The soil nail lengths were selected, from the top down, as L = 25, 21, 19, 15, 13 ft. The following was calculated: FS_{OS} =1.50, T_{max} = 15.9 kip, and minimum nail load 10.2 kip. These results indicate a much better utilization of the capacity of all nails.

Summary of Overall Stability

The analyses indicate that all construction stages, even temporary stages, are safe and produced adequate factors of safety. The analyses also indicate that a uniform pattern of soil nail lengths leads to an uneven utilization of the soil nail capacities over the height of the wall. A non-uniform pattern of soil nail lengths results in smaller soil nail loads, and presents the opportunity to consider a reduction in the section of the nails.

6.b Verify Global Stability

Because SNAP-2 allows extending the slip surfaces to extend below the toe of the wall, the user can check global stability. The calculated factors of safety for global stability were larger than the minimum recommended values. The geometry and subsurface conditions of the example do not warrant the use of a different limit-equilibrium program.

6.c Verify Basal Heave

This case does not need to be evaluated in this example because the soils underlying the wall cannot generate basal heave instability.

Step 7: Strength Limit States

7.a Verify Pullout Resistance

The nominal pullout resistance, R_{PO}, is calculated for the most critically loaded nail to conduct the LRFD verification as follows:

$$R_{PO} = r_{PO}\, L_P = \pi\, q_U\, D_{DH}\, L_P$$

Where:

q_U = nominal bond strength of the nail-grout-soil interface (force/unit area)

D_{DH} = diameter of the drill hole

L_P = pullout development length

After replacing:

$R_{PO} = \pi \times 14.5 \times 144 \times 0.5 \times 15.6/1000 = 51.2$ kip

SNAP-2 calculates this value as $24.7 \times 2.0 = 49.4$ kip.

The pullout resistance is verified through the capacity-to-demand ratio (CDR) as follows:

$$CDR = \frac{\phi_{PO} \; R_{PO}}{\gamma \; T_{max}} \geq 1.0$$

Where, $\gamma = \gamma_{EV} = 1.35$ is the load factor selected for LRFD verification of soil nail walls.
After replacing:

$$CDR = \frac{0.65 \times 51.2}{1.35 \times 24.7} = 1.0 \qquad \text{ok}$$

7.b Verify Sliding Stability

This case does not need to be evaluated in this example.

7.c Verify Nail Tensile Resistance

The nominal tensile resistance, R_T, is calculated for the LRFD verification as follows:

$$R_T = A_t f_y$$

Where:

A_t = cross-sectional area of nail tendon

f_y = nominal yield resistance of nail tendon

After replacing:

$R_T = 0.60 \times 75 = 45$ kip

SNAP-2 calculates this value as $24.7 \times 1.8 = 44.5$ kip.

The tensile CDR is verified as follows:

$$CDR = \frac{\phi_T \ R_T}{\gamma \ T_{max}} \geq 1.0$$

Considering again that $\gamma = \gamma_{EV} = 1.35$ and after replacing:

$$CDR = \frac{0.75 \times 45}{1.35 \times 24.7} = 1.01 > 1.0 \qquad \text{ok}$$

7.d Verify Facing Bending/Flexure Resistance

Initial Facing

The nominal bending/flexural resistance, R_{FF}, is calculated for the most critically loaded nail head to conduct the LRFD verification. Considering that the WWM and the rebar are both Grade 60 steel, the reinforcement is the same in the vertical and horizontal directions, $f'_c = 4{,}000$ psi for the shotcrete, and $S_V = S_H = 5$ ft, R_{FF} is estimated as follows:

$$R_{FF}[\text{kip}] = 3.8 \times C_F \times f_y[\text{ksi}] \times F$$

Where:

C_F = 2 for $h_i = 4$ in. (from Table 6.8)

F = $(a_{vn} + a_{vm})$ [in^2/ft] x h_i [ft]

a_{vn} = cross-sectional area per unit width of reinforcement in the vertical direction at head

a_{vm} = cross-sectional area per unit width of reinforcement in the vertical direction at midspan

For a 6 × 6 - W2.9 × W2.9 WWM, a_{vm} is 0.058 in.^{2}/ft, from Table A.5.

$$a_{vn} = a_{vm} + \frac{A_{VN}}{S_H}$$

Where:

A_{VN} is the cross-sectional area of additional rebar over the nail head, in the vertical direction. Two #5 bars are used in the vertical direction and, from Table A.6:

$A_{VN} = 2 \times 0.31 = 0.62 \text{ in.}^2$

$$a_{vn} = 0.058 + \frac{0.62}{5} = 0.18 \text{ in.}^2/\text{ft}$$

$F = (0.058 + 0.18) \times (4/12) = 0.079$

$$R_{FF} = 3.8 \times 2 \times 60 \times 0.079 = 36 \text{ kip}$$

Alternatively, R_{FF} can be estimated by first calculating the total reinforcement ratio for the initial facing, which is defined as:

$$\rho_{TOT}(\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)} \right] \times 100 = (0.058 + 0.18)/(12 \times 2) \times 100 = 0.99\%$$

Using Table 6.9b, R_{FF} is also estimated to be 36 kip.

The facing flexure CDR is verified as follows:

$$CDR = \frac{\phi_{FF} R_{FF}}{\gamma T_o} \geq 1.0$$

Considering again that $\gamma = 1.35$ and after replacing:

$$CDR = \frac{0.90 \times 36}{1.35 \times 17.3} = 1.39 > 1.0 \qquad \text{ok}$$

The additional reinforcement over the nail head could have also been accomplished by overlapping a rectangular piece of WWM with enough development length (AASHTO 2014 or ACI 2011) at this location.

The existing reinforcement ratio should also fall between the minimum and maximum recommended reinforcement ratios which are defined as:

$$\rho_{min}[\%] = 0.24 \frac{\sqrt{f'_c[\text{psi}]}}{f_y[\text{ksi}]} = 0.24 \frac{\sqrt{4{,}000}}{60} = 0.25\% < \rho_{TOT} = 0.99\% \qquad \text{ok}$$

$$\rho_{max}[\%] = 0.05 \frac{f'_c[\text{psi}]}{f_y[\text{ksi}]} \left(\frac{90}{90 + f_y[\text{ksi}]} \right) = 0.05 \frac{4{,}000}{60} \left(\frac{90}{90 + 60} \right) = 2\% > \rho_{TOT} = 0.99\% \qquad \text{ok}$$

Reinforcement details should follow AASHTO (2014) specifications or ACI (2011) and should include:

- Minimum cover of 2 in.
- Appropriate development lengths
- Appropriate splice locations and lengths

Final Facing

For the final facing, RFF can be estimated using approximate expression or using Table 6.9b, considering $C_F = 1$.

The same reinforcement is used in both directions (Grade 60 No. 4 @ 12 in. each way), $h_f = 8$ in. and $f'_c = 4{,}000$ psi.

$a_{vm} = a_{vn} = 0.20$ in.^{2}/ft (from Table A.6).

$$\rho_{TOT}\ (\%) = \left[\frac{a_{vn} + a_{vm}}{12\left(\frac{h_i}{2}\right)} \right] \times 100 = 2 \times 0.20 / (12 \times 4) = 0.83\%$$

Therefore:

$$R_{FF} = 3.8 \times 1 \times 60 \times 2 \times 0.20 \times \left(\frac{8}{12}\right) = 60.8 \text{ kip}$$

Using Table 6.9b and interpolating, R_{FF} can be estimated to be 59.1 kip.

The facing flexure CDR is verified as follows:

$$CDR = \frac{\phi_{FF}\ R_{FF}}{\gamma\ T_o} \geq 1.0$$

Considering again that $\gamma = 1.35$ and after replacing:

$$CDR = \frac{0.90 \times 60.8}{1.35 \times 17.3} = 2.34 > 1.0 \qquad \text{ok}$$

Note that for a shotcrete with $f'_c = 3{,}000$ psi, CDR ~ 1.76.

In addition, the existing reinforcement ratio should fall between the minimum and maximum recommended reinforcement ratios:

$\rho_{min}[\%] = 0.25\% < \rho_{TOT} = 0.83\%$ ok

$\rho_{max}[\%] = 2\% > \rho_{TOT} = 0.83\%$ ok

7.e Verify Facing Punching Shear Resistance

Initial Facing

The punching shear limit state includes slippage along a truncated cone of mean diameter $D'_C = L_{BP} + h_i$. The resisting shear force developed in this slip surface, V_F, is calculated as:

$$V_F\,[\text{kip}] = 0.58\sqrt{f'_c\,[\text{psi}]}\;\pi\;D_c'[\text{ft}]h_i[\text{ft}]$$

For the problem conditions:

$$V_F = 0.58\sqrt{4000}\;\pi \times \frac{(9+4)}{12} \times \frac{4}{12} = 41.6\,\text{kip}$$

For shotcrete with $f'_c = 3{,}000$ psi, $V_F \sim 36$ kip.

The nominal resistance against punching shear, R_{FP}, is calculated as follows:

$$R_{FP} = C_P V_F$$

Assuming conservatively that C_P is equal to 1.0; therefore, $R_{FP} = 41.6$ kip.

This value can also be estimated from Table 6.9.b, for $h_i = 4$ in., $f'_c = 4{,}000$ psi, and $L_{BP} = 9$ in. and $R_{FP} = 40$ kip, which is about the same as the value above.

Considering again that $\gamma = \gamma_{EV} = 1.35$, the facing punching shear CDR is verified as follows:

$$\text{CDR} = \frac{\phi_{FP}\;R_{FP}}{\gamma\;T_o} \geq 1.0$$

After replacing:

$$\text{CDR} = \frac{0.90 \times 41.6}{1.35 \times 17.3} = 1.58 > 1.0$$ ok

For a shotcrete with $f'_c = 3{,}000$ psi, CDR ~ 1.38.

Final Facing

The resisting shear force, V_F, is calculated for the final facing as:

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

D'_C is defined for final facings as D'_c = minimum of (S_{SH} + h_c, 2 h_c). S_{SH} is the separation between headed studs, and h_c is the effective length of the headed stud, which is defined as:

$h_c = L_S + t_P - t_{SH} = 4.125 + 1 - 0.31 = 4.8$ in.

$S_{SH} = 6$ in.

$D'_c = \min(6 + 4.8,\ 2 \times 4.8) = \min(10.8,\ 9.6) = 9.6$ in.

The resisting shear force is then:

$$V_F = 0.58\,\sqrt{4000}\;\pi\left(\frac{9.6}{12}\right)\frac{4.8}{12} = 36.9 \text{ kip}$$

R_{FP} is calculated again as $R_{FP} = C_P\,V_F$. Assuming conservatively that C_P is equal to 1.0, then R_{FP} = 36.9 kip. SNAP-2 calculates this value equal to 35.1 kip.

The facing punching shear CDR is verified as follows:

$$\text{CDR} = \frac{\phi_{FP}\;R_{FP}}{\gamma\;T_o} \geq 1.0$$

After replacing:

$$\text{CDR} = \frac{0.90 \times 36.9}{1.35 \times 17.3} = 1.42 > 1.0 \qquad \text{ok}$$

7.f Verify Facing Headed Stud Tensile Resistance

Calculate the facing headed stud tensile resistance as:

$R_{FH} = N\,A_S\,f_y = 4 \times (\pi\,D_{SC}^2/4) \times 60 = 4 \times (\pi\,0.5^2/4) \times 60 = 47.1$ kips.

Considering again that $\gamma = 1.35$, the facing headed stud tensile CDR is verified as follows:

$$\text{CDR} = \frac{\phi_{FH}\;R_{FS}}{\gamma\;T_o} \geq 1.0$$

After replacing:

$$CDR = \frac{0.70 \times 47.1}{1.35 \times 17.3} = 1.42 > 1.0 \qquad \text{ok}$$

Summary of Strength Limit States

The verifications indicate that for all strength limit states, the existing factored resistances are greater than the factored loads. The analyses also suggest that, while the section of the tendons is appropriate for a uniform pattern of soil nail length, other elements of the wall can be optimized. For example, the strength of the initial shotcrete can be decreased to 3,000 psi, or the amount of reinforcement in the final facing may be decreased.

7.g Other Facing Design Considerations

No other considerations for facing are necessary in this example.

Step 8: Service Limit States

8.a Evaluate Wall Lateral and Vertical Displacements

Wall deflections induced by construction and operation can be estimated from correlations presented in Section 5.9. 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:

$\delta_h = \delta_v = H / 500 = 25 \times 12 / 500 = 0.6$ in.

The wall deformations are expected to decrease to insignificant values over a distance D_{DEF} behind the wall. Considering that wall has batter equal to zero, this distance is estimated as (Figure 5.15):

$D_{DEF} = C\ H\ (1 - \tan \alpha) = 0.8\ H = 20$ ft

8.b Evaluate Lateral Squeeze (if applicable)

This case is not applicable in this example.

Attachment 1: Input in SNAP-2

SNAP-2 Input

General Information

Entry	Symbol	Unit	Description/Remarks
Name	NA	-	Name of project
Number	NA	-	Project number/ID
Wall #	NA	-	Wall number
Station	NA	-	Roadway station number
Designer	NA	-	Name of designer

Geometry

Enter coordinates of ground surface and top of each soil layer as a coordinated pair (X, Y), where: X: horizontal coordinate, Y: vertical coordinate. (0, 0) is the base of the wall.

Soil Properties

Entry	Symbol	Unit	Description/Remarks
Name	-	-	Name of soil
Texture	-	-	Soil/rock type
Unit weight	γ'_s	pcf	Effective
Friction angle	ϕ'	degree	Eff., ultimate/nominal value
Wall-Soil interface friction angle	δ_s	degree	$\delta_s = 2/3\ \phi'$
Cohesion	c'	psf	Effective
Bond strength	q_U	psi	Ultimate/nominal

SNAP calculates bearing capacity factor (N_c [cohesion], N_q [overburden] and N_γ [soil weight]) from above input.

Groundwater

Enter coordinates as (X, Y) of groundwater table, where X: horizontal coordinate, Y: vertical coordinate.

Nail properties

Factors of Safety

Entry	Symbol	Unit	Description/Remarks
FS - yield strength – Static	FoS_y	-	Default value = 1.80
FS - yield strength – Seismic	FoS_{ys}	-	Default value = 1.35
FS - pullout – Static	FoS_p	-	Default value = 2.00
FS - pullout – Seismic	FoS_{ps}	-	Default value = 1.50

Note: Use the same values for all bars.

Bar Properties

Entry	Symbol	Unit	Comment
Drill hole diameter	D	inch	$D > D_{out}$
Outside diameter of bar	D_{out}	inch	$D_{out} > D_{in}$ only for HBSNs
Inside diameter of bar	D_{in}	inch	only for HBSNs
Nail size	Bar No.	#	Most common: 6 – 10
Steel yield strength	F_y	ksi	Typically Grade 60 or 75

Temporary Facing *(same as initial facing)*

General

Entry	Symbol	Unit	Description/Remarks
Mesh	NA	-	Enter **true** if initial facing has mesh
Bars	NA	-	Enter **true** if initial facing has rebar

Mesh Properties

Entry	Symbol	Unit	Description/Remarks
Vertical spacing of wire mesh	S_{vw}	inch	Consult Table A.6
Horizontal spacing of wire mesh	S_{hw}	inch	Usually $S_{hw} = S_{vw}$
Cross-sectional area of wire mesh	A_{wire}	$inch^2$	Consult Table A.6
Wire-mesh yield strength	Mesh F_y	ksi	Consult Table A.6. Usual. Grade 60

Rebar Properties

Entry	Symbol	Unit	Description/Remarks
Number of horizontal bars	H_{Bars}	-	Minimum of 2
Horizontal reinforcement spacing	h_r	inch	Can use S_{vw}
Horizontal bar size	H	#	3-10
Horizontal bar diameter	d_W	inch	Consult Table A.6
Horizontal bar yield strength	HF_y	ksi	Usually Grade 60
Number of vertical bearing bars	V_{Bars}		Minimum of 2
Vertical reinforcement spacing	v_r	inch	Can use S_{hw}
Vertical bearing bar size	V	#	3-10
Vertical bearing bar diameter	d_B	inch	Consult Table A.6
Vertical bearing bar length	L_{cvb}	ft	Min. 0.75 soil nail vert. spacing
Bearing bar yield strength	VF_y	ksi	Same as HF_y

Shotcrete Properties

Entry	Symbol	Unit	Description/Remarks
Shotcrete compressive strength	f'_c	psi	Typically 3 ksi or 4 ksi
Shotcrete facing thickness	h_c	inch	Minimum 4 in.
Factor - non-uniform pressures	CF	-	Usually CF = 2 for h_c = 4 in.
Shear pressure factor	CS	-	Usually CS = 1
FS for flexure/punching – Static	TF FoS	-	Typically 1.5
FS for flexure/punching – Seismic	TF_s FoS	-	Typically 1.1

Bearing Plate Properties

Entry	Symbol	Unit	Description/Remarks
Bearing plate side length	b_{PL}	inch	Typical minimum 8 in.
Bearing plate thickness	b_d	inch	Minimum 1 in.
Nail head service load factor	F_F	-	Usually 1.0

Permanent Facing *(same as final facing)*

Mesh Properties

Entry	Symbol	Unit	Description/Remarks
Vertical spacing of wire mesh	S_{vw}	inch	Typically not used in final facing
Horizontal spacing of wire mesh	S_{hw}	inch	Typically not used in final facing
Cross-sectional area of wire mesh	A_{wire}	$inch^2$	Typically not used in final facing
Wire-mesh yield strength	Mesh F_y	ksi	Typically not used in final facing

Rebar Properties

Entry	Symbol	Unit	Description/Remarks
Horizontal spacing of bars	h_r	inch	Typically 12 in.
Horizontal bar size	H	#	3-10
Horizontal bar diameter	d_W	inch	Consult Table A.1
Horizontal bar yield strength	HF_y	ksi	Usually Grade 60
Vertical bar spacing	v_r	inch	Usually same as horizontal spacing
Vertical bar size	V	#	Usually same as horizontal bars
Spacing bar diameter	d_B	inch	Usually same as horizontal bars
Spacing bar yield strength	VF_y	ksi	Usually same as for horizontal bars

Shotcrete/Concrete Properties

Entry	Symbol	Unit	Description/Remarks
Compressive strength	f'_c	psi	Typically 3 ksi or 4 ksi
Facing thickness	h_c	inch	Typically 8 in.
Factor - non-uniform pressures	CF	-	Usually CF = 1 for h_c = 8 in.
Shear pressure factor	CS	-	Usually CS = 1
FS for flexure/punching – Static	TF FoS	-	Typically 1.5
FS for flexure/punching – Seismic	TF_s FoS	-	Typically 1.1

Headed Stud Properties

Entry	Symbol	Unit	Remarks
Headed Stud body diameter	D_{HS}	inch	Consult Table A.8
Headed Stud head diameter	D_H	inch	Consult Table A.8
Headed Stud head thickness	t_H	inch	Consult Table A.8
Headed Stud overall length	L_S	inch	Consult Table A.8
Headed Stud spacing	S_{hs}	inch	4 in. to 6 in.
Headed Stud yield strength	F_y	ksi	Grade 60 (usual) Grade or 75
No. of headed studs in connection	N_H	-	Commonly 4
Tensile fracture factor – Static	HT-FoS	-	$1/FS_{FH}$ = 0.50 (A307), 0.60 (A325)
Tensile fracture factor – Seismic	HT_s-FoS		$1/FS_{FH}$ = 0.67 (A307), 0.77 (A325)

Attachment 2: Abridged Output from SNAP-2

SNAP_2 Report

Name	Number	Company	Wall #	Station	Designer	Reviewer	Date
Design Example GEC7 2014	1	Geo USer 1	1	0+00	JD	JD Jr.	6/8/14

Name: Name of project.
Number: Project number or ID
Company: Name of company
Wall #: Wall number
Station: Roadway station number
Designer: Name of person performing design.
Reviewer: Name of person performing project review.
Date: Date of project

Soils

Soil Properties

Name	Texture	γ'_s, pcf	ϕ', ^{o}	δ_s, ^{o}	c', psf	q_u, psi	N_c	N_q	N_γ
Med. Dense Silty Sand	sand	125	32	21.3	100.0	14.5	35.5	23.2	30.2

Name: Name of soil
Texture: Soil/rock Type
γ'_s: Effective unit weight of soil
ϕ': Effective soil friction angle / angle of internal friction
δ_s: Wall-soil interface friction angle, $\delta = 2/3\phi$
c': Effective cohesion of soil
q_u: Ultimate bond strength
N_c: N_c bearing capacity factor
N_q: N_q bearing capacity factor
N_γ: N_γ bearing capacity factor

Nails

Default Factors of Safety

U	F_y FoS	F_{ys} FoS	F_p FoS	F_{ps} FoS
true	1.80	1.35	2.00	1.50

U: Use same factors of safety for each bar
F_y FoS: Factor of safety for yield strength
F_{ys} FoS: Seismic factor of safety for yield strength
F_p FoS: Factor of safety for pullout
F_{ps} FoS: Seismic factor of safety for pullout

Bar Properties

Name	D, in	D_{out}, in	D_{in}, in	Bar No, Bar #	F_y, ksi
Bar 1	6.0	0.874	0.000	7	0.874

Name: Name of bar set
D: Drill hole diameter
D_{out}: Outside diameter of bar
D_{in}: Inside diameter of bar
Bar No: Nail size 3-18
F_y: Steel yield strength of bar

Facings

Facing Properties

#	Type	Name	Description
1	Temp SNW	Temp SNW 1	-
2	Perm SNW	Perm SNW 1	-

Type: Facing type
Name: Name of facing
Description: Facing description

Temp SNW 1:

Mesh	Bars
true	true

Mesh: true if temporary facing has mesh reinforcement
Bars: true if temporary facing has bar reinforcement

Mesh: Temporary facing mesh

S_{vw}, in	S_{hw}, in	A_{wire}, in^2	$Mesh_{Fy}$, ksi
6.0	6.0	0.029	60.0

S_{vw}: Vertical mesh spacing of wires
S_{hw}: Horizontal mesh spacing of wires
A_{wire}: Mesh area of wire
$Mesh_{Fy}$: Wire mesh yield strength

Bars: Temporary facing bars

H_{Bars}	hr, in	H, Bar #	d_W, in	H_{Fy}, ksi	V_{Bars}	vr, in	V, Bar #	d_B, in	L_{cvb}, ft	V_{Fy}, ksi
2	12	5	0.626	60.0	2	12	5	0.626	5.0	60.0

H_{Bars}: Number of horizontal waler bars
hr: Horizontal reinforcement spacing
H: Horizontal waler bar size, 3-10
d_W: Horizontal bar diameter
H_{Fy}: Horizontal bar yield strength
V_{Bars}: Number of vertical bearing bars
vr: Vertical reinforcement spacing
V: Vertical bearing bar size, 3-10
d_B: Vertical bearing bar diameter
L_{cvb}: Vertical bearing bar length
V_{Fy}: Bearing bar yield strength

Shotcrete: Temporary shotcrete facing

f'_c, psi	h_c, in	C_F	C_S	TF FoS	TF_s FoS
4000	4.0	2	1	1.50	1.10

f'_c: Shotcrete facing compressive strength
h_c: Shotcrete facing thickness
C_F: Flexure pressure factor (Accounts for non-uniformity of pressure at back of facing)
C_S: Shear pressure factor
TF FoS: Factor of safety for flexure and punching
TF_s FoS: Seismic factor of safety for flexure and punching

Plate: Temporary facing plate

b_{PL}, in	b_d, in	F_F
9.0	1.0	1

b_{PL}: Bearing plate side length
b_d: Bearing plate thickness
F_F: Nail head service load factor

Perm SNW 1:

Temporary Facing	Mesh	Bars	Studs
Temp SNW 1	false	true	true

Temporary Facing: Temporary wall facing behind this permanent facing
Mesh: True if permanent facing has mesh reinforcement
Bars: true if permanent facing has bar reinforcement
Studs: true if permanent facing has studs

Bars: Permanent facing bars

hr, in	H, Bar #	d_W, in	H_{Fy}, ksi	vr, in	V, Bar #	d_B, in	V_{Fy}, ksi
12	4	0.500	60.0	12	4	0.500	60.0

hr: Horizontal reinforcement spacing
H: Horizontal waler bar size, 3-10
d_W: Horizontal bar diameter
H_{Fy}: Horizontal bar yield strength
vr: Vertical reinforcement spacing
V: Vertical bearing bar size, 3-10
d_B: Vertical bearing bar diameter
V_{Fy}: Bearing bar yield strength

Concrete: Permanent facing concrete

f'c, psi	h_c, in	C_F	C_S	PF FoS	PF_s FoS
4000	8	1.000	1.000	1.50	1.10

f'_c: Concrete compressive strength
h_c: Permanent facing thickness
C_F: Flexure Pressure Factor, Table 4.2 (Accounts for non-uniformity of pressure at back of facing)
C_S: Shear pressure factor
PF FoS: Factor of safety for flexure and punching
PF_s FoS: Seismic factor of safety for flexure and punching

Studs: Permanent facing studs

D_{Hs}, in	D_H, in	t_H, in	L_S, in	S_{hs}, in	F_y, ksi	P_{Thick}, in	N_H	HT FoS	HT_s FoS
0.5	1	0.31	4.125	6	60	1	4	2.00	1.50

D_{Hs}: Stud body diameter
DH: Stud head diameter, dh: Stud head diameter OK: dh >= 1.58 * dhs, 1 >= 1.58 * 0.5, 1 >= 0.79
tH: Stud head thickness, th: Stud head thickness Ok: th >= (dh - dhs) / 2, 0.3 >= (1 - 0.5) / 2, 0.3 >= 0.25
LS: Stud overall length
S_{hs}: Stud spacing
F_y: Stud yield strength
P_{Thick}: Plate thickness
N_H: Number of headed studs in the connection
HT FoS: Headed stud tensile fracture factor (for ASTM A307, $?_{FHS}$ = 0.50; for ASTM A325 $?_{FHS}$ = 0.59)
HT_s FoS: Seismic headed stud tensile fracture factor

SN Wall 1:

Static Case

Construction: Construction specification

Construction #	Con_{seq}
20	1

Construction #: Construction number, adds stage cuts and nails according to assigned construction sequences
Con_{seq}: Construction (stage cut) sequence when wall construction begins, i.e., "1" or "2,4-6"

Wall: Soil nail wall size and location

Facing	H_{Bars}	V_{Bars}	Base, ft	Top, ft	H, ft	θ, °	Emb, ft	Width, ft
Perm SNW 1	25	200	0.0,0.0	0.0,25.0	25.0	0.0	0.0	200

Facing: Wall facing
H_{Bars}: Number of horizontal waler bars
V_{Bars}: Number of vertical bearing bars
Base: Base of wall
Top: Top of wall
H: Wall height
θ: Wall batter angle, degrees from vertical
Emb: Embedment, depth below ground surface at toe
Width: Width of wall, extending along Z-Axis

Nails: Soil nail sizes and locations

Nail	L, ft	S_V, ft	S_H, ft	δ, °	C_d, ft	O	U
Bar 1	18.0	5.00	5.00	15.00	3.00	false	true

Nail: Bar used for this nail
L: Nail length
S_V: Vertical nail spacing
S_H: Horizontal nail spacing
δ: Nail inclination, degrees from horizontal
C_d: Cantilever distance, vertical distance from top of wall to top nail
O: Offset pattern, true if nails in even rows are offset to midspan, otherwise nails are in a square pattern
U: Use uniform nails

Nail List: Nail properties

Nail[1]

C_{dH}, ft	Failure	L_{fail}, ft	T_{Force}, kip
3.00	Pullout from Med. Dense Silty Sand	16.1	3.2

C_{dH}: Cantilever distance, vertical distance from top of wall to this nail
Failure: Failure mode for wall slip surface
L_{fail}: Distance from nail head to failure surface
T_{Force}: Nail T-force

T-Forces: Nail T-forces

#	Dist, ft	T-Force, kip	Soil	Failure
1	0.00	23.9	Med. Dense Silty Sand	Punching/Flexure Failure
2	4.87	24.0	Med. Dense Silty Sand	Pullout
3	18.02	0.0	Med. Dense Silty Sand	Pullout

Dist: Horizontal distance of T-force from nail head
T-Force: Nail T-force
Soil: Soil layer at T-force location
Failure: Failure mode at T-force location

Nail[2]

C_{dH}, ft	Failure	L_{fail}, ft	T_{Force}, kip
8.00	Pullout from Med. Dense Silty Sand	13.6	7.2

C_{dH}: Cantilever distance, vertical distance from top of wall to this nail
Failure: Failure mode for wall slip surface
L_{fail}: Distance from nail head to failure surface
T_{Force}: Nail T-force

T-Forces: Nail T-forces

#	Dist, ft	T-Force, kip	Soil	Failure
1	0.00	23.9	Med. Dense Silty Sand	Punching/Flexure Failure
2	4.87	24.0	Med. Dense Silty Sand	Pullout
3	18.00	0.0	Med. Dense Silty Sand	Pullout

Dist: Horizontal distance of T-force from nail head
T-Force: Nail T-force
Soil: Soil layer at T-force location
Failure: Failure mode at T-force location

Nail[3]

C_{dH}, ft	Failure	L_{fail}, ft	T_{Force}, kip
13.00	Pullout from Med. Dense Silty Sand	10.4	12.5

C_{dH}: Cantilever distance, vertical distance from top of wall to this nail
Failure: Failure mode for wall slip surface
L_{fail}: Distance from nail head to failure surface
T_{Force}: Nail T-force

T-Forces: Nail T-forces

#	Dist, ft	T-Force, kip	Soil	Failure
1	0.00	23.9	Med. Dense Silty Sand	Punching/Flexure Failure
2	4.87	24.0	Med. Dense Silty Sand	Pullout
3	18.00	0.0	Med. Dense Silty Sand	Pullout

Dist: Horizontal distance of T-force from nail head
T-Force: Nail T-force
Soil: Soil layer at T-force location
Failure: Failure mode at T-force location

Nail[4]

C_{dH}, ft	Failure	L_{fail}, ft	T_{Force}, kip
18.00	Pullout from Med. Dense Silty Sand	6.4	19.0

C_{dH}: Cantilever distance, vertical distance from top of wall to this nail
Failure: Failure mode for wall slip surface
L_{fail}: Distance from nail head to failure surface
T_{Force}: Nail T-force

T-Forces: Nail T-forces

#	Dist, ft	T-Force, kip	Soil	Failure
1	0.00	23.9	Med. Dense Silty Sand	Punching/Flexure Failure
2	4.87	24.0	Med. Dense Silty Sand	Pullout
3	18.00	0.0	Med. Dense Silty Sand	Pullout

Dist: Horizontal distance of T-force from nail head
T-Force: Nail T-force
Soil: Soil layer at T-force location
Failure: Failure mode at T-force location

Nail[5]

C_{dH}, ft	Failure	L_{fail}, ft	T_{Force}, kip
22.00	Tendon Failure	2.4	24.7

C_{dH}: Cantilever distance, vertical distance from top of wall to this nail
Failure: Failure mode for wall slip surface
L_{fail}: Distance from nail head to failure surface
T_{Force}: Nail T-force

T-Forces: Nail T-forces

#	Dist, ft	T-Force, kip	Soil	Failure
1	0.00	23.9	Med. Dense Silty Sand	Punching/Flexure Failure
2	4.87	24.0	Med. Dense Silty Sand	Pullout
3	18.00	0.0	Med. Dense Silty Sand	Pullout

Dist: Horizontal distance of T-force from nail head
T-Force: Nail T-force
Soil: Soil layer at T-force location
Failure: Failure mode at T-force location

Appendix D: Soil Nail Wall – Reference Manual Example Plans

FEDERAL HIGHWAY ADMINISTRATION

SOIL NAIL WALL

EXAMPLE PLANS

DRAWING INDEX

SHEET NO.	TITLE
SHEET 1	COVER
SHEET 2	CONSTRUCTION NOTES
SHEET 3	SITE PLAN
SHEET 4	WALL ELEVATIONS - SEGMENTS A AND B
SHEET 5	WALL ELEVATIONS - SEGMENTS C, D AND E
SHEET 6	TYPICAL SECTION AND DETAILS
SHEET 7	CORROSION PROTECTION DETAILS
SHEET 8	WALL FACING DETAILS
SHEET 9	WALL DRAINAGE DETAILS
SHEET 10	SUBSURFACE DATA

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	COVER SHEET 1 OF 10

GENERAL NOTES AND SOIL NAIL WALL DESIGN PARAMETERS

1. ALL MATERIAL AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF _____, DEPARTMENT OF TRANSPORTATION, STANDARD SPECIFICATIONS FOR _____ DATED _____.

2. THE SOIL NAIL WALL HAS BEEN DESIGNED IN ACCORDANCE WITH THE LRFD (LOAD AND RESISTANCE FACTOR DESIGN) PROCEDURES CONTAINED IN THE FHWA GEOTECHNICAL ENGINEERING CIRCULAR NO. 7 "SOIL NAIL WALLS - REFERENCE MANUAL". STRUCTURAL DESIGN OF WALL ELEMENTS NOT COVERED IN THE FHWA MANUAL HAVE BEEN DESIGNED IN ACCORDANCE WITH THE REQUIREMENTS OF THE _____ AASHTO _____. A SEISMIC ACCELERATION COEFFICIENT OF ___ WAS USED IN DESIGN.

3. CONCRETE MINIMUM COMPRESSIVE STRENGTHS

   SHOTCRETE: Fc = 4 KSI

   NAIL GROUT: Fc = 3 KSI

4. TENSILE STRENGTH OF NAIL BARS: Fy = 60 KSI AASHTO M31/ASTM A615

5. UNLESS NOTED ON THE PLANS OTHERWISE, MINIMUM CONCRETE/SHOTCRETE COVER MEASURED FROM THE FACE OF CONCRETE/SHOTCRETE TO THE FACE OF ANY REINFORCING BAR OR METALLIC ELEMENT SHALL BE AS FOLLOWS:
   - FRONT SIDE OF FINAL FACINGS EXPOSED TO WEATHER: 2 IN.
   - FINAL SHOTCRETE FACINGS EXPOSED TO SOIL: 3 IN.
   - FINAL CIP FACINGS CAST AGAINST INITIAL SHOTCRETE: 1.5 IN.

6. UNLESS OTHERWISE SHOWN ON THE PLANS, ALL EXTERIOR CORNERS AND EDGES SHALL HAVE A 0.75 IN. CHAMFER AND ALL INTERIOR CORNERS SHALL HAVE A 0.75 IN. FILLET.

7. DESIGN SOIL PARAMETERS:

SOIL/ROCK TYPE	NOMINAL FRICTION ANGLE (DEGREES)	NOMINAL COHESION (PSF)	UNIT WEIGHT (PCF)	NOMINAL PULLOUT RESISTANCE (KIP/FT)
CLAYEY SILT	34	150	125	3.0
CLAYEY SILT	30	250	120	2.0
SILTY SANDY GRAVEL	38	100	130	6.0

8. ALL NAIL DRILLED LENGTHS (L) AND BAR SIZES SHALL BE IN ACCORDANCE WITH NAIL WALL ELEVATION SHEETS.

9. THE CONTRACTOR IS RESPONSIBLE FOR FIELD-LOCATING ALL UTILITIES.

10. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING AND MAINTAINING STABLE SLOPES ABOVE AND BELOW THE SOIL NAIL WALLS.

11. NO GENERAL EXCAVATION OPEN CUTS STEEPER THAN 1H:1V SHALL BE MADE WITHIN 15 FT IN FRONT OF THE WALLS WITHOUT APPROVAL OF THE OWNER'S ENGINEER.

12. WHERE GRADE SLOPES AWAY FROM THE BACK OF THE WALL, THE CONCRETE DRAINAGE GUTTER SHALL NOT BE REQUIRED.

13. UNLESS SPECIFIED OTHERWISE, OWNER'S ENGINEER WILL PROVIDE SURVEY CONTROL POINTS FOR TOP OF WALL ALIGNMENT. THE CONTRACTOR IS RESPONSIBLE FOR SURVEY CONTROL DURING EXCAVATION.

14. AT THE CONTRACTOR'S OPTION, NAILS MAY BE DRILLED AND INSTALLED THROUGH A TEMPORARY STABILIZING BERM.

15. EXCAVATION IN THE VICINITY OF THE WALL FACE SHALL REQUIRE SPECIAL CARE AND EFFORT COMPARED TO GENERAL ROADWAY EXCAVATION. SEE SPECIAL PROVISIONS.

VERIFICATION TEST NAIL LOCATIONS			
SOIL TYPE	WALL SEGMENT	WALL STATION (FT)	APPROXIMATE ELEVATION (FT)
SILTY SAND	B	0 + 78.6	226
	E	0 + 395	240
CLAYEY SILT	B	0 + 153.6	228
	E	0 + 332	247
SILTY SANDY GRAVEL	D	0 + 307	236
	E	0 + 426	235

NOTES:
- SEE SPECIFICATIONS FOR TESTING REQUIREMENTS.
- SET BONDED LENGTH COMPLETELY INTO ONE SOIL UNIT FOR VERIFICATION TEST NAILS.

SOIL NAIL WALL QUANTITIES						
ITEM	SEGMENT A	SEGMENT B	SEGMENT C	SEGMENT D	SEGMENT E	SUBTOTAL
SOIL NAIL LENGTH (ft) BAR SIZE #8	1,017	1,516	1,168	2,139	942	6,781
BAR SIZE #9	0	610	367	1,473	2,843	5,294
BAR SIZE #10	0	0	0	0	1,868	1,868
BEARING PLATES (8x8x0.75 IN.)(No.)	62	107	66	172	217	624
WEIGHT (LB)	845	1,455	900	2,340	2,950	8,490
SHOTCRETE AREA - 4 IN. THICK (SF)	1,350	2,605	1,830	4,375	5,335	15,495
CIP AREA - 8 IN. THICK (SF)	1,420	2,820	2,005	4,810	5,600	16,660
TOTAL SHOTCRETE CONSTRUCTION FACING REINFORCING STEEL (LB)	1,215	2,250	1,585	3,760	4,300	13,110
TOTAL CIP WALL REINFORCING STEEL (LB)	4,450	8,675	6,140	14,680	17,540	51,480

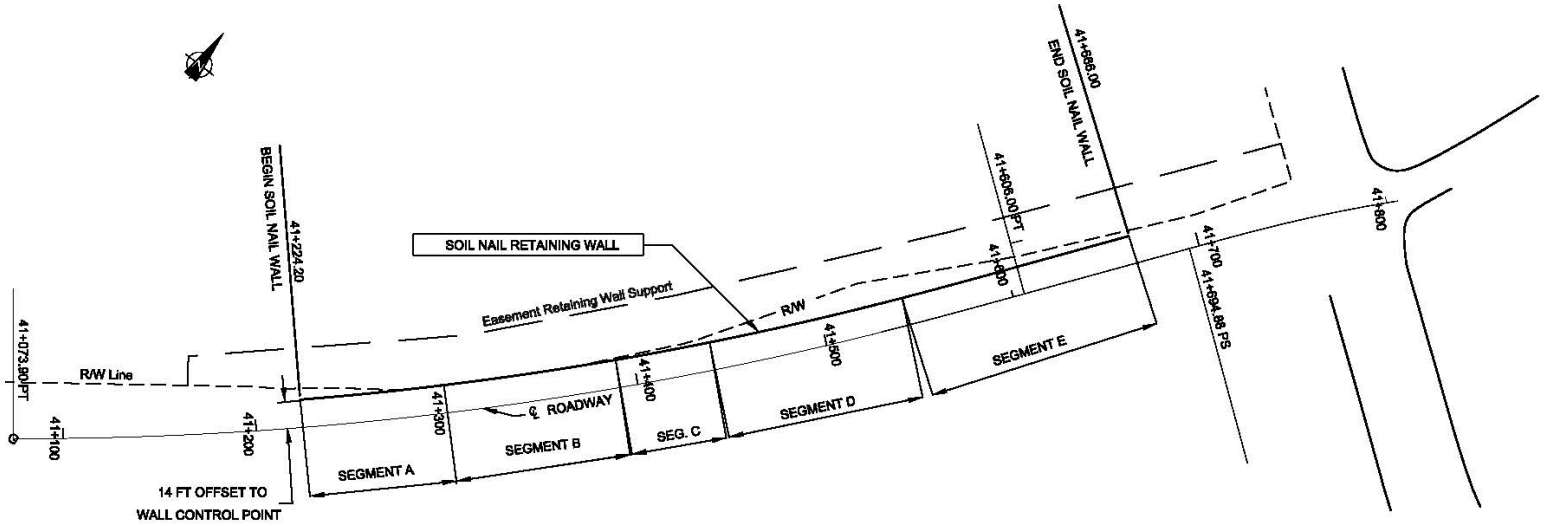
TYPICAL CONSTRUCTION SEQUENCE

1. WALLS SHALL BE BUILT FROM THE TOP DOWN IN ACCORDANCE WITH THE STAGED EXCAVATION LIFTS SHOWN ON SHEET 6 AND SPECIAL PROVISIONS.

2. THE FOLLOWING WALL CONSTRUCTION SEQUENCE FOR EACH EXCAVATION LIFT SHALL BE COMPLETE PRIOR TO INITIATING WORK ON THE NEXT EXCAVATION LIFT UNLESS OTHERWISE APPROVED BY THE OWNER'S ENGINEER.

   2.1. INSTALL PRE-PRODUCTION VERIFICATION TEST NAILS. SEE VERIFICATION TEST LOCATION IN TABLE SHOWN ON THIS SHEET.
   2.2 INSTALL INCLINOMETER(S) AS SHOWN.
   2.3. EXCAVATE TO STAGE 1 ROUGH GRADE.
   2.4. TRIM TO FINAL WALL FACE EXCAVATION LINE OR TO STABILIZING BERM (IF USED).
   2.5. INSTALL AND GROUT NAILS. TRIM STABILIZATION BERM (IF USED) TO FINAL WALL FACE EXCAVATION LINE.
   2.6. INSTALL GEOCOMPOSITE DRAINAGE STRIP.
   2.7. PLACE REINFORCING AND APPLY SHOTCRETE. NO EXPOSED EXCAVATION SHALL BE LEFT UNSTABILIZED BY SHOTCRETE AT THE END OF THE WORK DAY UNLESS ENGINEER APPROVES OTHERWISE.
   2.8. PERFORM NAIL PULLOUT TESTS PER SPECIFICATIONS BEFORE SHOTCRETE IS APPLIED AND AFTER NAIL GROUT HAS ATTAINED ITS SPECIFIED STRENGTH.
   2.9. CONDUCT VERIFICATION AND PROOF LOAD TESTS PER SPECIFICATIONS. PROTECT PROOF TEST LOCATIONS FROM SHOTCRETE. GROUT PROOF TEST SOIL NAILS.
   2.10. CONDUCT QC OF MATERIALS, INCLUDING GROUT AND SHOTCRETE PER SPECIFICATIONS.
   2.11. CONSTRUCT FOOTING DRAIN. INSTALL PVC CONNECTOR PIPES DURING CONSTRUCTION OF THE FINAL SHOTCRETE LIFT TO PROVIDE DRAINAGE OF THE GEOCOMPOSITE DRAINAGE STRIPS INTO THE FOOTING DRAIN OR WALL BASE AS SHOWN ON SHEET 9.
   2.12. INSTALL CIP FINAL FACING.
   2.13. INSTALL CONCRETE DRAINAGE GUTTER.

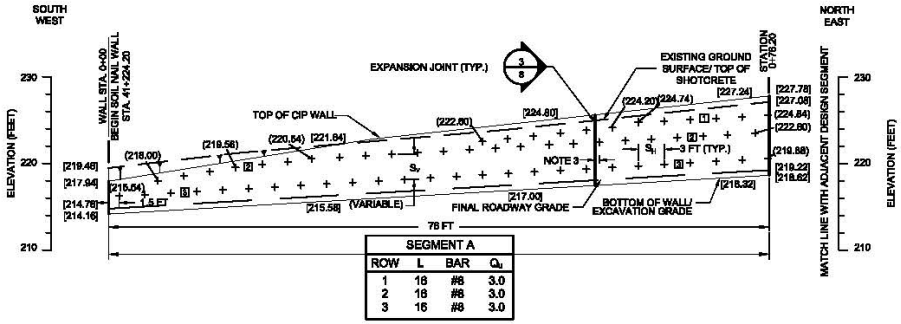
DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	CONSTRUCTION NOTES SHEET 2 OF 10



1 SITE PLAN

1" = 60'

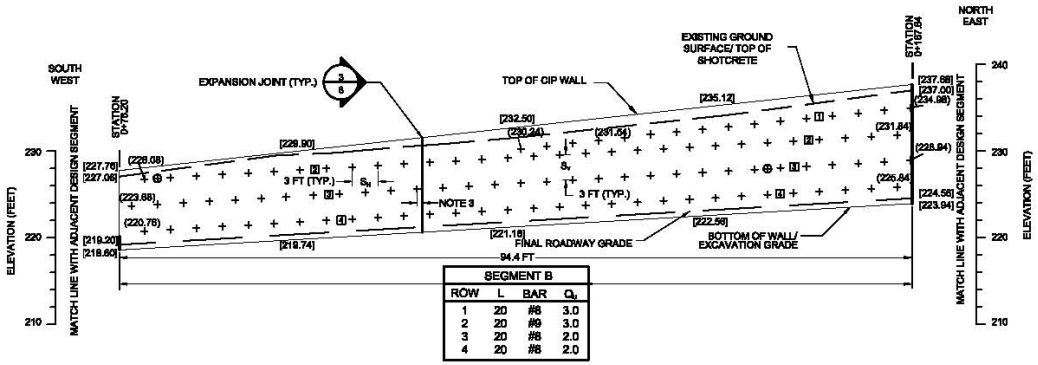
DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	SITE PLAN SHEET 3 OF 10



SEGMENT A			
ROW	L	BAR	Q_u
1	18	#8	3.0
2	18	#8	3.0
3	16	#8	3.0

1 WEST ELEVATION

0 10 FEET



SEGMENT B			
ROW	L	BAR	Q_u
1	20	#8	3.0
2	20	#9	3.0
3	20	#8	2.0
4	20	#8	2.0

2 WEST ELEVATION

0 10 FEET

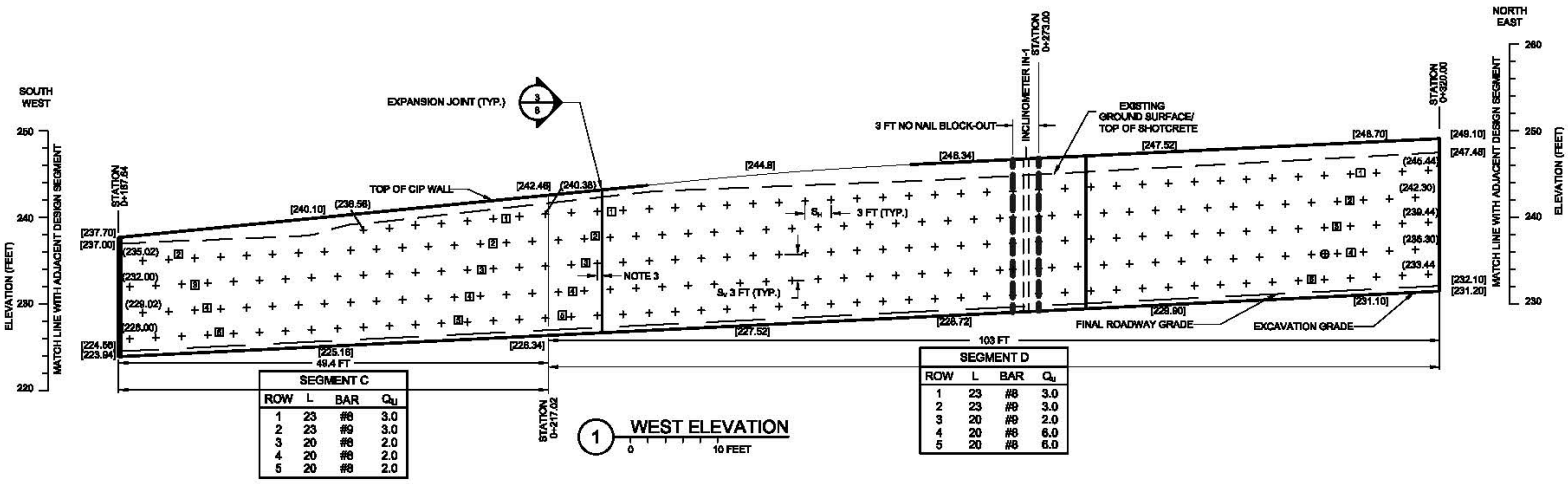
LEGEND:

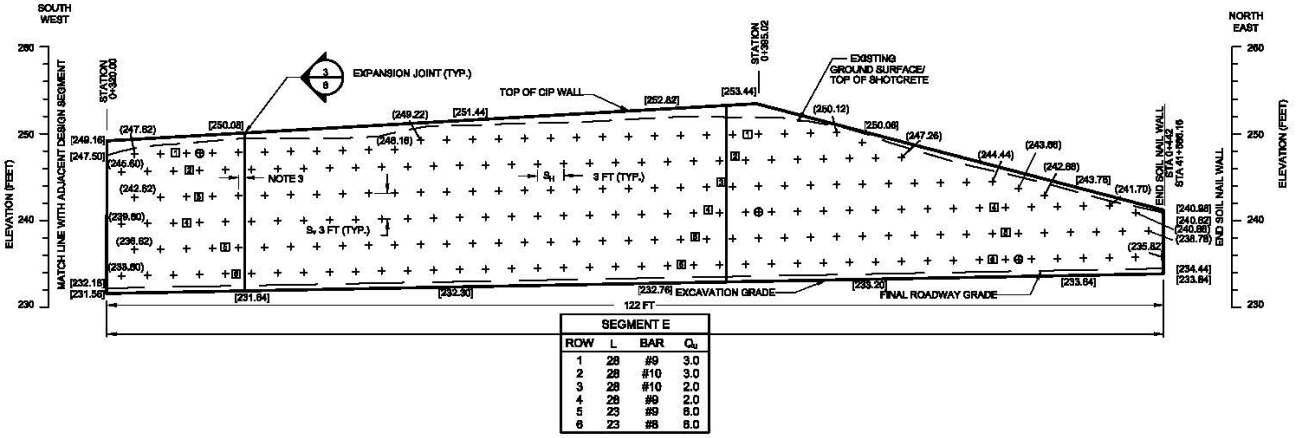
+	NAIL	L	DRILLED LENGTH OF NAIL (FEET)
⊕	VERIFICATION TEST NAIL	BAR	STEEL BAR SIZE
[2]	NAIL ROW	Q_u	NOMINAL PULLOUT RESISTANCE (KIP/FT)
(370.7)	NAIL ELEVATION	S_H	HORIZONTAL NAIL SPACING (FEET)
[373.6]	GRADE ELEVATION	S_V	VERTICAL NAIL SPACING (FEET)

NOTES:

1. ELEVATIONS SHOWN ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (M.S.L.=0.00).
2. NAIL ELEVATIONS THAT ARE NOT SHOWN SHALL BE LINEARLY INTERPOLATED BETWEEN THE ELEVATIONS SHOWN.
3. EXPANSION JOINT LOCATIONS SHALL BE SELECTED PRIOR TO THE INSTALLATION OF SOIL NAILS. AT LEAST 12 IN. CLEAR DISTANCE SHALL BE PROVIDED BETWEEN JOINTS AND NAILS.
4. PROOF TEST NAIL LOCATIONS TO BE SELECTED BEFORE GROUTING.

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	WALL ELEVATIONS - SEGMENTS A AND B SHEET 4 OF 10



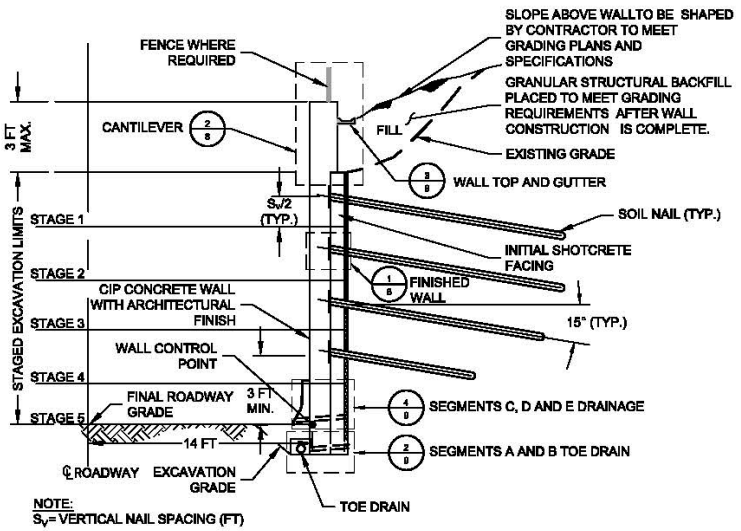


LEGEND:

+	NAIL	L	DRILLED LENGTH OF NAIL (FEET)
⊕	VERIFICATION TEST NAIL	BAR	STEEL BAR SIZE
[2]	NAIL ROW	Q_u	NOMINAL PULLOUT RESISTANCE (KIP/FT)
(370.7)	NAIL ELEVATION	S_H	HORIZONTAL NAIL SPACING (FEET)
[373.6]	GRADE ELEVATION	S_V	VERTICAL NAIL SPACING (FEET)

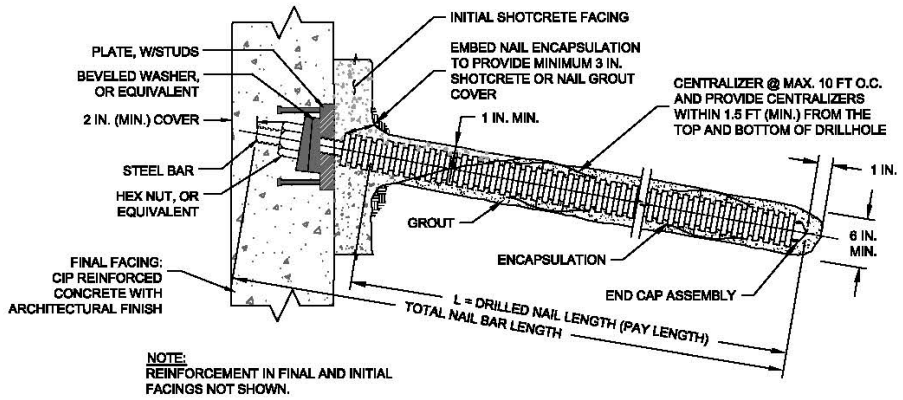
NOTES:
1. ELEVATIONS SHOWN ARE BASED ON THE NATIONAL GEODETIC VERTICAL DATUM OF 1929 (M.S.L.=0.00).
2. NAIL ELEVATIONS THAT ARE NOT SHOWN SHALL BE LINEARLY INTERPOLATED BETWEEN THOSE SHOWN ELEVATIONS.
3. EXPANSION JOINT LOCATIONS SHALL BE SELECTED PRIOR TO THE INSTALLATION OF SOIL NAILS. AT LEAST 12 IN. CLEAR DISTANCE SHALL BE PROVIDED BETWEEN JOINTS AND NAILS.

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	WALL ELEVATIONS - SEGMENTS C, D, AND E SHEET 5 OF 10



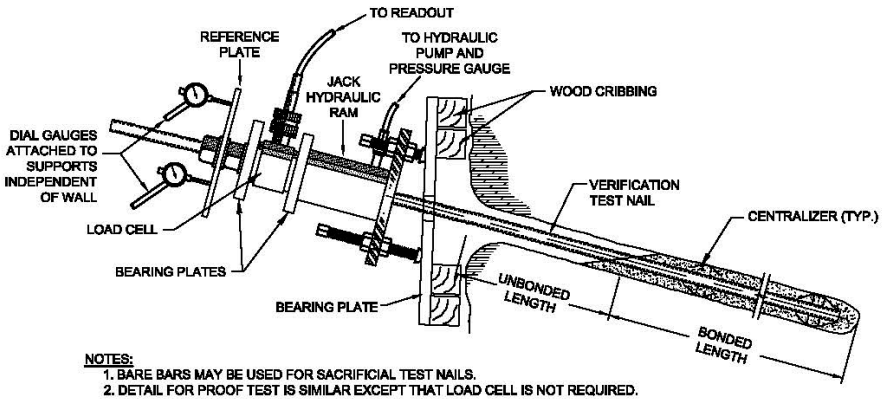
1 TYPICAL SECTION

NTS



3 ENCAPSULATED PRODUCTION SOIL NAIL DETAIL

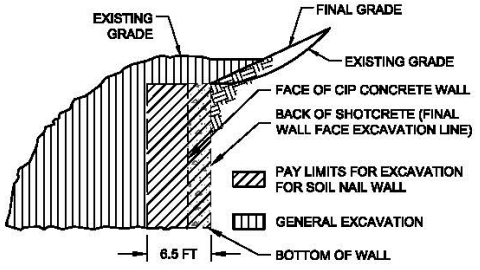
NTS



NOTES:
1. BARE BARS MAY BE USED FOR SACRIFICIAL TEST NAILS.
2. DETAIL FOR PROOF TEST IS SIMILAR EXCEPT THAT LOAD CELL IS NOT REQUIRED.

2 VERIFICATION TEST SOIL NAIL DETAIL

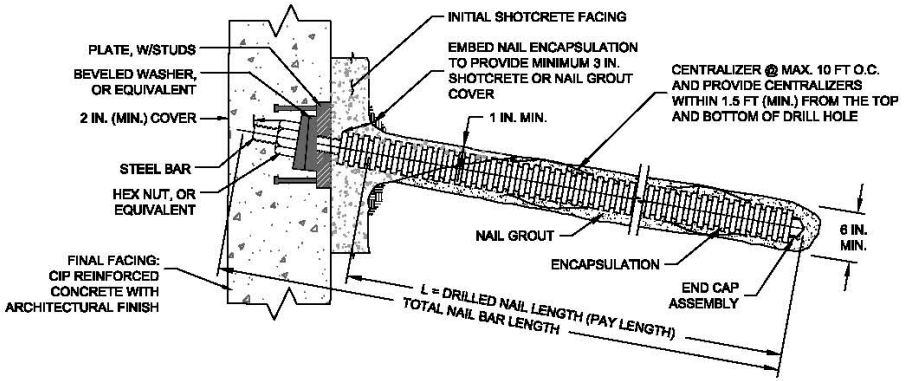
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4 EXCAVATION PAY LIMITS

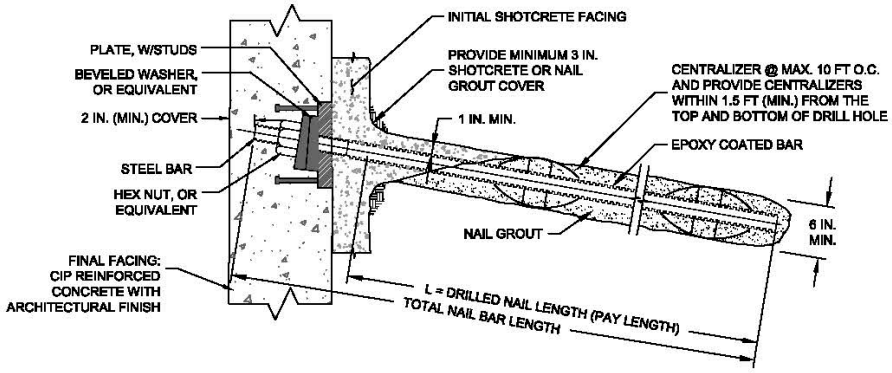
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DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	TYPICAL SECTIONS AND DETAILS SHEET 6 OF 10



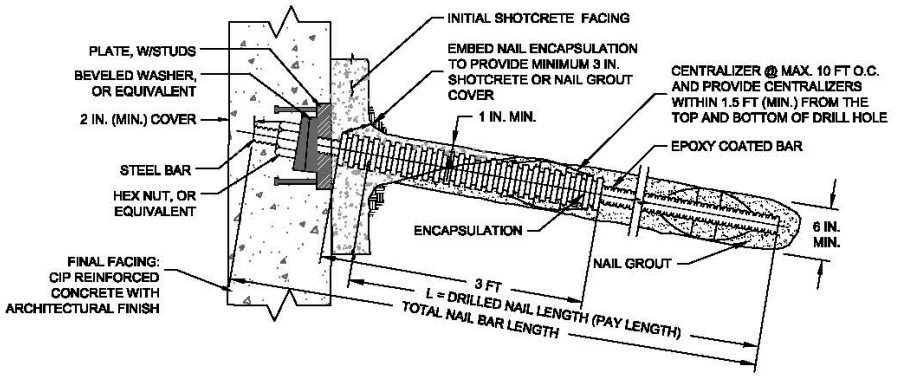
1 ENCAPSULATED SOIL NAIL WITH BARE STEEL AT THE TOP DETAIL

NTS



2 EPOXY COATED NAILS WITH MACHINE THREADS DETAIL

NTS

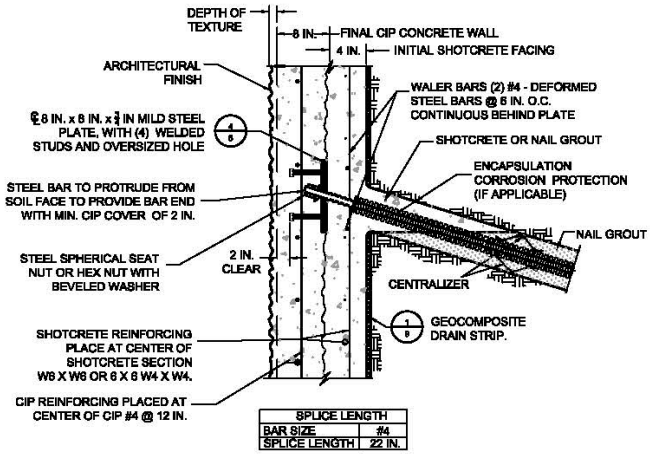


3 ENCAPSULATED (UPPER 3 FT) EPOXY COATED SOIL NAIL DETAIL

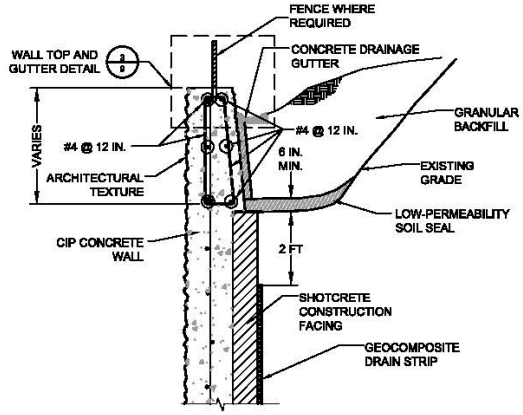
NTS

NOTE:
THE CORROSION PROTECTION SHOWN ABOVE ARE INTENDED TO ILLUSTRATE TYPICAL CORROSION PROTECTION SYSTEMS ALTERNATIVES. ONE OR MORE OF THE DETAILS WOULD BE SPECIFIED ON AN INDIVIDUAL PROJECT.

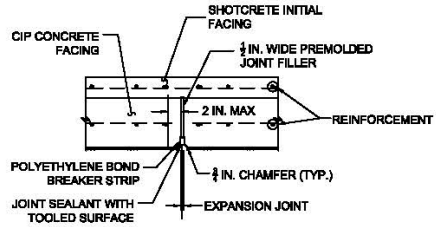
DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	CORROSION PROTECTION DETAILS SHEET 7 OF 10

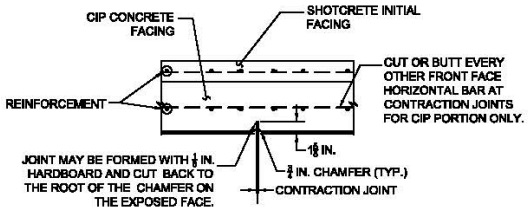


1 FINISHED WALL SECTION
NTS



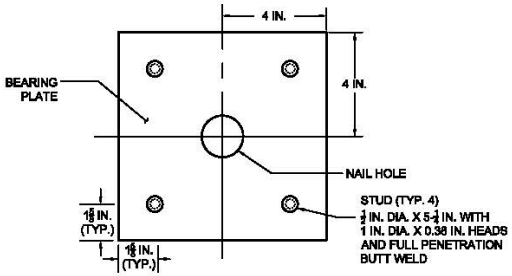
2 CANTILEVER DETAIL
NTS



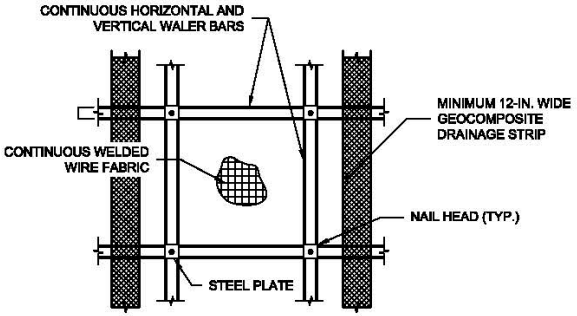


3 EXPANSION AND CONTRACTION JOINTS DETAIL
NTS

NOTE:
EXPANSION JOINTS IN CIP WALL SHALL BE LOCATED AT A MAXIMUM SPACING OF 100 FT CENTER TO CENTER, AND SHALL BE CENTERED BETWEEN NAILS, EXCEPT IF THE JOINT IS WITHIN 2 FT OF A STEP AT THE TOP OF WALL, IN WHICH CASE THE JOINT SHALL BE LOCATED AT THAT STEP. CONTRACTION JOINTS IN CIP WALL SHALL BE LOCATED AT MAXIMUM SPACING OF 50 FT CENTER TO CENTER. EXPANSION AND CONTRACTION JOINTS NOT REQUIRED THROUGH SHOTCRETE CONSTRUCTION FACING.

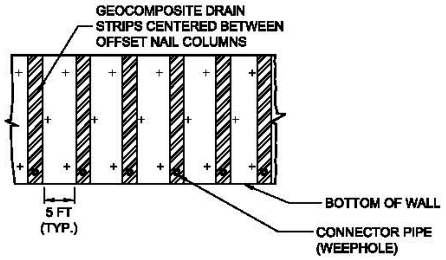


4 CONNECTOR PLATE DETAIL
NTS



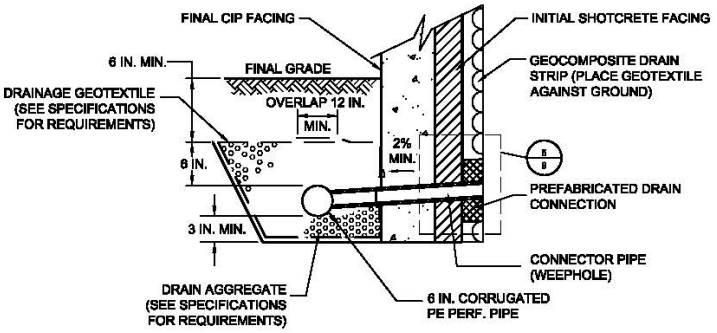
5 SOIL NAIL ELEVATION
NTS

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	WALL FACING DETAILS SHEET 8 OF 10



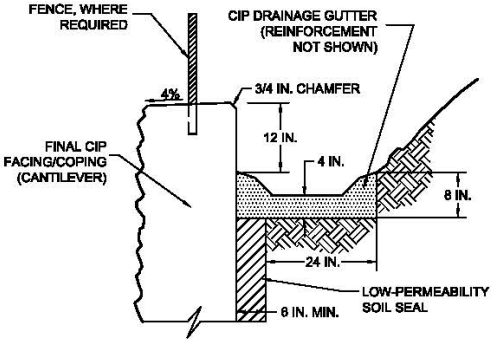
1 GEOCOMPOSITE DRAINAGE STRIP ELEVATION

NTS



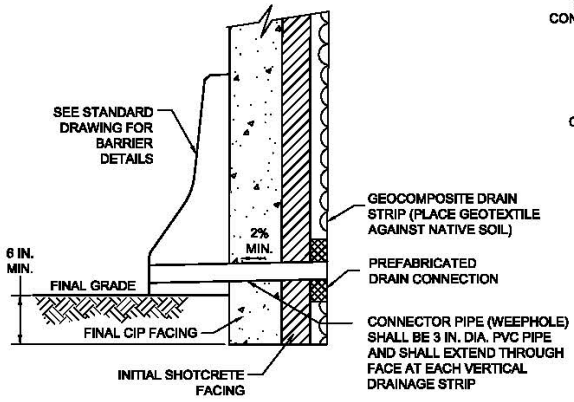
2 SEGMENTS A AND B TOE DRAIN DETAIL

NTS



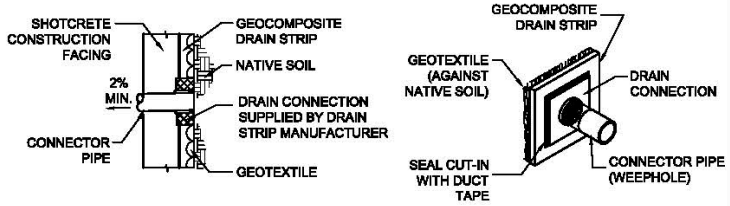
3 WALL TOP AND GUTTER DETAIL

NTS



4 SEGMENTS C, D, AND E DRAINAGE DETAIL

NTS



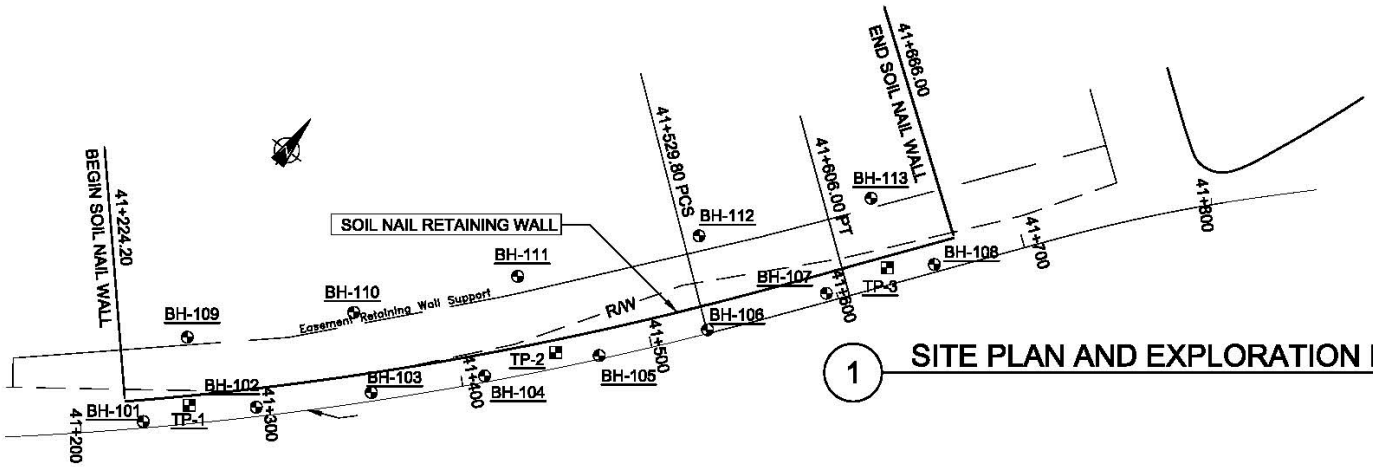
SECTIONAL VIEW ISOMETRIC VIEW

NOTE: INSTALLATION OF DRAIN CONNECTION SHALL NOT DISRUPT GEOTEXTILE

5 DRAIN CONNECTION DETAIL

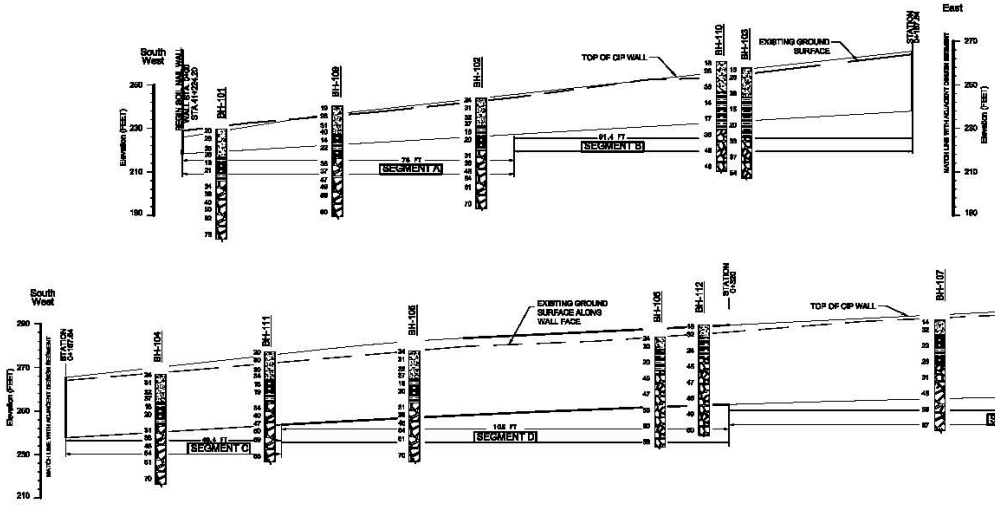
NTS

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	WALL DRAINAGE DETAILS SHEET 9 OF 10



1 SITE PLAN AND EXPLORATION LOCATIONS

1:1200



2 WEST ELEVATION

0 20 FEET

LEGEND

- BH-107 — SPT BOREHOLE
- TP-3 — TEST PIT
- 25/33 — MEDIUM DENSE TO DENSE, BROWN, MOIST, SILTY SAND (SM)
- 16/20 — STIFF TO VERY STIFF, GRAY, MOIST, CLAYEY SILT (ML)
- 35/46 — DENSE TO VERY DENSE, BROWN, MOIST, SILTY SANDY GRAVEL (GW)
- STANDARD PENETRATION TEST (SPT) VALUE (UNCORRECTED)

DATE: 01/01/2015	SOIL NAIL WALL - REFERENCE MANUAL EXAMPLE PLANS	SUBSURFACE DATA SHEET 10 OF 10

Appendix E: Soil Nail Wall Procedural Specifications

Procedural-based construction specifications are those in which all details of design, construction materials, and methods are specified in the contract documents.

When preparing the procedural-based specification, Part 1 – General may not be necessary if the Standard Specifications for the Construction of Roads and Bridges on Federal Highway Projects, FP-14 (FHWA, 2014) are used. If the soil nail wall specification is part of a much larger set of specifications, sections of this specification should be moved accordingly into appropriate sections to avoid conflicting information and redundancy. FP-14 contains additional specification language pertaining to soil nail walls in Sections 259 and 722.

Part 1 – General

1.1 Description

This work consists of constructing a permanent soil nail wall as specified herein and as shown on the Contract Drawings *(also referred to as Contract Plans)*. The Contractor shall furnish all labor, materials, and equipment required to complete the work.

The work shall be performed in accordance with the Contract Plans, the required Contractor submittals, and these Specifications. The work consists of the following elements: excavating in staged lifts; drilling soil nail drill holes to the diameter and length required; grouting of soil nails; supplying and installing the specified drainage features; supplying and installing bearing plates, washers, nuts, and other required hardware and miscellaneous materials; and constructing the initial and final soil nail wall facing.

Commentary: *The type of final facing will depend on the project conditions and may include reinforced shotcrete, cast-in-place concrete, and/or precast concrete panels.*

1.2 Definitions

Contractor: The specialty firm who is responsible for construction of the soil nail wall.

Engineer: The Owner's project engineer, project manager, or other representative.

Inspector: The Owner's field representative on the project site.

Owner: The agency responsible for the project.

Contract Drawings *[or Contract Drawings]*: Drawings developed by the Engineer and provided by the Owner for bidding purposes.

Project Manager: An employee of the Contractor supervising the work and who has a minimum of three years of experience with soil nail projects of similar size and scope.

1.3 Materials

1.3.1 Facing

Facing materials shall conform to the following sections and subsections.

1.3.1.a Cast-in-place Concrete

Cast-in-place (CIP) concrete shall meet the requirements of Section 8 of the AASHTO LRFD Bridge Construction Specifications.

Commentary: *Suitable ACI specifications or equivalent may be used instead.*

1.3.1.b Reinforcing Steel

Reinforcing steel shall meet the requirements of Section 9 of the AASHTO LRFD Bridge Construction Specifications.

1.3.1.c Initial Shotcrete

All materials, methods, and control procedures for initial shotcrete shall be submitted to the Owner's Engineer for review and approval.

1.3.1.d Final Shotcrete

Final shotcrete shall meet the requirements of Section 24, "Pneumatically Applied Mortar" of the AASHTO LRFD Bridge Construction Specifications.

1.3.1.e Architectural Surface Finishes

Architectural surface finishes may include textured surfaces or a surface finish with color/stain application.

1.3.2 Soil Nails

1.3.2.a Soil Nail Solid Bar (Tendon)

Tendons shall meet the requirements of AASHTO M31/ASTM A615 for Grade 60 or 75 steel bars (Grade 420 or 520 in SI Units), and ASTM A722 for Grade 150 steel (Grade 1035 in SI Units). Tendons shall be continuous without splices or welds, new, straight, undamaged, bare, epoxy coated, galvanized, or encapsulated as shown on the Contract Drawings. The length of the threaded portion of the bar at the wall anchorage shall be as needed to allow proper attachment of the bearing plate and nut. If threads are cut into a tendon, the contractor shall verify that the bar meets the minimum capacity required at the threaded section.

1.3.2.b Bar Couplers

Bar couplers shall develop the full nominal tensile capacity of the tendon as certified by the manufacturer.

1.3.2.c Fusion-Bonded Epoxy Coating

Fusion-bonded epoxy coating shall meet the requirements of ASTM A775 and have a minimum thickness of 12 mils (12 thousandths of an inch) up to a maximum of 17 mils as applied electrostatically. Bend test requirements are waived. The 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.

1.3.2.d Zinc Coating

Zinc galvanized coating shall meet the requirements of Article 11.10.6.4.2a (AASHTO 2014 or current edition) and have a minimum of 2.0 oz/ft^2 or 3.4 mil thickness applied in accordance with ASTM A123 for bars and structural steel shapes, and ASTM A153 for nuts, plates, and other hardware. *[ASTM A767/A767M can also be used as a reference.]*

1.3.2.e Encapsulation

Bar encapsulation shall be a minimum 0.04-in. thick, corrugated, HDPE tube conforming to AASHTO M252, or corrugated PVC tube conforming to ASTM D1784, Class 13464 B filled with neat cement grout.

1.3.3 Other Soil Nail Components

1.3.3.a Centralizers

Centralizers shall be manufactured from Schedule 20 or 40 PVC pipe or other materials not detrimental to the soil nail steel bar. Wood shall not be used. Centralizers shall be securely attached to the tendon and shall be sized to allow: (i) positioning of the soil nail bar within 1 in. of the center of the drill hole; (ii) tremie pipe insertion to the bottom of the drill hole; and (iii) grout to freely flow up the drill hole. They shall be installed at regular intervals not to exceed 10 ft along the length of the nail and a distance of 1.5 ft from each end of the nail.

1.3.3.b Grout

Grout shall be neat cement or sand/cement mixture with a minimum 3-day compressive strength of 1,500 psi and a minimum 28-day compressive strength of 3,000 psi, meeting the requirements of AASHTO T106/ASTM C109. The specific gravity of the freshly prepared neat cement grout shall range between 1.8 and 1.9.

1.3.3.c Sand

Sand for grout and/or shotcrete shall meet the requirements of AASHTO M6/ASTM C33.

1.3.3.d Portland Cement

Portland cement for grout and/or shotcrete shall meet the requirements of AASHTO M85/ASTM C150, Types I, II, III, V, or Type I/II and shall be the product of one manufacturer. If the brand or type of cement is changed during the project, additional grout mix tests shall be conducted to ensure consistency of quality and performance in situ.

1.3.3.e Admixtures

If admixtures are used, they shall meet the requirements of AASHTO M194/ASTM C494. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations. Accelerators shall not be permitted. Expansive admixtures shall not be permitted except where the grout is used as part of corrosion protecting encapsulation.

1.3.3.f Film Protection

Polyethylene film for moisture loss control shall meet the requirements of AASHTO M171.

1.3.4 Connection Components

1.3.4.a Bearing Plates

Bearing plates shall meet the requirements of AASHTO M183/ASTM A36 or ASTM A572 Grade 50 (Grade 350 in SI Units).

1.3.4.b Nuts

Nuts shall meet the requirements of AASHTO M291/ASTM A563, Grade B, hexagonal, and fitted with beveled washer or spherical seat to provide uniform bearing.

1.3.4.c Shear Connectors

Shear connectors of the soil nail head shall consist of headed-studs or anchor bolts.

1.3.5 Welded-Wire Mesh

Welded wire mesh (WWM) shall meet the requirements of AASHTO M55/ASTM A185 or A497.

1.3.6 Reinforcing Steel

Reinforcing steel shall meet the requirements of AASHTO M31/ASTM A615, Grade 60 (Grade 420 in SI Units), deformed.

1.3.7 Geocomposite Strip Drain

Geocomposite strip drain shall be manufactured with a drainage core (e.g., geonet) and a filtration geotextile attached to or encapsulating the core. Drainage core shall be manufactured from long-chain synthetic polymers composed of at least 85 percent by mass of polypropylenes, polyester, polyamine, polyvinyl chloride, polyolefin, or polystyrene, and have a minimum compressive strength of 40 psi when tested in accordance with ASTM D1621 Procedure A. The drainage core shall have a minimum flow rate of 0.1 gallons per second per foot of strip width under a gradient of 1.0 tested in accordance with ASTM D4716.

1.3.8 Underdrain Elements

1.3.8.a Pipe

Underdrain and perforated pipe shall meet the requirements of ASTM D1785 Schedule 40 PVC solid and perforated wall; cell classification 12454-B or 12454-C, wall thickness SDR 35, with solvent weld or elastomeric joints.

1.3.8.b Fittings

Fittings for underdrain and perforated pipe shall meet the requirements of ASTM D3034, Cell classification 12454-B or 12454-C, wall thickness SDR 35, with solvent or elastomeric joints.

1.4 Contractor Experience

The soil nailing contractor shall meet the following experience requirements:

1. Have completed at least three permanent soil nail wall projects during the past three years totaling at least 10,000 ft^2 of soil nail wall face area and at least 500 permanent soil nails.

2. Provide a registered Professional Engineer with experience in the design of permanent soil nail walls with at least three completed projects over the past three years.

3. Provide on-site supervisors and drill operators with experience installing permanent soil nail walls on at least three projects over the past three years.

4. Submit a brief description of at least three projects, including the Owner's name, address, and current phone number; location of project; project contract value; and scheduled completion date and completion date for the project.

1.5 Submittals

1.5.1 Personnel Experience

At least 45 calendar days before starting soil nail work, submit names of the Professional Engineer, on-site supervisors, and drill operators assigned to the project, and a summary of each individual's experience. Only those individuals designated as meeting the experience 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 7 calendar days

following receipt of the submission. Work shall not be started nor materials ordered until the Contractor's qualifications have been approved by the Owner's Engineer. The Owner's Engineer may suspend the work if the Contractor substitutes unapproved personnel for approved personnel during construction. If work is suspended due to the substitution of unapproved personnel, the Contractor shall be fully liable for all additional costs resulting from the suspension of work, and no adjustment in contract time resulting from the suspension of the work shall be allowed.

1.5.2 Site Surveys

The Contractor shall be responsible for providing the necessary survey and alignment control during the excavation for each lift, locating drill holes and verifying limits of the soil nail wall installation.

1.5.3 Construction Plan

At least 30 calendar days before starting soil nail work, the Contractor shall submit a Construction Plan to the Owner's Engineer that includes the following:

1. Project start date and proposed detailed wall construction sequence.

2. Drilling and grouting methods and equipment, including the drill hole diameter proposed to achieve the specified nominal pullout resistance values shown on the Contract Drawings.

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. For neat cement grout include specific gravity test results of the fresh grout used for compressive testing.

4. Nail grout placement procedures and equipment.

5. Temporary shotcrete materials and methods, including mix and anticipated strength meeting the minimum compressive strengths.

6. Soil nail testing methods and equipment setup.

7. Identification number and certified calibration records for each test jack, pressure gauge, dial 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 calendar days of submittal.

8. Manufacturer Certificates of Compliance for materials including: the tendon ultimate strength, welded wire mesh steel, rebar steel, couplers, bearing plates, epoxy coating, and encapsulation.

9. Product information, details, and cut sheets of products to be used in this project, including, but limited to: geocomposite strip drains, centralizers, couplers, and geotextiles.

10. Procedures and material for repairing corrosion protection coatings in the field and for applying epoxy finish coatings on end hardware.

11. Shop drawings that shall include all details, dimensions, quantities, ground profiles, and cross-sections necessary to construct the wall. The Contractor shall verify the limits of the wall and ground survey data before preparing the shop drawings. The working drawings shall be prepared to the Owner's standards.

The Owner's Engineer will approve or reject the Contractor's submittals within 14 calendar days of the receipt of the complete submission. Approval of the Construction Plans does not relieve the Contractor of his responsibility for the successful completion of the work. The Contractor shall not begin construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Owner's Engineer.

1.6 Storage and Handling

Soil nail bars shall be stored and handled in a manner to avoid damage or corrosion. Soil nail bars exhibiting abrasions, cuts, welds, weld splatter, corrosion, or pitting shall be replaced. Bars exhibiting damage to encapsulation or epoxy coating shall be repaired or replaced at no additional cost. Repaired epoxy coating areas shall have a minimum 12-mil. thick coating. Damaged galvanization shall be repaired by coating the damaged area with a field grade, zinc-rich paint.

1.7 Execution

1.7.1 Excavation

The height of the exposed unsupported final excavation face cut shall be established by the Contractor and 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. Excavation to the final wall

excavation line and shotcrete application shall be completed in the same work shift, unless otherwise approved by the Owner's Engineer.

Excavation of the next-lower lift shall not proceed until soil nail installation, initial 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 their specified 3-day compressive strength, but not lesser than 1,500 psi before excavating the next underlying lift.

1.7.2 Soil Nail Installation

The soil nail length and drill hole diameter necessary to develop the load capacity and to satisfy the acceptance criteria for the design load required shall be provided, but not less than the lengths or diameters shown in the Contract Drawings.

Drill holes for the soil nails shall be drilled at the locations, elevations, orientations, and lengths shown on the Contract Drawings. The drilling equipment and methods shall be selected to be suitable for the ground conditions and in accordance with the accepted installation methods submitted by the Contractor. If caving ground is encountered, cased drilling methods or other suitable means shall be used to support the sides of the drill holes. Soil nail bars shall be provided as shown in the Contract Drawings.

Centralizers shall be provided and sized to position the soil nail bars to within 1 in. of the center of the drill hole. Centralizers shall be positioned as shown on the Contract Drawings so that their maximum center-to-center spacing does not exceed 10 ft, and shall be located to within 1.5 ft from each end of the nail bar.

1.7.3 Grouting

The drill hole shall be grouted after installation of the soil nail bar and within 2 hours of completion of drilling. The grout shall be injected at the lowest point of each drill hole through a grout tube, casing, hollow-stem auger, or drill rods. The outlet end of the conduit shall deliver grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. The drill hole shall be filled in one continuous operation. Cold joints in the grout column shall not be allowed except at the top of the test bond length of proof-tested production nails. The space above the bottom elevation of the inclined drill hole opening, called a "bird's beak" due to its shape, shall be filled up with additional grout after a temporary cover is placed in front of the drill hole, or filled with shotcrete.

1.8 Soil Nail Testing

1.8.1 General

The Contractor shall perform both verification and proof testing of designated test soil nails. Verification tests on sacrificial test nails shall be conducted at locations shown on the Contract Drawings. Proof tests on production nails shall be conducted at locations selected by the Owner's 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 their specified 3-day compressive strength, or at least 1,500 psi. Do not apply loads greater than 80 percent of the minimum guaranteed ultimate tensile strength of the tendon for Grade 150 bars, or 90 percent of the yield strength of the tendon for Grade 60 or 75 bars.

1.8.2 Equipment

Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, load cell, and a reaction frame. The pressure gauge shall be graduated in 50 psi increments or less. The dial gauges shall be supported by a frame that is independent from the jacking and wall. Nail head movement shall be measured with a minimum of 2 dial gauges capable of measuring to 0.001 in.

1.8.3 Verification Testing

The Contractor shall perform a number of verification tests on sacrificial soil nails as established in the Contract Drawings. Verification testing shall be conducted prior to installation of production soil nails on sacrificial soil nails to confirm the appropriateness of the Contractor's drilling and installation methods, and to verify the required nail pullout resistance.

1.8.3.a Methods

The verification tests shall be conducted on nails of the same design and constructed with the same construction methods to be used on production nails.

Soils nails in verification tests shall have both a bonded length and an unbonded length. The nail bar shall not be grouted along the unbonded length. Fully grouted soil nails shall not be used for load testing. The unbonded length shall be at least 3 ft within the drill hole.

The bonded length in verification tests, $L_{B\ VT}$, shall be selected as follows:

(1) Select $L_{B\ VT\ max}$

For Grades 60 and 75 and other mild steel in accordance with ASTM A615, the maximum length, $L_{B\ VT\ max}$, is defined as:

$$L_{B\ VT\ max} = \frac{A_t \times f_y \times C_{RTY}}{r_{PO}}$$

Equation E.1: Maximum bond length in verification tests (Grades 60 and 75).

For Grade 150 and other high-strength steel in accordance with ASTM A722, the maximum length is defined as:

$$L_{B\ VT\ max} = \frac{A_t \times f_u \times C_{RTU}}{r_{PO}}$$

Equation E.2: Maximum bond length in verification tests (Grade 150).

Where:

C_{RTY} = reduction coefficient for mild-grade steel = 0.9

C_{RTU} = reduction coefficient for high-strength steel = 0.8

A_t = cross-sectional area of the test tendon

f_y = nominal yield resistance of the test tendon (mild steel)

f_u = minimum ultimate tensile strength of the test tendon (high-strength steel)

r_{PO} = nominal pullout resistance (per unit length) of soil nail = $\pi \times q_u \times D_{DH}$

q_u = bond strength

D_{DH} = drill hole diameter

(2) If $L_{B\ VT\ max} > 10$ ft

Select $L_{B\ VT}$ to be 10 ft $\leq L_{B\ VT} \leq L_{B\ VT\ max}$.

(3) If $L_{B\ VT\ max} < 10$

Select $L_{B\ VT} = 10$ ft, increase the test tendon size as needed, and recalculate $L_{B\ VT\ max}$ until $L_{B\ VT\ max} > 10$ ft.

The maximum load during the verification test is defined as the Verification Test Load (VTL), and shall be calculated as follows:

$$VTL = L_{B\,VT} \times r_{PO}$$

Equation E.3: Maximum load in verification tests.

1.8.3.b Schedule

Verification tests shall be conducted by incrementally loading the verification test nails to pullout or a maximum test load VTL in accordance with the loading schedule presented below.

The Contractor shall record soil nail movements at each load increment and at each time step. Dial gauges shall be set to "zero" after the alignment (AL) load has been applied.

Load levels beyond VTL are optional. Following application of VTL, the maximum load shall be reduced to the AL and the permanent set shall be recorded. Each load increment shall be held for at least 10 minutes.

The verification test nail shall be monitored for creep at a load of 0.75 VTL. Nail movements shall be measured and recorded during the creep portion of the test in increments of 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. The load shall be maintained during the creep test to within 2 percent of the intended load by use of a load cell. Repump jack as needed to maintain load during hold times.

Table E.1: Verification Test Loading Schedule

Load	Hold Time (minutes)[2]
AL[1]	1
0.13 VTL	10 (record soil nail movement at 1, 2, 5, 10)
0.25 VTL	10 (record soil nail movement at 1, 2, 5, 10)
0.38 VTL	10 (record soil nail movement at 1, 2, 5, 10)
0.50 VTL	10 (record soil nail movement at 1, 2, 5, 10)
0.63 VTL	10 (record soil nail movement at 1, 2, 5, 10)
0.75 VTL (Creep Test)	60 (record at 1, 2, 5, 10, 20, 30, 50, 60)
0.88 VTL	10
1.00 VTL	10
AL	1[3]

Notes:
(1) Alignment load shall be 0.025 VTL.
(2) Measure soil nail movement after each load increment has been achieved and at each time step indicated above.
(3) Record permanent soil nail movement.

Stepped unloading is optional. Consider between 1 to 7 intermediate steps in stepped unloading. If stepped unloading is conducted, hold each load step until the readings in gauges are stable.

1.8.4 Proof Testing

Successful proof testing shall be demonstrated on at least 5 percent of production soil nails in each nail row or a minimum of one per row. *[The quantity of proof tests must be established by the Owner prior to bidding.]* The Owner's Engineer shall determine the locations and number of proof tests prior to nail installation in each row. Verification tests shall not be counted towards the minimum of 5 percent of production nails.

1.8.4.a Methods

Proof test nails shall have a bonded length and a temporary unbonded length. The temporary unbonded length in proof tests shall be at least 3 ft within the drill hole.

The bonded length in proof tests, $L_{B\,PT}$, shall be selected such that $L_{B\,PT}$ is 10 ft or $L_{B\,PT\,max}$, whichever is smaller.

For Grade 75 and other mild steel in accordance with ASTM A615, the maximum length, $L_{B\,PT\,max}$, is defined as:

$$L_{B\,PT\,max} = \frac{A_t \times f_y \times C_{RTY}}{r_{PO} \times 0.75}$$

Equation E.4: Maximum bond length in proof tests (Grades 60 and 75).

For Grade 150 and other high strength steel in accordance with ASTM A722, the maximum length, $L_{B\,PT\,max}$, is defined as:

$$L_{B\,PT\,max} = \frac{A_t \times f_u \times C_{RTU}}{r_{PO} \times 0.75}$$

Equation E.5: Maximum bond length in proof tests (Grade 150).

Production proof test nails that are shorter than 12 ft may be tested with less than the minimum 10-ft bond length. Fully grouted test nails shall not be proof tested. The maximum load in proof tests is defined as the Proof Test Load (PTL) and shall be calculated as follows:

$$PTL = L_{B\,PT} \times r_{PO} \times 0.75$$

Equation E.6: Maximum load in proof tests.

1.8.4.b Schedule

Proof tests shall be conducted by incrementally loading the proof test nail according to the test loading schedule shown below.

Table E.2: Proof Test Loading Schedule

Load	Hold Time (minutes)[3]
AL[1]	1
0.17 PTL	until movement stabilizes
0.33 PTL	until movement stabilizes
0.50 PTL	until movement stabilizes
0.67 PTL	until movement stabilizes
0.83 PTL	until movement stabilizes
1.0 PTL (Creep Test)[2]	10 (record soil movement at 1, 2, 5, 6, 10)
AL	1

Notes:
(1) AL = alignment load shall be AL ≤ 0.025 PTL.
(2) If the nail movement measured between 1 and 10 minutes exceeds 0.04 in., PTL shall be maintained for 50 more minutes and movements shall be recorded at 20, 30, 50, and 60 minutes. Record permanent soil movement.
(3) Measure times after the target load has been attained in each increment.
(4) If the soils reinforced with nails are relatively susceptible to deformation or creep, it is recommended to hold each load increment for 10 minutes and to record the soil nail movement at 1, 2, 5, and 10 minutes.

Hold each load increment until stabilized and record soil nail movements at each load increment and time interval shown in the schedule. All load increments shall be maintained to within 5 percent of the intended load. Repump jack as needed to maintain load during hold times.

Dial gauges shall be set to "zero" after the alignment load has been applied.

Perform creep tests at 1.00 PTL. Creep testing shall start as soon as PTL is applied. Soil nail movement shall be measured and recorded at 1 minute, 2, 3, 5, 6 and 10 minutes. If the nail movement measured between 1 minute and 10 minutes exceeds 0.04 in., the maximum test load shall be maintained for 50 additional minutes, and soil nail movements shall be recorded at 20 minutes, 30, 50, and 60 minutes.

1.9 Acceptance Criteria of Test Soil Nails

1.9.1 Verification Tests

Considering that pullout is defined as the load at which attempts to further increase the test load increments simply result in continued movement of the tested nail, a test nail shall be considered acceptable when all of the following criteria are met.

(1) The total creep movement is less than 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) The total movement (Δ_{VTL}) measured at VTL exceeds 80 percent of the theoretical elastic elongation of the unbonded length (L_{UB}) of the test nail, or:

$$\Delta_{VTL} > 0.8 \frac{VTL\, L_{UB}}{E\, A_t}$$

Equation E.7: Theoretical elastic elongation of the unbonded length in verification tests.

Where: E = Young's modulus of steel (29,000 ksi)

(3) Pullout does not occur before achieving 1.0 × VTL.

1.9.2 Proof Tests

(1) The creep movement between the 1- and the 10-minute readings is less than 0.04 in.

(2) In cases when the creep movement between the 1- and the 10-minute readings is greater than 0.04 in., the creep movement between the 6- and the 60-minute readings is less than 0.08 in., and the creep rate is linear or decreasing throughout the creep test load hold period.

(3) The total soil nail movement (ΔPTL) measured at PTL exceeds 80 percent of the theoretical elastic elongation of the unbonded length, or:

$$\Delta_{PTL} > 0.8 \frac{PTL\, L_{UB}}{E\, A_t}$$

Equation E.8: Theoretical elastic elongation of the unbonded length proof tests.

Where: E = Young's modulus of steel (29,000 ksi)

(4) Pullout does not occur before the test load is 1.0 × PTL.

(5) The temporary unbonded test length in proof tests is successfully maintained for subsequent satisfactory grouting. If the unbonded test length of production proof test nails cannot be satisfactorily grouted after 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.

1.10 Rejection of Test Soil Nails

1.10.1 Verification Test Soil Nails

The Owner's 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 for review by the Owner's Engineer and shall install replacement verification test nails.

1.10.2 Proof Test Soil Nails

For proof test nails, the Owner's Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test soil nail and the adjacent passing proof test nail. Alternatively, the Owner's Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient nominal pullout resistance.

1.11 Wall Drainage Network

1.11.1 General

All elements of the soil nail wall drainage network shall be installed and secured as shown on the Contract Drawings. The drainage network shall consist of geocomposite drain strips, PVC connection pipes, soil nail wall footing drains, and weepholes, as shown on the Contract Drawings. The drainage network shall provide a continuous and unrestricted flow path to discharge the water collected behind the wall face. Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting.

1.11.2 Geocomposite Drain Strips

Geocomposite drain strips shall be centered between the columns of soil nails, as shown on the Contract Drawings. Drain strips shall be at least 12 in. wide and placed with the geotextile side against the ground. Strips shall be secured to the excavation face.

Contamination of the geotextile with shotcrete shall be prevented. Drain strips shall be vertically continuous.

1.11.3 Footing Drains

Footing drains shall be installed at the bottom of the wall, as shown on the Contract Drawings. The drainage geotextile shall envelope the footing drain aggregate and pipe and shall conform to the dimensions of the trench. The drainage geotextile shall overlap on top of the drainage aggregate as shown on the Contract Drawings. Damaged or defective drainage geotextile shall be repaired or replaced.

1.12 Shotcrete

1.12.1 General

Shotcrete for initial and final facing shall be provided as required in the Specifications. Where shotcrete is used to complete the top ungrouted zone of the soil nail drill hole near the face, the nozzle shall be positioned into the mouth of the drill hole to completely fill the void.

1.12.2 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 Contract Drawings.

1.12.3 Attachment of Nail Head Bearing Plate and Nut

A bearing plate, washers, and nut shall be attached to each nail head as shown on the Contract Drawings. While the shotcrete construction facing is still plastic and before its initial set, the plate shall be uniformly seated on the shotcrete by hand-wrench tightening the nut. Where uniform contact between the plate and the shotcrete cannot be provided, the plate shall be set in a bed of grout. After grout has set for 24 hours, re-tighten the nut by hand with a wrench. The bearing plates with headed studs shall be located within the tolerances shown on the Contract Drawings.

1.12.4 Shotcrete Facing Tolerances

Construction tolerances for the shotcrete facing from plan location and plan dimensions shall be as follows:

Table E.3: Construction Tolerances for Shotcrete Facing

Item	Tolerance
Horizontal location of welded wire mesh, reinforcing bars, and headed studs	3/8 in.
Location of headed-studs on bearing plate	1/4 in.
Spacing between reinforcing bars	1 in.
Reinforcing lap	1 in.
Thickness of shotcrete, if troweled or screeded	9/16 in. [approximation of 0.6 in.]
Thickness of shotcrete, if left as shot	1-1/8 in. [approximation of 1.2 in.]
Planeness of finish face surface, gap under 10-ft straightedge, if troweled or screeded	9/16 in. [approximation of 0.6 in.]
Planeness of finish face surface, gap under 10-ft straightedge, if left as shot	1-1/8 in. [approximation of 1.2 in.]
Nail head bearing plate deviation from parallel to wall face:	10 degrees

1.13 Reinforcing Steel

The Contractor shall submit all order lists and reinforcement bending diagrams to the Owner's Engineer, and shall fabricate reinforcing steel, ship and protect material, place, fasten, and splice reinforcing steel as required by the Contract Drawings.

1.14 Structural Concrete

The Contractor shall design the concrete mix, store, handle, batch, and mix material and deliver concrete, provide quality control, and construct concrete facing to meet the concrete resistance and characteristics contained in the Specifications.

1.15 Architectural Surface Finishes

[If applicable] Textured form liners shall be furnished, form liners installed, and a surface finish (color/stain application) applied that will duplicate the architectural surface finish shown on the Contract Drawings. The Contractor shall submit detailed drawings of the form liner for approval by the Owner's Engineer at least 7 calendar days before form liner work begins. Before production work begins, a 3-ft high, 10-ft long test panel shall be constructed 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. One test panel per wall finish type shall be furnished.

1.16 Backfilling Behind Upper Cantilever of Wall Facing

If backfilling is required behind an extension of the wall facing at the top of a soil nail wall, compaction of the soil backfill within 3 ft of the wall extension shall be performed using light mechanical tampers.

1.17 Corrosion Protection

Soil nails and soil nail head components shall be protected against corrosion consistent with the ground and groundwater conditions at the site and the minimum class of corrosion protection shown on Contract Drawings. *[Owner must perform corrosivity testing on representative soil samples as described in Chapter 4 to determine requirements for the minimum class of corrosion protection of the soil nails, before the preparation of bid documents. The Owner should always provide the results of corrosivity testing to the Contractor. The Contractor may also elect to provide a higher corrosion class than specified as a substitution.]*

1.18 Protection of Adjacent Structures

Structures located within a horizontal distance equal to ____ shall be monitored for vertical and horizontal movement in a manner approved by the Engineer within an accuracy of 0.01 in. Monitoring of adjacent structures shall be done by an independent party working for the Contractor.

A Monitoring Plan, including the locations of measurement points and the frequency of recording measurements, shall be submitted to the Engineer as part of the Construction Plan. Monitoring shall begin with a base-line measurement recorded no less than 10 calendar days prior to construction of the wall.

In addition to monitoring for movement, the condition of the adjacent structure, including cracks and crack widths, before and after construction of the wall, shall be documented by visual inspection, photographs, and/or video. Structures owned by Owner shall be monitored for movement, but need not be monitored for condition unless called for on the Contract Drawings. Thresholds of movement for immediate notification and work stoppage shall be as defined in the Contract Drawings.

As soon as the movements measured exceed thresholds established for adjacent structures, the Contractor shall stop construction, notify the Engineer, and take any immediate remedial measures required to prevent damage to the adjacent structures. The Contractor and the Engineer shall then review the current installation procedures. If revisions to the installation

procedures are deemed necessary, the Contractor shall submit a revised installation plan for approval by the Engineer before resuming work.

1.19 Acceptance

Material for soil nail retaining walls will be accepted based on the manufacturer's production certification or from production records.

Construction of soil nail retaining walls will be accepted based on visual inspection and the examination of relevant production testing records by the Owner's Engineer.

Part 2 – Measurement and Payment Soil Nails

2.1 Soil Nails

Production soil nails shall be measured by the linear foot. The length to be paid will be the length measured along the soil nail bar centerline from the back face of shotcrete to the bottom tip end of nail bar as shown on the Contract Drawings. No separate measurement will be made for proof test nails, which shall be considered incidental to production nail installation.

2.2 Soil Nails for Verification Test

Verification test soil nails shall be measured by each test meeting the acceptance criteria of Section 1.8 of this specification. Rejected verification test soil nails or additional verification test nails installed to verify alternative nail installation methods proposed by the Contractor will not be measured. No payment shall be paid for temporary soil nails.

2.3 Excavation

Excavation for the soil nail wall shall be measured as the theoretical plan volume in cubic yards within the structure excavation limits shown on the Contract Drawings. This will be the excavation volume within the zone measured from top to bottom of shotcrete wall facing and extending out 6 ft horizontally in front of the wall final excavation line shown in the Contract Drawings. 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, including hauling, will not be a separate wall pay item but will be measured and paid as part of the general roadway excavation.

2.4 Wall Facing

The wall facing shall be measured by the square foot. Measurement will be made on the vertical plane of the front face as accepted in the final work. No measurement or payment will be made for: (i) additional shotcrete or CIP concrete needed to fill voids created by irregularities in the cut face; (ii) excavation overbreak or inadvertent excavation made beyond the excavation line shown on the Contract Drawings when creating the back of the wall; or (iii) failure to construct the facing to the specified line, 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 Contract Drawings and this Specification. The final pay quantity shall be the design quantity increased or decreased by any changes authorized by the Owner's Engineer.

2.5 Payment

2.5.1 General

The accepted quantities, measured as provided in Sections 2.1 through 2.4 of this specification, 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.

2.5.2 Pay Items

Payment will be made for the pay items listed in the table below.

Table E.4: Pay Items

Pay Item	Pay Unit
Production Soil Nails (No. _ Bar, Grade _)	Linear foot
Verification Test Soil Nails	Each
Excavation for Soil Nail Walls	Cubic yard
Soil Nail Wall Facing	Square foot

Commentary: *Depending on special conditions that may be expected in the project (e.g., additional test panels for different types of facing finishes over the wall alignment, potential*

need to add proof tests depending on encountered conditions, etc.), the Owner may consider including additional pay items.

Part 3 – Special Considerations for Hollow Bar Soil Nails

Hollow Bar Soil Nails shall follow the Specifications included above except for the following subsections which can be used in place of the subsections above as needed.

3.1 Hollow Bar Soil Nails (HBSNs)

HBSNs shall be fully threaded, hollow steel tubing used as the drilling steel, grout transfer medium, and the reinforcing element of the soil nail. Tendons shall have a yield tensile strength between 60 ksi and 90 ksi (410 MPa and 620 MPa in SI Units). Tendons shall be new, straight, undamaged, bare, epoxy coated, galvanized, or encapsulated as shown on the Contract Drawings. The length of the threaded portion of the bar at the wall anchorage shall be as needed to allow proper attachment of the bearing plate and nut. Hollow bars shall meet the following requirements:

1. The hollow bar shall be of a fine-grained structural steel.

2. The ductility of the steel or the uniform elongation without necking shall be $A_{gt} \geq 5\%$.

The minimum Charpy impact resistance shall be 40 Joules at -20°C. *[This is to ensure damage does not occur to the hollow bar during drilling.]*

3.2 Bar Couplers

Bar couplers shall develop the nominal tensile capacity of the hollow bar as certified by the manufacturer and shall have a seal or a similar mechanism to ensure minimum grout loss through them. Couplers shall have a means of transferring the percussive forces when drilling, and tensile/compressive cyclic-loading forces when in service. *[This requirement allows for greater percussive forces when drilling, and 2 million tension/compression cyclic loading with a cyclic stress range of 10 ksi.]*

3.3 Centralizers

Centralizers shall be manufactured from Schedule 20 or 40 PVC pipe or tube, steel, or other materials not detrimental to the soil nail steel bar. Wood shall not be used. Centralizers shall be securely attached to the hollow bar and shall be sized to allow: (a) positioning of the hollow bar within 1 in. of the center of the drill-hole; and (b) grout to freely flow up the drill hole during installation. Centralizers shall be installed at regular intervals not to exceed 10 ft

along the length of the nail, and a distance of 1.5 ft from the back of the bit and the end of the nail.

3.4 Grout

1. Grout shall be made of Portland cement conforming to AASHTO M85/ASTM C150 Type I, II, III, V or Type I/II, and shall be the product of one manufacturer. If the brand or type of cement is changed during the project, additional grout mix tests shall be conducted to ensure consistency of quality and performance in situ.

2. Admixtures are typically not required.

3. Fillers such as sand shall not be used. *[Aggregates in the grout may cause blockages in the drill bit.]*

Final grout shall have a minimum 3-day compressive strength of 1,500 psi and a minimum 28-day compressive strength of 3,000 psi, meeting the requirements of AASHTO T106/ASTM C109. The specific gravity of the final grout shall range between 1.8 and 1.9. The specific gravity of the grout used for drilling and installing the hollow bars may be lower than the final grout and range from 1.4 to 1.6. After the bar is installed to the desired depth, the final grout mixture shall be pumped through the hollow bar, and the nail is considered complete when the heavier mixture returns to the excavation face, signaling that the lighter drilling grout was flushed from the hole and that all drilling spoils have also been removed.

3.5 Drill Bits

Drill bits shall be selected based on the subsurface information on site and the minimum grout column required in the design. Bits require a minimum of two ports to allow grout to exit the system. Additional ports (with smaller diameters) can be used to increase grout pressure and hole diameter.

3.6 Grout Mixer and Pump

A high shear colloidal mixer with separate holding tank and water and cement dosing system should be used to ensure continuous grouting independent from mixing. Pumps should have flow rates of at least 15 gal/minute for the smaller diameter bars, and 45 gal/minute for the larger diameter bars (2 in. and above) shall be provided. A minimum of 250 psi pressure capability for sand and gravel and 1,500 psi capability should be available in clays and silts. To record the grout volume and pressure, an automated monitoring system may be used.

Appendix F: Soil Nail Wall Performance-Based Specifications

Commentary: *Performance-based specifications should include any required outcome of the design to be completed by others than the Owner or their representatives. Performance-based specifications should serve as a set of criteria for the designer. These criteria can then be used to assess the outcome of the project. Performance-based specifications should be as specific as possible without directing the means and methods for the work to be completed.*

Performance-based specifications may reflect some amount of design performed by the Owner's Engineer and some amount performed by the Contractor. The Owner Agency provides preliminary Contract Drawings showing wall layout and also provides related design criteria and requirements, subsurface data, right-of-way limits, utility locations, site limitations, construction material and testing specifications, required submittals, and review requirements for the Contractor. The Contractor designs the soil nail wall and selects the construction means and methods and equipment. Once the contract is awarded, the selected Contractor prepares a detailed design and drawings, and submits them to the Owner's Engineer for review.

Part 1 – General

1.1 Description

This work consists of constructing a permanent soil nail wall as specified herein and as shown on the Contract Drawings *[also referred to as Contract Plans]*. The Contractor shall furnish all labor, materials, and equipment required to complete the work. The Contractor shall design the soil nail wall to meet the performance requirements specified herein or shown on the Contract Drawings. Temporary soil nail retaining walls are not included in this specification.

1.2 Definitions

Contractor: The specialty firm who is responsible for performing the soil nail wall design and construction.

Design Drawings *[also referred to as Contract Plans]*: Drawings prepared by the Contractor's Design Engineer and submitted by the Contractor to include the detailed soil nail design.

Design Engineer: The Professional Engineer employed or hired by the Contractor who designs the soil nail structure. This person must meet the experience requirements in Section 1.5.

Engineer: The Owner's project engineer, project manager, or other representative.

Inspector: The Owner's field representative on the project site.

Owner: The Agency responsible for the project.

Contract Drawings *[or Contract Plans]***:** Drawings developed by the Engineer and provided by the Owner for bidding purposes.

Project Manager: An employee of the Contractor supervising the work and who has a minimum of three years of experience with soil nail projects of similar size and scope.

1.3 References

1.3.1 Codes and Standards

- Standard Specifications for the Construction of Roads and Bridges on Federal Highway Projects, FP-14 (FHWA, 2014)
- AASHTO Standard Specification for Highway Bridges, current Edition, including current interim specifications *(Note that soil nail walls are not specifically included in this specification. It is used for reference because it covers other elements of the wall and structure.)*
- Geotechnical Engineering Circular No. 7, Soil Nail Walls - Reference Manual

1.3.2 Related Codes and Standards

Commentary: *Engineer will select appropriate codes and specify all related standards and specifications for the project.*

1.3.3 Available Information

Available information developed by the Owner, or the Owner's duly authorized representative, including the following items:

1. Plan Set(s) ____, Project No. _____, prepared by _____, dated _____. The Contract Drawings include the preliminary design developed for the project, as well as plan view, profile, and typical cross sections for the proposed walls.

Commentary: *The Owner should provide preliminary design information for the Contractor during the pre-bid process. The complete list of items will vary according to project and local procedures, but should include:*

1. Contract Drawings showing the design loadings, and wall layout and dimensions.

2. *Design criteria and requirements, such as design loads and maximum allowable displacements.*

3. *Geotechnical Engineering Report (s)_____, prepared by _____, dated _____, must be included or referenced in the bid documents, containing the results of exploratory borings, observation pits, or other site investigation data, including soil corrosion tests, obtained in the vicinity of the proposed walls. The Owner may elect to include the results of global stability and deformation analyses, or require that these aspects be entirely handled by the Design Engineer. If the Geotechnical Report is not included in the bid documents, there must be a definition on who performs global stability, deformation analyses, and selection of corrosion protection. If the Geotechnical Engineering Report does not include a selection of a Corrosion Protection Class, the bid documents must identify who will select the Corrosion Protection Class.*

The subsurface conditions expected can significantly impact the Contractor's choice of procedures, methods, equipment, the biding process, and contract administration. A geotechnical summary included in the contract special provisions can be advantageous toward achieving a successful contract. The purpose of the summary is to alert bidders of the subsurface conditions and hopefully reduce the potential for differing site condition construction claims and disputes. By including the summary in the contract special provisions, it becomes a legal part of the contract documents.

1.4 Project Site Survey

Before bidding the work, the Contractor shall review the available subsurface information and visit the project site to assess the site geometry, equipment access conditions, and locations of existing structures and above-ground facilities. The Contractor, at their own expense, may perform an additional subsurface investigation to aid in preparation of their bid. Requests for access to the site for performing subsurface investigations must be made in writing to _____ no later than _____.

Commentary: *Allowing the Contractor to perform his own subsurface investigation with sufficient time may be advantageous on projects with challenging subsurface conditions. Some contractors perform their own investigations to supplement data provided by the Owner and aid in refining their bids, as long as the Contractor is allowed onto the site during bidding and sufficient time is allowed. However, allowing the Contractor access to the site prior to bidding for such explorations will not be suitable for all projects, particularly smaller projects or those on a tight time schedule. Ample consideration should also be given to the suitability of the site for a soil nail wall as discussed in this manual,*

including factors such as subsurface conditions, site access, and project schedule, among others.

When specifying a soil nail wall as opposed to listing soil nailing as an option among a list of optional wall types, it is implied that the Owner has determined that soil nail wall construction is feasible. If the Owner prefers not to prescribe a soil nail wall, then a general specification for a design-build earth-retention system should list soil nailing as an allowed option, subject to the Contractor's sole assessment of feasibility. The soil nail performance specification is then invoked in the instance the Contractor selects soil nailing as the preferred option.

The Contractor is responsible for field locating and verifying the locations of all utilities shown on the Contract Drawings prior to starting the work. The Contractor shall notify the Engineer of any utility locations different from those shown on the Contract Drawings that may require relocation of foundation elements or modification to the structure design. Subject to the Engineer's approval, additional cost to the Contractor due to relocation and/or design modifications resulting from incorrect utility locations on the Contract Drawings will be paid as Extra Work.

The Contractor shall be responsible for providing the necessary survey and alignment control during the excavation for each lift, locating drill holes, and verifying limits of the soil nail wall installation. Verify the limits of the structures and ground survey data before preparing the detailed drawings.

Commentary: *Many state DOTs have standard specifications or procedures for handling utilities on a project, so this section may not apply or can be modified to include reference to the applicable state DOT specification/procedure. The location of active and abandoned underground utilities can have a significant impact on the installation of soil nails.*

1.5 Contractor Experience

The soil nailing contractor shall meet the following experience requirements:

1. Have completed at least three permanent soil nail wall projects during the past three years totaling at least 10,000 ft^2 of soil nail wall face area and at least 500 permanent soil nails.
2. Provide a registered Professional Engineer with experience in the design of permanent soil nail walls for at least three completed projects over the past three years.

3. Provide on-site supervisors and drill operators with experience installing permanent soil nail walls on at least three projects over the past three years.

4. Submit a brief description of at least three projects, including the Owner Agency's name, address, and current phone number; location of project; project contract value; and scheduled completion date for the project.

1.6 Submittals

1.6.1 Personnel

At least 45 calendar days before starting soil nail work, submit names of the Engineer, on-site supervisors, and drill operators assigned to the project, and a summary of each individual's experience. Only those individuals designated as meeting the experience 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 7 calendar days of receipt of the submission. Work shall not be started nor materials ordered until the Contractor's qualifications have been approved by the Owner's Engineer. The Owner's Engineer may suspend the work if the Contractor substitutes unapproved personnel for approved personnel during construction. If work is suspended due to the substitution of unqualified personnel, the Contractor shall be fully liable for all additional costs resulting from the suspension of work, and no adjustment in contract time resulting from the suspension of the work shall be allowed. The Contractor shall have overall contractual responsibility for both the design and construction.

1.6.2 Design Submittals

1.6.2a General

1. At least 30 calendar days before the planned start of the wall construction, the Contractor shall submit complete design calculations and Design Drawings to the Engineer for review and approval. Include all details, dimensions, quantities, ground profiles, and cross-sections necessary to conduct the work. *[Forty-five (45) calendar days may be necessary for some projects. In these cases, the submittal for personnel experience may need to also have an extended timeframe.]*

2. Design Drawings and calculations shall be signed and sealed by the Contractor's Design Engineer, previously approved by the Owner's Engineer.

3. Submit ___ sets of the Design Drawings with the initial submission. The Design Drawings shall be prepared to the (Owner) standards. The Owner's Engineer will

approve or reject the Contractor's submittal within 15 calendar days of the receipt of the complete submission.

4. The Owner's Engineer will approve or reject the Contractor's submittals within 14 calendar days of the receipt of the complete submission.
5. One set will be returned with any indicated corrections. If revisions are necessary, make the necessary changes and resubmit ____ revised sets. When the drawings are approved, furnish ___ sets of the approved drawings.
6. The Contractor shall not begin the work until the submittal requirements are satisfied and found acceptable by the Engineer.
7. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time or delay, or impact claims will be allowed due to incomplete submittals.

1.6.2b Design Calculations

Design calculations shall include, but not be limited to, the following items.

1. A narrative describing the overall soil nail wall design.
2. Applicable code requirements and references.
3. Design criteria, including soil/rock shear strength parameters (friction angle and cohesion), unit weights, pullout resistances, steel resistances, and shotcrete resistance values. Any additional subsurface borings, laboratory work, or other subsurface data collected for the design shall also be included.
4. Soil nail critical cross-section(s) including soil/rock strata, piezometric levels, and location, magnitude, and direction of applied loads.
5. Values and associated load factors used in design for pullout resistance, surcharges, soil/rock unit weights, nail head strengths, and strengths of steel, shotcrete, and concrete materials. Global stability soil resistance/load factors used in LRFD verifications.
6. Seismic design coefficient and other seismic design criteria applicable for the geographic area of the project.
7. Design calculation sheets with the project number, structure location, designation, date of preparation, initials of designer and checker, and page number at the top of each page. Provide an index page with the design calculations.
8. Design notes including an explanation of symbols and computer programs used in the design.

9. Structural design calculations for wall facing(s) and nail head/facing connections including consideration of facing flexural and punching shear strength, headed stud tensile strength, upper cantilever, minimum reinforcement ratio, cover, and splice requirements.

10. Other design calculations. *[Other items may include drainage evaluation and items not considered above.]*

1.6.2c Design Drawings

Design Drawings shall include all information required for the construction and quality control of the work. Design Drawings shall include, but not be limited to, the following items unless provided in the Contract Drawings:

1. A plan view of the structure(s) identifying:
    a. A reference baseline and elevation datum.
    b. The offset from the construction centerline or baseline to the face of the wall at its base at all changes in horizontal alignment.
    c. Beginning and end station of wall.
    d. Soil nail locations.
    e. Right-of-way and permanent or temporary construction easement limits, location of all known active and abandoned existing utilities, adjacent structures or other potential interferences. The centerline of any drainage structure or drainage pipe behind, passing through, or passing under the wall.
    f. Subsurface exploration locations shown on a plan view of the proposed wall alignment with appropriate reference base lines to fix the locations of the explorations relative to the structure(s).
    g. Limit of longest nails.
2. An elevation view of the structure(s) identifying:
    a. The elevation at the top of the wall, at all horizontal and vertical break points, and at least every 50 ft along the wall.
    b. Elevations at the base of the wall and the top of leveling pads for casting CIP facing (if applicable).
    c. Soil-nail elevations, vertical and horizontal spacing, and the location of wall drainage elements and permanent facing expansion/contraction joints (if applicable) along the wall length.
    d. Existing and finished grade profiles both behind and in front of the wall.
3. All necessary cross-section(s) to construct the wall. *[The number of cross sections may depend on the geometry of the wall and the soil nails, presence of utilities, and*

the variability of subsurface conditions. The number of cross sections may need to match the criterion for spacing test borings, as described in Chapter 4.]

4. General notes for constructing the soil nails including construction sequencing or other special construction requirements.
5. Design parameters and applicable codes.
6. Horizontal and vertical curve data affecting the wall and control points, including match lines or other details to relate to wall stationing to centerline stationing.
7. A listing of the summary of quantities on the elevation drawing of each wall showing the estimated surface area __ *[expressed in square feet or in a system of units consistent with that required by Owner]* and other pay items.
8. Nail wall typical sections including staged excavation lift elevations, wall and excavation face batter, nail spacing and inclination, sizes of nail bars (also referred to as tendons), and corrosion protection details.
9. Typical details of production and test nails defining the nail length, minimum drill hole diameter, inclination, and test nail bonded and unbonded test lengths.
10. Details, dimensions, and schedules for all nails, reinforcing steel, wire mesh, bearing plates, headed studs, etc. and/or attachment devices for shotcrete, cast-in-place or prefabricated facings.
11. Dimensions and schedules of all reinforcing steel including reinforcing bar bending details.
12. Details and dimensions for wall appurtenances such as barriers, coping, drainage gutters, fences, etc.
13. Details for constructing walls around drainage facilities.
14. Details for terminating walls and adjacent slope construction.
15. Facing finishes, color and architectural treatment requirements (if applicable) for permanent wall facing elements.

1.6.3 Construction Submittals

1.6.3a Construction Plan

At least 30 calendar days before starting the soil nail work, the Contractor shall submit a Construction Plan to the Owner's Engineer that includes the following:

1. Project start date and proposed detailed wall construction sequence.

2. Drilling and grouting methods and equipment, including the drill hole diameter proposed to achieve the specified nominal pullout resistance values 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. For neat cement grout include specific gravity test results of the fresh grout used for compressive testing.

4. Nail grout placement procedures and equipment.

5. Temporary shotcrete materials and methods. Provide the minimum 3-day and 28-day shotcrete compressive strengths.

6. Soil nail testing methods and equipment setup.

7. Identification number and certified calibration records for each test jack, pressure gauge, dial 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 calendar days prior to submittal.

8. Manufacturer's Certificates of Compliance for materials including: ultimate strength of the tendon, Portland cement, centralizers, bearing plates, epoxy coating, and encapsulation.

9. The Owner's Engineer shall approve or reject the Contractor's Construction Plan within 15 calendar days of receipt of the submission. Approval of the Construction Plan does not relieve the Contractor of his responsibility for the successful completion of the work.

1.6.3b Monitoring Plan

At least 30 calendar days before starting the soil nail work, the Contractor shall submit a Monitoring Plan, if applicable for the project, to the Owner's Engineer for approval. The Owner's Engineer shall approve or reject the Contractor's Monitoring Plan within 14 calendar days of receipt of the submission.

1.7 Construction Requirements

1.7.1 Storage

Tendons shall be stored and handled in a manner to avoid damage or corrosion. Tendons exhibiting abrasions, cuts, welds, weld splatter, corrosion, or pitting shall be replaced. Bars exhibiting damage to encapsulation or epoxy coating shall be repaired or replaced at no additional cost.

1.7.2 Excavation

The height of exposed unsupported final excavation face cut shall be established by the Contractor and 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.

Excavation to the final wall excavation line and shotcrete application shall be completed in the same work shift, unless otherwise approved by the Owner's Engineer.

Nail grout and shotcrete shall achieve sufficient strength before excavating the next underlying lift.

1.7.3 Soil Nail Installation

Drill holes for the soil nails shall be drilled at the locations, elevations, orientations, and lengths shown on the approved Design Drawings. The drilling equipment and methods shall be selected by Contractor to be suitable for the ground conditions and in accordance with the accepted installation methods submitted by the Contractor.

1.7.4 Protection of Adjacent Structures

Structures located within a horizontal distance equal to ____ shall be monitored for vertical and horizontal movement in a manner approved by the Engineer within an accuracy of 0.01 in. Monitoring of adjacent structures will be done by an independent party who must be approved by the Engineer and will work under the Contractor.

A monitoring plan, including the locations of measurement points and the frequency of recording measurements shall be submitted to the Engineer as part of the Construction Plan. Monitoring shall begin with a base-line measurement recorded no less than 10 calendar days prior to construction of the wall. In addition to monitoring for movement, the condition of the adjacent structure, including cracks and crack widths, before and after construction of the wall, shall be documented by visual inspection, photographs, and/or video. Structures owned

by Owner shall be monitored for movement as long as this requirement is called for on the Contract Drawings.

As soon as the movements measured exceed thresholds established for adjacent structures, the Contractor shall stop construction, notify the Engineer, and take any immediate remedial measures required to prevent damage to the adjacent structures. The Contractor and the Engineer shall then review the current installation procedures. If revisions to the installation procedures are deemed necessary, the Contractor shall submit a revised installation plan for approval by the Engineer before resuming work.

1.8 Soil Nail Testing

1.8.1 Tests

The Contractor shall perform both verification and proof testing of designated test soil nails. Verification tests on sacrificial test nails shall be conducted at locations shown on the approved Design Drawings. Proof tests on production nails shall be conducted at locations selected by the Owner's 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. The Contractor shall not apply loads greater than 80 percent of the minimum guaranteed ultimate tensile strength of the tendon for Grade 150 bars or 90 percent of the minimum guaranteed ultimate tensile strength of the tendon for Grade 60 or 75 bars. Preliminary results shall be submitted to the Owner and/or Owner's Engineer within 24 hours of the test completion. A full report containing test load results shall be submitted to the Owner and/or Owner's Engineer within 5 working days of the test completion.

Refer to FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9, for detailed guidance on soil nail testing.

1.8.2 Verification Testing

The Contractor shall perform a number of verification tests on sacrificial soil nails as established in the Design Drawings. Verification testing shall be conducted prior to installation of production soil nails on sacrificial soil nails to confirm the appropriateness of the Contractor's drilling and installation methods, and verify the required nail pullout resistance.

The maximum test load in verification tests (VTL) shall be calculated based on as-built bonded lengths per FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9. The Load Schedule for Verification Testing shall comply with FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9.

1.8.3 Proof Testing

Successful proof testing shall be demonstrated on at least 5 percent of production soil nails in each nail row or a minimum of one per row. The Owner's Engineer shall determine the locations and number of proof tests prior to nail installation in each row. *[The quantity of proof tests should be established by the Owner prior to bidding.]* Verification tests shall not be counted towards the minimum of 5 percent of production nails.

The maximum test load in proof tests (PTL) shall be calculated based on as-built bonded lengths per FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9. The Load Schedule for Proof Testing shall comply with FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9.

1.9 Acceptance Criteria

1.9.1 Verification Tests

Considering that pullout is defined as the load at which attempts to further increase the test load increments simply result in continued movement of the tested nail, a test nail shall be considered acceptable when all of the following criteria are met.

1. The total creep movement is less than 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. The total movement (ΔVTL) measured at VTL exceeds 80 percent of the theoretical elastic elongation of the unbonded length of the test nail, as defined in FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9.
3. Pullout does not occur before achieving 1.0 × VTL.

1.9.2 Proof Tests

1. The creep movement between the 1- and the 10-minute readings is less than 0.04 in.
2. In cases when the creep movement between the 1- and the 10-minute readings is greater than 0.04 in., the creep movement between the 6- and the 60-minute readings is less than 0.08 in., and the creep rate is linear or decreasing throughout the creep test load hold period.
3. The total soil nail movement (ΔPTL) measured at PTL exceeds 80 percent of the theoretical elastic elongation of the unbonded length, as defined in FHWA Geotechnical Circular No. 7 "Soil Nail Walls," Chapter 9.
4. Pullout does not occur before the test load is 1.0 × PTL.

5. The temporary unbonded test length in proof tests is successfully maintained for subsequent satisfactory grouting. If the unbonded test length of production proof test nails cannot be satisfactorily grouted after 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.

1.10 Rejection of Test Soil Nails

1.10.1 Verification Test Soil Nails

The Owner's 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 for review by the Owner's Engineer and shall install replacement verification test nails. Replacement test nails shall be installed and tested at no additional cost.

1.10.2 Proof Test Soil Nails

For proof test nails, the Owner's Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test soil nail and the adjacent passing proof test nail. Alternatively, the Owner's Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient nominal pullout resistance. Installation and testing of additional proof test nails or installation of additional or modified nails as a result of proof test nail failure(s) shall be at no additional cost.

1.11 Wall Drainage System

1.11.1 General

Provide all elements of a soil nail wall drainage system consisting of geocomposite drain strips, PVC connection pipes, soil nail wall footing drains, and weepholes, or similar, that will provide a continuous path for water flow and prevent pore water pressure from building up behind the wall.

1.11.2 Geocomposite Strip Drain

Provide geocomposite strip drain or similar to prevent pore water pressure from building up behind the wall and to convey the collected groundwater to the base of the wall behind the wall. Geocomposite strip drain shall have sufficient capacity to convey all drained groundwater, and sufficient resistance to prevent collapse during construction and operation.

1.11.3 Footing Drains

Footing drains shall collect groundwater from the drainage system and convey it to ____ *[e.g., ditch collectors]*, as shown on Contract Drawings.

1.12 Shotcrete Facing

1.12.1 General

Initial shotcrete facing and final shotcrete facing shall be provided as required and as shown on the approved Design Drawings.

1.12.2 Shotcrete Facing Tolerances

Construction tolerances for the shotcrete facing from plan location and plan dimensions shall be as follows:

Table F.1: Construction Tolerances for Shotcrete Facing

Item	Tolerance
Horizontal location of welded wire mesh, reinforcing bars, and headed studs	3/8 in.
Location of headed-studs on bearing plate	1/4 in.
Spacing between reinforcing bars	1 in.
Reinforcing lap	1 in.
Thickness of shotcrete, if troweled or screeded	9/16 in. [approximation of 0.6 in.]
Thickness of shotcrete, if left as shot	1-1/8 in. [approximation of 1.2 in.]
Planeness of finish face surface, gap under 10-ft straightedge, if troweled or screeded	9/16 in. [approximation of 0.6 in.]
Planeness of finish face surface, gap under 10-ft straightedge, if left as shot	1-1/8 in. [approximation of 1.2 in.]
Nail head bearing plate deviation from parallel to wall face	10 degrees

1.13 Reinforcing Steel

The Contractor shall submit all order lists and reinforcement bending diagrams to the Owner's Engineer, and shall fabricate reinforcing steel, ship and protect material, place, fasten, and splice reinforcing steel as shown on the approved Design Drawings.

1.14 Structural Concrete

The Contractor shall design the concrete mix; store, handle, batch, and mix material; and deliver concrete; provide quality control; and construct concrete facing to meet the resistance requirements shown on the approved Design Drawings.

1.15 Architectural Surface Finishes

[If applicable] The Contractor shall submit detailed drawings of the form liner for approval by the Owner's Engineer at least 7 calendar days before form liner work begins. Before production work begins, a 3-ft high, 10-ft long test panel shall be constructed 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. One test panel per wall finish type shall be furnished.

1.16 Corrosion Protection

Soil nails and soil nail head components shall be protected against corrosion consistent with the ground and groundwater conditions at the site and the minimum class of corrosion protection shown on Contract Drawings.

Part 2 – Measurement and Payment

[Per this Performance-Based Specification, the Contractor is responsible for design and construction. The single pay item is the finished wall area, which is inclusive of all soil nail costs, including design, materials, installation, and testing.]

The finished work shall be measured by the pay item "Finished Wall" and shall be paid by the square foot. Measurement will be made on the vertical plane of the front face as accepted in the final work. The final pay quantity shall include all excavation, all materials and installation, all testing and monitoring, as described on Design Plans and this Specification.

Commentary: *The Owner may consider including additional pay items.*

Appendix G: Similar Technologies

G.1 Launched Nails

Launched nails are bars "launched" into the soil at high speeds approaching 200 miles per hour using a firing mechanism involving compressed air. Bars can be perforated galvanized steel tube, perforated fiberglass, or bare steel tubes. Bars are typically 1.5 in. in diameter and can be 2 in. in diameter. They can be up to 20 ft in length. An epoxy-coated, small-diameter threaded bar can be inserted into the tube after pressure grouting is applied to increase structural capacities.

Launched nails allow for a fast installation with little impact to the project site; however, it may be difficult to control the length of nail that penetrates the ground. Advantages include rapid construction, easy monitoring and testing, construction with limited headroom and right-of-way, and ability to withstand large deformations. Potential disadvantages with launched nails include: (i) this is a proprietary and licensed technology; (ii) specialized contractor and equipment are required; and (iii) lack of simple, comprehensive design procedures.

This technique is applicable to landslide repairs, and to roadway and embankment widening. The bars are generally not grouted and, thus, this technology does not meet the definition of a soil nail presented in Chapter 1. A launched nail acts as a dowel in the soil, and the contribution to stability is primarily by shear and associated, localized bending, and not primarily by tension as with a drilled and grouted nail. Launched nails develop limited axial capacity without grout. However, the technique mentioned above includes inserting a threaded bar for some to allow pressure grouting, and some increased capacity is possible. Figure G.1 shows a launched nail installation. Detailed information on this technology is available at the GeoTechTools web-based information and guidance system (SHRP 2013) and in USDA (1994a and 1994b).



Figure G.1: Photo. Launched nail installation. Photograph courtesy of GeoStabilization International.

G.2 Screw-in Nails

Screw-in soil nailing consists of helical soil nails that stabilize retained soils. These nails typically comprise a 1.5-in. square solid steel shaft on which steel helices are welded at regular intervals. Helical soil nails are installed using drilling equipment with sufficient torque output to penetrate the native soils. The spacing of the helices is a function of the helix diameter and is typically about 3.6 times the diameter. Screw-in nails are typically used in places difficult to access or for small areas (Collin and Cowell, 1998). The bars are not grouted and, thus, this technology does not meet the definition of a soil nail, as defined in Chapter 1. A screw-in nail acts as an anchor in the soil, and the contribution to stability is by bearing resistance of the helices, not by bond stresses developed along the reinforcement as is the case with a drilled and grouted soil nail. Figure G.2 shows the installation of a screw-in nail. Detailed information on this technology is available at the GeoTechTools site (SHRP 2013).

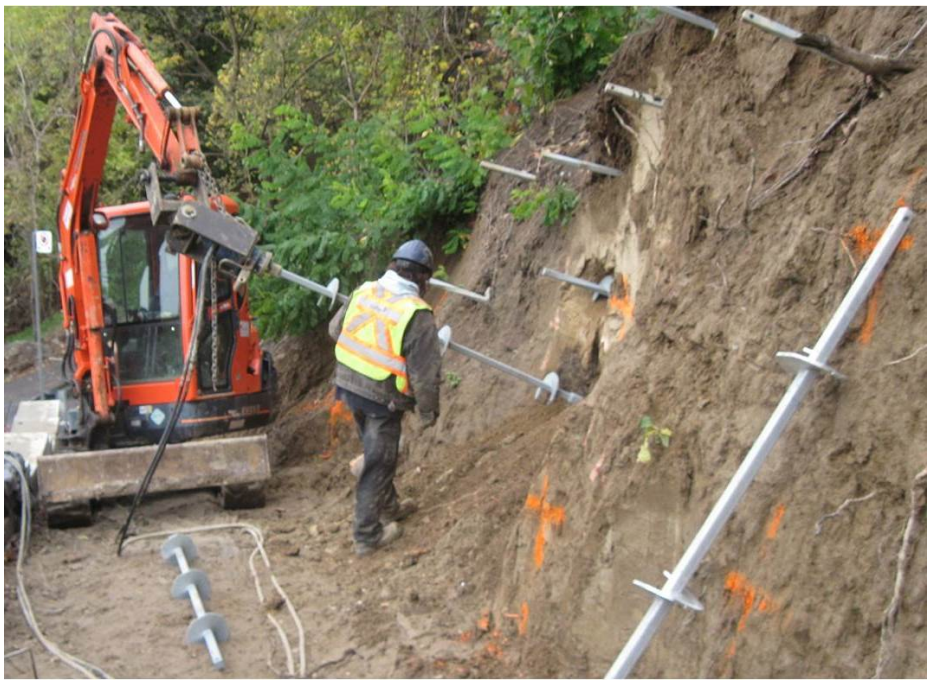


Figure G.2: Photo. Screw-in nail installation. Photograph courtesy of Hubbell Power Systems, Inc.