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MANUAL FOR DESIGN & CONSTRUCTION MONITORING OF SOIL NAIL WALLS

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Innovation Through Partnerships

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16. Abstract: This revised version of SA-96-069 incorporates primarily clarification and format changes received in the project workshops. However, the basic design and construction procedures are unchanged from the original. The long-term performance of soil nail walls has been proven after 20 years of use on Europe and the United States. The purpose of this manul is to facilitate the implementation of soil nailing into the American transportation design and construction practice and to provide guidance for selecting, designing, and specifying soil nailing where it is technically suited and economically attractive. A comprehensive review of current design and construction methods has been made and the results compiled into a guideline procedure. The intent of presenting the guideline procedure is to ensure that agencies adopting soil nail wall design and construction follow a safe, rational procedure from site investigation through construction. This manual is practitioner oriented and includes: descriptions of soil nailing concept and applications; summary of experimental programs and monitoring of in -service walls; recommended methods of site investigation and testing; recommended design procedures for both Service Load Design (SLD) and Load Resistance Factors (LRFD); worked design examples; simplified design charts for the preliminary design of cut slope walls; wall performance monitoring recommendations; discussion on practice and quality control of shotcrete application in soil nailing; discussion of contracting procedures and guidance on the preparation of soil nail design and construction; presentation of procedures for determining the structural capacity of nail head connectors and wall facings, including demonstration calculations. This manual is intended to be used by civil engineers who are knowledgeable about soil mechanics and structural engineering fundamentals and have an understanding of the principles of soil-reinforcement technology and earth work construction. A companion document titled FHWA Soil Nailing Inspectors Manual (FHWA-SA-93-068) is also available from NTIS.

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ENGLISH TO METRIC (SI) CONVERSION FACTORS

The primary metric (SI) units used in civil and structural engineering are:

length	-	meter (m)
mass	-	kilogram (kg)
time	-	second (s)
force	-	newton (N) or kilonewton (kN)
pressure	-	pascal (Pa = N/m^2) or kilopascal (kPa = kN/m^2)

The following are the conversion factors for units presented in this manual:

Quantity	From English Units	To Metric (SI) Units	Multiply by	For aid to Quick Mental Calculations
Mass	lb	kg	0.453 592	1 lb(mass) = 0.5kg
Force	lb kip	N kN	4.448 22 4.448 22	1 lb(force) = 4.5N 1 kip(force) = 4.5kN
Force/unit length	plf klf	N/m kN/m	14.593 9 14.593 9	1 plf = 14.5N/m 1 klf = 14.5kN/m
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa Mpa	47.880 3 47.880 3 6.894 76 6.894 76	1 psf = 48 Pa 1 ksf = 48 kPa 1 psi = 6.9 kPa 1 ksi = 6.9 Mpa
Length	inch foot	mm m mm	25.4 0.3048 304.8	1 in = 25 mm 1 ft = 0.3 m 1 ft - 300 mm
Area	square inch square foot square yard	mm^2 m^2 m^2	645.16 0.09290304 0.83612736	1 sq in = 650 mm^2 1 sq ft = 0.09 m^2 1 sq yd = 0.84 m^2
Volume	cubic inch cubic foot cubic yard	mm^3 m^3 m^3	16386.064 0.0283168 0.764555	1 cu in = 16,400 mm^3 1 cu ft = 0.03 m^3 1 cu yd = 0.76 m^3

A few points to remember:

1. In a **"soft"** conversion, an English measurement is mathematically converted to its **exact** metric equivalent.
2. In a **"hard"** conversion, a new **rounded**, metric number is created that is convenient to work with and remember.
3. Use only the meter and millimeter for length (avoid centimeter).
4. The pascal (Pa) is the unit for pressure and stress (Pa = N/m^2).
5. Structural calculations should be shown in MPa or kPa.
6. A few basic comparisons worth remembering to help visualize metric dimensions are:
   - One mm is about 1/25 inch or slightly less than the thickness of a dime.
   - One m is the length of a yardstick plus about 3 inches.
   - One inch is just a fraction (1/64 inch) longer than 25 mm (1 inch = 25.4 mm).
   - Four inches are about 1/16 inch longer than 100 mm (4 inches = 101.6 mm).
   - One foot is about 3/16 inch longer than 300 mm (12 inches = 304.8 mm).

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General		Units
A	design pseudo-static seismic coefficient	(g's)
A_B	area of an individual bar	(mm^2)
A_C	area of punching shear cone base at back of facing	(mm^2)
A_E	effective tension area of concrete surrounding the flexural tension reinforcement	(mm^2)
A_{GC}	cross-sectional area of soil nail borehole	(mm^2)
A_{HS}	cross-sectional area of body of headed stud	(mm^2)
A_{LB}	area of largest bar in width S_H	(mm^2)
A_{PK}	design peak ground acceleration	(g's)
A_S	area of tension reinforcement in facing panel width 'b'	(mm^2)
$A_{S,NEG}$	area of tension reinforcement in facing panel width 'b' (negative moments)	(mm^2)
A_{SN}	cross-sectional area of strut nail borehole	(mm^2)
A_{ST}	cross-sectional area of steel in strut nail	(mm^2)
A_{TOTAL}	total area of reinforcement in width S_H	(mm^2)
A_{WIRE}	area of individual wire to be spliced	(mm^2)
B	width of base of soil nailed block	(m)
b	width of unit facing panel (equal to S_H)	(m)
B'	effective width of base of soil nailed block, accounting for eccentric loading	(m)
b_{PL}	width of bearing plate	(mm)
c_D	dimensionless cohesion	-
C_F	facing flexure pressure factor	-

C_S	facing punching shear pressure factor	-
C_{ST}	service compressive load in strut nail	(kN)
c_U	ultimate soil cohesion	(kN/m^2)
d	distance from extreme compression fiber to centroid of tension reinforcement	(mm)
D'_C	effective diameter of punching shear cone	(mm)
d_B	nominal diameter of bar or wire	(mm)
D_C	effective diameter of punching shear cone at back of facing	(mm)
d_C	thickness of concrete cover from extreme tension fiber to center of closest bar or wire (not greater than 50 mm)	(mm)
D_{GC}	diameter of the soil nail borehole	(mm)
d_H	body diameter of headed stud	(mm)
d_{HS}	head diameter of headed stud	(mm)
e	load eccentricity	(m)
E_C	modulus of elasticity of concrete	(MN/m^2)
E_S	modulus of elasticity of reinforcement	(MN/m^2)
F	global soil factor of safety	-
f'_C	concrete compressive strength	(MN/m^2)
F_F	nail head service load factor	-
F_S	allowable tensile stress in reinforcements in zones of flexure	(MN/m^2)
f_S	service steel stress	(MN/m^2)
F_U	ultimate tensile stress of headed stud	(MN/m^2)
F_Y	tensile yield stress of reinforcement	(MN/m^2)
H	vertical height of soil nail wall	(m)
h_C	effective height of punching shear cone	(mm)
i_c	load inclination factor	-

i_γ	load inclination factor	-
j	internal moment arm to steel depth ratio (ultimate conditions)	-
k	neutral axis to steel depth ratio (service conditions)	-
K_A	active earth pressure coefficient	-
L	nail length	(m)
L_C	clear span between nail heads (nail spacing)	(m)
L_D	development length of reinforcement	(mm)
L_{DB}	basic development length of reinforcement	(mm)
L_{HS}	length of headed stud	(mm)
L_S	length of splice or lap	(mm)
L_{SN}	length of strut nail	(m)
L_{VB}	length of vertical bearing bars	(m)
m	nominal unit moment resistance (facing)	(kN-m/m)
m_S	one-way unit service moment (facing)	(kN-m/m)
$m_{V,NEG}$	nominal unit moment resistance (vertical direction, negative moments)	(kN-m/m)
n	modular ratio of elasticity (E_S/E_C)	-
N_c	bearing capacity factor	-
N_γ	bearing capacity factor	-
P_A	active earth pressure load ($K_A \gamma H^2 / 2$)	(kN/m)
P_{EQ}	seismic earth load on back of nailed block	(kN/m)
Q_D	dimensionless soil pullout resistance	-
q_{MAX}	maximum applied bearing pressure	(kN/m^2)
Q_U	ultimate pullout resistance	(kN/m)
q_{ULT}	ultimate bearing capacity	(kN/m^2)
s_c	footing shape factor	-
s_γ	footing shape factor	-

S_H	nail spacing (horizontal)	(m)
S_{HS}	centerline spacing of headed studs	(mm)
S_V	nail spacing (vertical)	(m)
S_{WIRE}	spacing of wires to be spliced	(mm)
t_C	concrete/shotcrete cover thickness	(mm)
T_D	dimensionless nail tensile capacity	-
t_F	nail head service load	(kN)
T_{FN}	nominal nail head strength	(kN)
t_H	head thickness of headed stud	(mm)
T_{NN}	nominal nail tendon strength	(kN)
t_{PL}	thickness of bearing plate	(mm)
T_{ST}	service tensile load in soil nail (strut nail support of facing self weight)	(kN)
v	one-way unit service shear force (facing)	(kN/m)
V_N	nominal internal punching shear strength (facing)	(kN)
V_{NS}	nominal one-way unit shear strength (facing)	(kN/m)
z	quantity limiting distribution of flexural reinforcement	(kN/m)
β	angle of backslope behind wall	(°)
δ	batter angle of facing (from vertical)	(°)
ϕ_D	factored friction angle	(°)
ϕ_U	ultimate soil friction angle	(°)
γ	soil unit weight	(kN/m^3)
ρ	tension reinforcement ratio	-

Service Load Design

FS	bearing capacity factor of safety	-
M	allowable one-way unit moment (facing)	(kN-m/m)
P_E	static active earth load on back of nailed block	(kN/m)
Q_d	allowable pullout resistance	(kN/m)
q_{ALL}	allowable foundation bearing pressure	(kN/m^2)
T	allowable nail load	(kN)
T_F	allowable nail head load	(kN)
T_N	allowable nail tendon load	(kN)
V	allowable one-way unit shear (facing)	(kN/m)
α_F	nail head strength factor	-
α_N	nail tendon strength factor	-
α_Q	nail pullout resistance strength factor	-

Load and Resistance Factor Design

A_G	gross area of section	(mm^2)
c	design soil cohesion	(kN/m^2)
M	design one-way unit moment resistance	(kN-m/m)
P_{EH}	static active earth load on back of nailed block	(kN/m)
Q_d	design pullout resistance	(kN/m)
T	design nail strength	(kN)
T_F	design nail head strength	(kN)
T_N	design nail tendon strength	(kN)
V	design one-way unit shear strength	(kN/m)
ϕ	design soil friction angle	(°)

Φ_C	soil cohesion resistance factor	-
Φ_F	nail head resistance factor	-
Φ_ϕ	soil friction resistance factor	-
Φ_N	nail tendon resistance factor	-
Φ_q	bearing capacity resistance factor	-
Φ_Q	nail pullout resistance factor	-
Γ	load factor	-

CHAPTER 1. INTRODUCTION AND APPLICATION CRITERIA

1.1 Purpose and Scope of Manual

The specific purpose of this manual is to introduce the concept of soil nailing use into American transportation construction practice and to provide guidance for selecting, designing, and specifying soil nailing for those many applications to which it is technically suited and economically attractive. A comprehensive review of current design and construction methods has been made and the results compiled into a guideline procedure. The focus of this document is primarily on design methods and procedures for permanent soil nailing.

Permanent soil nailing systems are generally considered to have a service life of 75-100 years. However, soil nailing is commonly used for temporary applications. The service life of temporary earth support works is based on the time needed to support the ground while the permanent works are installed. This document has adopted the AASHTO guidance which considers temporary works to be those that are to be removed on completion of the permanent works. The time period for temporary works is commonly stated to be 18-36 months but may be shorter or longer based on actual project conditions.

The intent of presenting the guideline procedure is to ensure that agencies adopting permanent soil nail wall design and construction follow a safe, rational procedure from site investigation through construction. Close attention has been given to the presentation of suggested general specifications and plan details. Contract documents such as these, which provide the transition from design analyses to field construction, frequently decide the success or failure of new design concepts. Every effort has been made in the sample specifications to give all experienced nailing contractors an opportunity to use innovative methods or equipment in construction. Such specifications are needed to encourage contractors to seek cost-effective improvements to current soil nailing methods.

Engineers responsible for design and construction of public works usually need long-term monitoring of new techniques pioneered in the private sector before they can be confidently incorporated in permanent public projects. The long-term performance of soil nail walls has been proven after 20 years of use in Europe and the United States. This manual is intended to permit engineers to rationally and confidently specify permanent soil nailing in cost-effective situations. Implementation of permanent soil nailing is consistent with national efforts to upgrade the safety and efficiency of the transportation system in the most cost-effective manner possible. The goal of this FHWA demonstration project is to assist U.S. transportation agencies in implementing the safe and cost-effective use of permanent soil nail designs as alternate bid items to the standard wall systems presently used to retain steep excavation cut slopes.

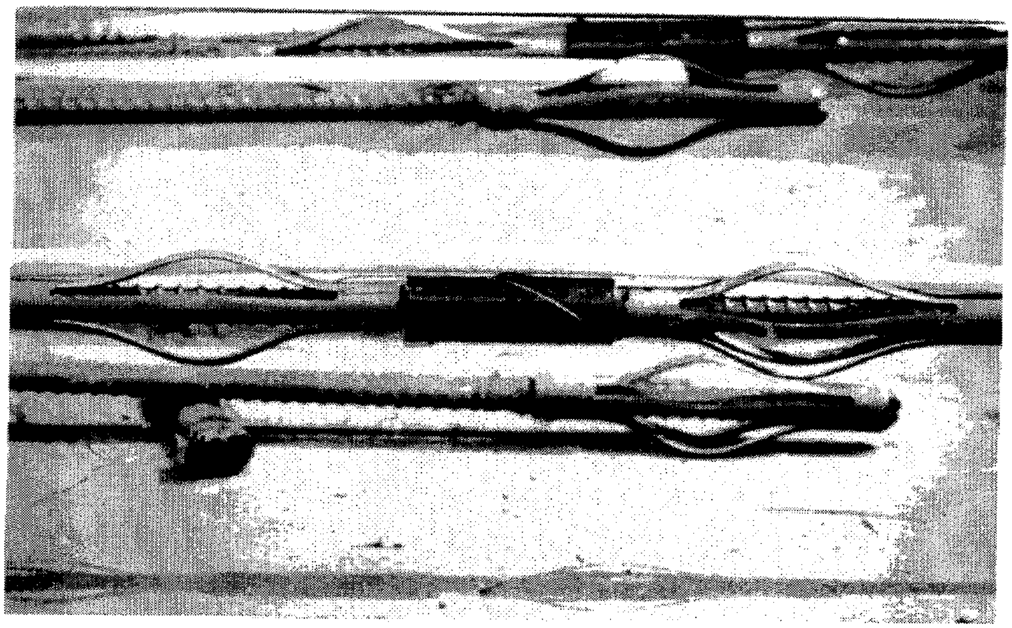
The scope of this manual includes:

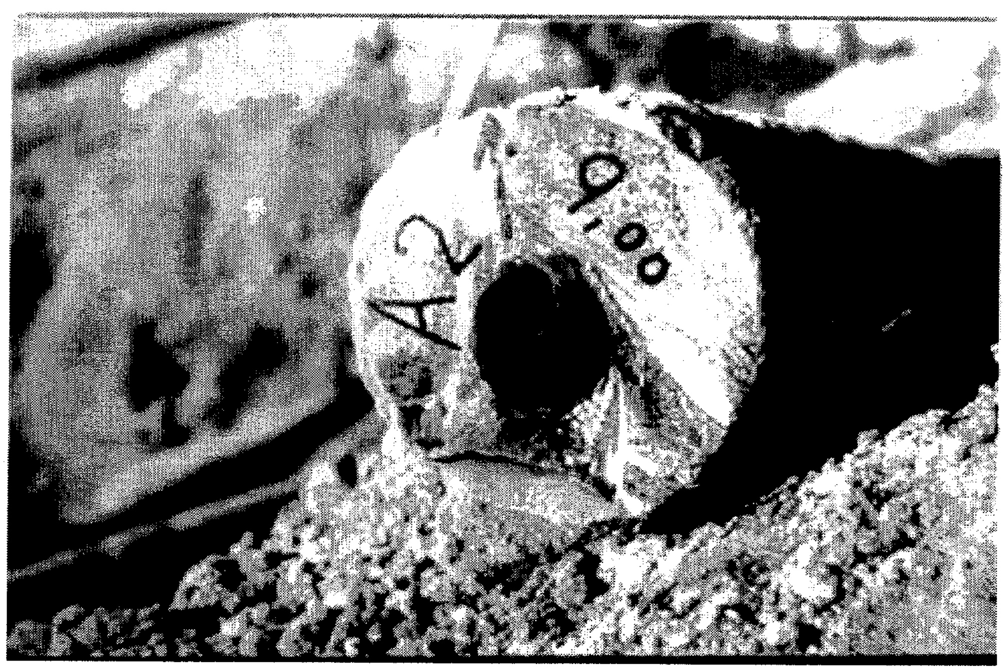
Chapter 1	A brief description of the soil nailing concept and a discussion of the advantages, limitations, and recommended applications of the technique.

Chapter 2	A description of the history of the use of soil nailing in both North America and Europe, of the method of construction (including construction equipment and materials), and of the behavior of soil nail walls as determined from both experimental programs and monitoring of in-service walls.
Chapter 3	Recommended methods of site investigation and testing.
Chapter 4	Recommended design procedures.
Chapter 5	Worked Design Example including simplified design charts for the preliminary design of cut slope walls.
Chapter 6	Wall performance monitoring recommendations.
Chapter 7	Discussion on the practice and quality control of shotcrete application in soil nailing.
Chapter 8	Discussion of contracting procedures and guidance on the preparation of plans and specifications.
Appendices A-D	Guide Specifications and Example Plans for Soil Nail Wall Construction and Performance Monitoring.
Appendix E	Quality Control Checklist for soil nail design and construction.
Appendix F	Presentation of procedures for determining the capacity of nail head connectors and wall facings, including demonstration calculations.
Appendix G	Worked design example 5.2 for cut-slope walls.

1.2 Soil Nail Description

A soil nail is a structural element which provides load-transfer to the ground in excavation reinforcement applications. The "nail" may simply consist of a steel tendon, but most commonly the tendon is encapsulated in a cement grouted body to provide corrosion protection and improved load-transfer to the ground.





1.3 The Soil Nailing Concept

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely-spaced steel bars, called "nails", into a slope or excavation as construction proceeds from the "top down." This process creates a reinforced section that is itself stable and able to retain the ground behind it. As with mechanically stabilized earth (MSE) walls, the reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction. Nails work predominantly in tension but are considered by some to work also in bending/shear in certain circumstances. Consideration of shear/bending contributions is not included in the recommended design methods presented later in this manual. The effect of the nail reinforcement is to improve stability by (a) increasing the normal force and hence the soil shear resistance along potential slip surfaces in frictional soils; and (b) reducing the driving force along potential slip surfaces in both frictional and cohesive soils. A construction facing is also usually required and is typically shotcrete reinforced by welded wire mesh.

There are three main categories of in-situ reinforcement techniques used to stabilize soil slopes and support excavations. These are nailing, reticulated micro-piling, and doweling.

In soil nailing (figure 1.1a), the reinforcement is installed horizontally or sub-horizontally (approximately parallel to the direction of major tensile straining in the soil) so that it contributes to the support of the soil partially by directly resisting the destabilizing forces and partially by increasing the normal loads (and hence the shear strength) on potential sliding surfaces.

Reticulated micro-piles are steeply inclined in the soil at various angles both perpendicular and parallel to the face (figure 1.1b). The overall aim is similar to soil nailing, namely to provide a stable block of reinforced soil which supports the unreinforced soil by acting like a gravity retaining structure. In this technique, the soil is held together by the multiplicity of reinforcement members acting to resist bending and shearing forces. [1] [2]

Soil doweling is applied to reduce or halt downslope movements on well-defined shear surfaces (figure 1.1c). The slopes treated by doweling are typically much flatter than those in soil nailing or reticulated micro-pile applications. Gudehus [3] has shown that the most efficient way to mechanically increase the shearing resistance on a weakened shear surface through the soil is to use relatively large diameter piles which combine a large surface area with high bending stiffness. Thus, the diameter of a soil dowel is generally greater than that of a soil nail or micro-pile.

This design manual is concerned only with soil nailing. Furthermore, the method of analysis presented herein addresses tension only as the resisting element for excavation support systems and slope stabilization. The reinforced soil body becomes the primary structural element. The reinforced zone performs as a homogenous and resistant unit to support the unreinforced soil behind it in a manner similar to a gravity wall.

Since the introduction of soil nailing, its application has extended to a wide variety of ground types ranging from soils to weathered and unweathered rock. While the term "ground" nail might be a more suitable generic term, "soil" nail has become established as the commonly accepted generic terminology. Therefore, to be consistent with established practice, the term "soil nail" is used in this manual as a generic term that applies to nails installed in all types of ground.

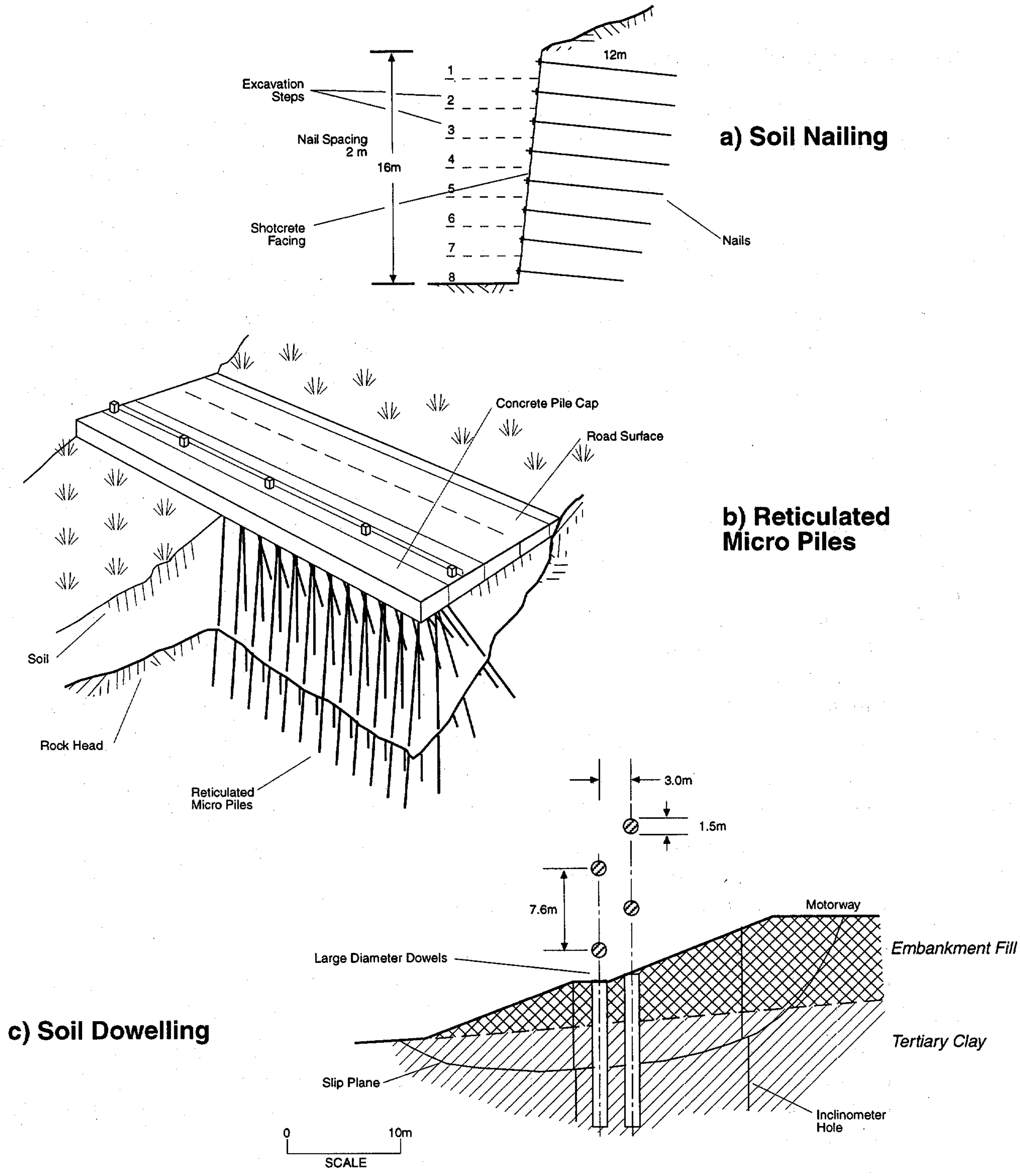


Figure 1.1 In Situ Soil Reinforcement Techniques

1.4 Advantages of Soil Nailing

Soil nailing exhibits many of the same advantages as tieback walls as a method of ground support/reinforcement, together with additional benefits that are unique to nailing. Like tieback walls, the top down construction technique of soil nailing offers the following benefits:

- Improved economy and lessened environmental impact compared to conventional retaining walls, through the elimination of the need for a cut excavation and backfilling.
- Improved economy and materials savings through the incorporation of the temporary excavation support system into the permanent support system.
- Improved economy and lessened environmental impact through reduction in the right-of-way (ROW) requirements.
- Improved safety by eliminating cramped excavations cluttered with internal bracing.

Compared to tieback walls, soil nailing may offer the following advantages in ground suitable for soil nailing:

- Elimination of the need for a high capacity structural facing (i.e., soldier piles and thick CIP) since the maximum earth pressure support loads are not transferred to the excavation face. For constructibility reasons, a permanent CIP facing is normally 200 mm thick. Most tieback walls have a permanent facing which is 250-300 mm thick.
- Improved construction flexibility in heterogeneous soils with cobbles, boulders or other hard inclusions, as these obstructions offer fewer problems for the relatively small diameter nail drillholes than they do for the large diameter soldier pile installations.
- Improved construction flexibility where overhead access is limited (e.g., road widening under an existing bridge) through the elimination of the requirement for drilled or driven soldier piles installed through the bridge deck or in hand dug pits.
- Ease of construction and reduced construction time - soldier pile installations are not required, soil nails are not prestressed, and construction equipment is relatively small, mobile, and quiet. This is particularly advantageous on urban sites.
- The vertical components of the nail reaction at the facing are smaller than those for tiebacks and are also distributed more evenly over the entire excavation face. This eliminates the need for significant wall embedment below grade, such as is required for tieback soldier piles.
- Higher system redundancy as the soil nails are installed at a far higher density than the prestressed tieback anchors, and the consequences of a unit failure are therefore correspondingly less severe. It should be noted that this does not necessarily imply higher system reliability for soil nail walls, since each tieback is tested during installation, whereas only a small percentage of nails are tested.

- Reduced right-of-way requirements, as the nails are typically shorter than the tieback anchors.

Other favorable features of soil nail retaining systems include the following:

- The method is well-suited to sites with difficult or remote access because of the relatively small size and the mobility of the required construction equipment.
- The method is well-suited to urban construction where noise, vibration, and access can pose problems.
- The construction method is flexible and can follow difficult excavation shapes using splayed nails and can cope with significant variations in soil conditions encountered during construction. Nail layout modifications during construction (e.g., moving nails to miss unanticipated obstructions) can be relatively easily accomplished.
- The system is relatively robust and flexible and can accommodate significant total and differential settlements. Soil nail retaining walls have been documented to perform well under seismic loading conditions [4].
- Field monitoring has indicated that overall movements required to mobilize the reinforcement forces are relatively small and correspond generally to the movements that would be expected for well braced systems (Category I) in Peck's classification [5]. Measured wall movements are usually in the range of 0.1 to 0.3 percent of the wall height.
- The method is well suited to specialist applications such as the rehabilitation of distressed retaining structures.

1.5 Limitations of Soil Nailing

Soil nailing and other cut retaining techniques share the following limitations:

- Permanent underground easements may be required.
- In urban areas, the closely spaced array of reinforcements may interfere with nearby utilities. Utility trenches represent potential planes of weakness that can contribute to failure, may contain poorly compacted or otherwise unsuitable fill for soil nailing, and may also carry ground water to the wall. Significant groundwater seepage at the excavation face can cause serious constructibility problems.
- Horizontal displacements may be somewhat greater than with prestressed tiebacks, and this may cause distortions to immediately adjoining structures.
- Nail capacity may not be economically developed in cohesive soils subject to creep, even at relatively low load levels.
- The long-term performance of shotcrete facings has not been fully demonstrated particularly in areas subject to freeze-thaw cycles.

The technique also has certain practical limitations to its application. These are:

- Soil nail construction requires the formation of cuts generally 1 to 2 m high in the soil. These must then stand unsupported, prior to shotcreting and nailing. The soil must therefore have some natural degree of "cohesion" or cementing, otherwise slotting, berming or reduced cut excavation lift heights may be necessary to stabilize the face, adding both complication and cost. Therefore, soil nailing is not well suited to applications in clean sands and gravels.
- A dewatered face in the excavation is highly desirable for soil nailing. If the ground water percolates through the face, the unreinforced soil may slump locally upon excavation, or the shotcrete to soil bond may be reduced, making it impossible to establish a satisfactory shotcrete skin.
- Excavations in soft clay are also unsuited to stabilization by soil nailing. The low frictional resistance of soft clay would require a very high density of in-situ reinforcement of considerable length to ensure adequate levels of stability. Tieback or bored pile walls are more suited to these conditions.
- Soil nailing in sensitive or expansive clays must be carefully evaluated. Care must be taken to prevent disturbing the soil or allowing water to soften and weaken the soil.

Finally, wall performance can be relatively sensitive to the selected method of construction, and is best achieved by experienced, specialty contractors.

1.6 Application Criteria

The most cost-effective application of soil nail retaining walls is usually as an alternative to tieback soldier pile walls or conventional retaining walls with temporary shoring i.e., where site geometry or adjacent property constraints do **not** permit an unsupported permanent cut excavation. Soil nail walls are particularly well-suited to the following highway applications, all of which have been successfully demonstrated on highway projects in both North America and Europe–roadway cut excavations, widening under an existing bridge end, tunnel portal cut stabilization, and repair and reconstruction of existing retaining structures.

1.6.1 Retaining Structures in Cuts

In ground suitable for soil nailing, soil nailing technology can be considered for permanent or temporary cut wall applications where conventional cast-in-place, tieback wall, precast, or mechanically stabilized earth (MSE) structures are applicable. In addition, a wide range of aesthetic requirements can be accommodated with the variety of final facing techniques available.

In particular, the following specific uses have been demonstrated in a variety of highway related projects:

- Vertical or near-vertical cut construction in both soils and weathered rock has been demonstrated to minimize excavation and backfill quantities, reduce right-of-way and clearing limits, and hence minimize environmental impacts within the transportation corridor. Soil nailing is therefore considered to be particularly applicable for uphill widening projects that must be constructed either within an existing ROW or in steep terrain.
- Tunnel portal cuts are often located in steep terrain of variable stratigraphy (soil/rock) and therefore subject to landslide development. Soil nailing has been successfully used to stabilize portal cuts on a number of tunnel projects carried out under these conditions.

1.6.2 End Slope Removal Under Existing Bridge Abutment

For underpass widening through removal of the bridge abutment end slope, soil nailing offers the major advantage of not requiring soldier pile installation. Because of limited head room conditions beneath the bridge structure, soldier piles must generally be installed through the existing bridge deck, with significant disruption to the overpass traffic and increased cost.

In this application, soil nailing provides both the temporary and permanent earth support function. If lateral displacements are of particular concern (e.g., adjacent a bridge spread footing), the upper nail rows immediately adjacent the footing may be installed in slots to help limit and control the displacements.

1.6.3 Repairs and Reconstruction of Existing Retaining Structures

Soil nails can be installed through existing retaining walls and the technique is finding application in the stabilization or strengthening of existing failing or distressed retaining structures. Relevant applications to date include:

- Masonry or reinforced concrete retaining walls that have suffered structural deterioration or excessive deflections, often related to loose or weak backfill.
- MSE walls or crib walls that have deteriorated because of reinforcing corrosion or poor quality backfill.

This type of application represents something of a departure from the original soil nailing concept of excavate and support, in that the ground deformations required to mobilize the reinforcing loads do not derive from removal of lateral support during excavation but from ongoing movements associated with the distressed structure. In this context, soil nailing can also be similarly used to stabilize marginally stable slopes.

1.7 Ground Conditions Best Suited for Soil Nailing

In general, the economical use of soil nailing requires that the ground be able to stand unsupported in a vertical or steeply-sloped cut of 1 to 2 m in height for one to two days. In addition, it is highly desirable that an open drill hole can maintain its stability for at least several hours. In this context, the following ground types are considered suitable for soil nailing:

- Residual soils and weathered rock without unfavorably oriented, low strength structure.
- Stiff cohesive soils such as clayey silts and low plasticity clays that are not prone to creep.
- Naturally cemented or dense sands and gravels with some cohesion.
- Fine to medium homogeneous sands with capillary cohesions of at least 5 kN/m^2 associated with a natural moisture content of at least 5 percent. This soil type can sometimes exhibit face stability problems when south facing slopes are subject to drying by the sun.
- Above the ground water table.

Because of the construction flexibility of the method, soil nailing is also well suited to mixed-face conditions (including competent ground containing cobbles and boulders in which soldier beam installation is very difficult and expensive), providing each of the different materials is individually suited to soil nailing in accordance with the above.

1.8 Ground Conditions Not Well Suited for Soil Nailing

It is unfortunately sometimes the case that innovative techniques such as soil nailing are applied only when very difficult conditions that cannot be addressed by more standard techniques, arise. Such an approach is dangerous, both to the project and to future routine applications of the technique itself. As with most construction methods, soil nailing is not universally applicable and its limitations must be clearly understood. Very often, these limitations can be technically solved by appropriate design or construction provisions, but this often results in the method no longer being cost-effective. The following ground types or conditions are not considered well suited to soil nailing or limit it's application:

- Loose clean granular soils with field standard penetration N values lower than about 10 or relative densities of less than about 30 percent. These types of soils will not generally exhibit adequate stand-up time and are also sensitive to vibrations induced by construction equipment.
- Granular cohesionless soils of uniform size (poorly graded) with a uniformity coefficient of less than 2, unless in a very dense condition. During construction, these soil types will tend to ravel when exposed due to a lack of apparent cohesion.
- Soils containing excessive moisture or wet pockets such that they tend to slough and create face stability problems when exposed i.e., the apparent cohesion is destroyed. For most ground types, soil nailing below the water table is not appropriate as such conditions usually create very difficult construction conditions. In addition, care must be applied to the control of surface water and perched water.
- Organic soils or clay soils with a Liquidity Index greater than 0.2 and undrained shear strength less than 50 kN/m^2. Clay soils with a Liquidity Index greater than 0.2 or an undrained shear strength lower than 50 kN/m^2 may continue to creep significantly over the long term and may also exhibit a significant decrease in the soil-grout adhesion and nail pullout resistance if saturated following construction. Therefore, nails in such soils should exhibit satisfactory long-term creep behavior by a suitable testing program prior to their use in a soil nailing application.
- Highly frost-susceptible and expansive (swelling) soils. These soils can result in significant increases in the nail loading near the face; wall damage has been reported under these conditions. With frost-susceptible soils (e.g. silts), it is recommended that the design prevent frost from penetrating the soil by provision of an appropriate protective structure (e.g., granular or synthetic insulating layer) at the face. Water must be prevented from reaching expansive soils that are soil nailed.
- Highly fractured rocks with open joints or voids (including cavernous limestones) and open graded coarse granular materials (e.g., cobbles) require special care because of the difficulty of satisfactorily grouting the nails. Construction measures such as the use of geotextile nail socks or low slump grout can sometimes be used to advantage in such materials.

- Rock or decomposed rock with weak (e.g., gouge filled) structural discontinuities that are inclined steeply toward and daylight into the excavation face.

1.9 Cost Data

Costs for soil nail retaining structures are a function of many factors, including type of ground, site accessibility, wall size, facing type, level of corrosion protection, temporary or permanent application, and regional availability of contractors skilled in the construction of nail or tieback-type walls and shotcrete facings.

In Europe, it is reported that soil nailing costs in ground suitable to soil nailing are, in general, 20 percent lower than comparable tieback structures.

A major cost item for permanent walls is the facing. The addition of cast-in-place or precast facings placed over an initial 100 mm-thick construction shotcrete facing may be 40 to 50 percent of the total wall cost.

If installed in ground conditions well suited for soil nail wall construction (i.e., ground with good short-term face stability and in which open hole drilling methods can be used), soil nailing has proven to be a very economical method of constructing retaining walls. For cut retention, experience on U.S. highway projects indicates soil nail walls when used in ground well suited to soil nailing, can provide 10 to 30 percent cost savings versus permanent tieback walls or conventional cast-in-place walls with temporary shoring.

Typical cost range for nail walls based on U.S. highway project bidding experience to date is:

• Temporary Walls:	\$200-\$300/m^2
• Permanent Walls: - Roadway Cut	\$300-\$400/m^2
- End Slope Removal Under Existing Bridges	\$400-\$600/m^2

Cost and other project data for a number of U.S. highway soil nail wall projects are included in the following table 1.1. Costs are total in-place cost in dollars per square meter of wall face area. Project costs significantly higher than the above typical cost ranges are due to factors such as small project site, very difficult ground, difficult access, remote site or highly congested urban location, limited bidding competition, etc.

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Project	Interstate 78 Allentown, PA	Interstate 10 San Bernadino, CA	Cumberland Gap Tunnel Project, KY	Interstate 90 Seattle, WA
Bid	1987	1988	1988	1989
Wall Application	Retain slope cut on ¼:1 slope between two rock cuts.	Allow widening of existing I-10 exit ramp.	Retain cut slope above tunnel portal.	Retain temporary cuts next to existing tunnel portal.
Soil/Rock Type	Colluvium and highly weathered rock	Silty, gravelly sand with cobbles and boulders.	Colluvium and weathered rock.	Silty sand (upper) Stiff clay (lower)
Field SPT (blowcounts)	NP	25 to 80	NP	7 to 77
Max. Wall H (m)	12.2 (2-tiered walls, each 6.1 m high with 3m offset).	6.7	Wall 1 – 12.2 Wall 2 – 5.5	10.7
Wall Length (m)	61	183	Wall 1 – 152 Wall 2 – 40	111
Nail Spacing (m)	1.5H x 1.5V	1.5H x 1.5V	1.5H x 1.2V 1.8H x 1.2V	1.2H x 1.2V
Nail Length (m)	8.2 to 10.7	5.8 to 7.0	5.8 to 13.1	3.0 to 13.4
Nail Design Load (kN)	127 to 232	NP	12 to 193	22 to 149
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	218 to 446 ult.	69 ult.	143 ult. * 207 ult. **	55 Design
Corrosion Protection	Galvanized bars	Fully encapsulated	Epoxy coated bar	Grout only (temporary nails)
Shotcrete Thickness (mm)	140 to 170	200	130 to 170	200
Permanent Facing	Precast panels (VSL).	Exposed shotcrete hand finish.	Exposed shotcrete gun finish sandstone color coloring agent	Reinforced shotcrete gun finish.
Face Batter	Vertical	1 on 10	1 on 8	1 on 4
Total Area (m^2) Nail Wall Face (bid)	646	809	999	744
Cost/m^2	\$580	\$290	\$390	\$300
Remarks	\$390 (nails and shotcrete) \$190 (face panels).		* Weathered Shale * Weathered Sandstone	

NP = Not Provided in Information Made Available

Table 1.1 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Interstate 5 Tacoma, WA	Route 37 Vallejo, CA	Interstate 5 Portland, OR	Interstate Highway 35 Laredo, TX
Bid	1989	1990	1990	1990
Wall Application	Widening under existing Bridge. Bridge abutment on spread footing.	Cut retention – Route 37, New I/C.	Widening under existing bridge. Abutment on piles.	Widening under existing bridge. Abutment on shafts.
Soil/Rock Type	Glacially consolidated dense sand, gravel, cobbles, boulders.	Weathered and fractures mudstone, clayey silt and gravel.	Clean loose sand	Gravelly sand.
Field SPT (blowcounts)	NP	25 to 57	4 to 29	NP
Max. Wall H (m)	6.1	10.4	6.1	5.6
Wall Length (m)	44	276	78	59
Nail Spacing (m)	1.5H x 1.5V	0.6 to 1.5H 0.8 to 1.5V	1.4H x 0.9V	0.9H x 0.9V
Nail Length (m)	7.3	3.0 to 6.4	4.0 to 7.3	5.5
Nail Design Load (kN)	35 to 136	NP	9 to 153	44
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	72 Design	55 ult.	48 ult.	25 Design
Corrosion Protection	Fully encapsulated	epoxy coated bar	epoxy coated bar	epoxy coated bar
Shotcrete Thickness (mm)	230	100	200	50
Permanent Facing	Finished structural concrete with pigmented sealer.	150 mm CIP plus precast panels and masonry block veneer.	Exposed shotcrete Class 1 finish with horizontal and vertical scoring strips. Pigmented sealer.	230mm CIP face with 25mm Fractured Rib surface treatment.
Face Batter	Vertical	Vertical	1 on 12	Vertical
Total Area (m^2) Nail Wall Face (bid)	186	1620 (one wall)	382	205
Cost/m^2	$430	$510* $910**	$630	$340
Remarks	Nail wall and tieback wall bid as alternates. Tieback wall bid at $610/m	* Soil nail wall and facing. ** Cost including change order for additional longer nails to stop landsliding.		

NP = Not Provided in Information Made Available

Table 1.2 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Interstate Highway 35 Olympia Park, TX	GW parkway at I-495, VA	Route 85 San Jose, CA	Route 23A Hunter, NY
Bid	1990	1990	1990	1990
Wall Application	Widening under existing bridge.	Widening under existing bridge. Bridge abutment on spread footing.	Cut retention – new depressed freeway.	Slope retention to accommodate structure.
Soil/Rock Type	Clayey sand.	Dense micaceous silt and weathered schist.	Clay with gravels and cobbles.	Silty gravel, sandy with clay and boulders.
Field SPT (blowcounts)	NP	25 to 50	NA	4 to 55
Max. Wall H (m)	5.6	7.9	8.5	8.5
Wall Length (m)	38	198	604	146
Nail Spacing (m)	0.9H x 0.8V	1.2H x 1.2V (under abut.) 1.5H x 1.5V (outside abut.)	1.5 to 2.4H 1.4 to 2.4V	1.8H x 1.8V
Nail Length (m)	5.5	6.1 to 10.1	7.9	3.1 to 7.6
Nail Design Load (kN)	44	126 to 203	NP	35 to 269
Estimated Ground-Grout Bond (Adhesion) (kN/m²)	22 Design	48 Design	NP	219 Design
Corrosion Protection	Epoxy coated bar	Fully encapsulated	Epoxy coated bars	Epoxy coated bars
Shotcrete Thickness (mm)	50	180	100	279
Permanent Facing	230mm CIP face with 25mm Fractured Rib surface treatment.	180mm structural shotcrete. 150 mm CIP ribbed fascia wall.	200mm CIP with fractured fin texture.	CIP concrete and stone facing.
Face Batter	Vertical	Vertical	Vertical	1 on 12
Total Area (m²) Nail Wall Face (bid)	79	1358	4438 (two walls)	777
Cost/m²	$370	$580	$300 *$330	$748
Remarks		Soil nail wall VE substitute for tieback wall to eliminate traffic disruption caused by installing soldier piles through holes in bridge deck.	* Including change order for differing site conditions. NA – Not available.	Seepage at exc. face caused soil to slough prior to application of shotcrete.

NP = Not Provided in Information Made Available

Table 1.3 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Virginia Beach Toll Road at Independence Blvd, Virginia Beach, VA	I-66 over 495 Fairfax Co. VA	Minnesota Ave, NE Washington, DC	Highway 50 Sacramento, CA
Bid	1990	1990	1990	1991
Wall Application	Widening into bridge end slope, temporary wall.	Widening into bridge end slope.	Cut retention street widening.	Bridge end slope retention for freeway widening, spread footing abutment.
Soil/Rock Type	Silty clayey sands	Silt, silty clay	Silty clay	Silty sand and sandy silt with some clay
Field SPT (blowcounts)	7 to 13	7 to 13	7 to 25	16 to 100+
Max. Wall H (m)	5.0	6.0	3.5	4.7 at 1 ½ :1
Wall Length (m)	71	67	30	34
Nail Spacing (m)	1.8H x 1.8V	1.5H x 1.4V	0.9H x 0.9V	1.2H x 1.2V
Nail Length (m)	2.4	6.7	5.0	7.3
Nail Design Load (kN)	112	112	NP	NP
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	96 Design	96 Design	92 Design	42 ult.
Corrosion Protection	None	Fully encapsulated	None	Epoxy coated bars
Shotcrete Thickness (mm)	80	80	100	NA
Permanent Facing	None	250mm CIP	None	80mm CIP slope paving. 200mm CIP at bearing plate.
Face Batter	Vertical	1 on 12	Vertical	1 ½ on 1
Total Area (m^2) Nail Wall Face (bid)	317	330	100	257 (two slopes)
Cost/m^2	NP	$1000	$410	$450
Remarks	Temporary shoring for permanent CIP wall.	Field modification from auger/socketed soldier pile wall.	Temporary wall for construction.	

NP = Not Provided in Information Made Available

Table 1.4 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Industrial Parkway OC @ I-880 Hayward, CA	Highway 101 San Jose, CA	Route 85 San Jose, CA	Route 89 Tahoe Pines, CA
Bid	1991	1991	1991	1991
Wall Application	Cut retention at bridge overcrossing for on-ramp. Abutment on piles.	Cut retention – freeway widening.	Cut retention – new depressed freeway.	Stepped wall, cut retention – road widening.
Soil/Rock Type	Silt clay with minor gravel.	Silty clay to clayey silt with minor sand and gravel.	Silty, gravelly sand and sandy gravel with minor clay.	Silty decomposed granite sand with cobbles and boulders.
Field SPT (blowcounts)	9 to 27	6 to 63	20 to 100+	NP
Max. Wall H (m)	4.0	6.1	6.7	4.3 (two tiered wall)
Wall Length (m)	55	685	2035	193
Nail Spacing (m)	1.1 to 2.6H x 1.1V	1.5H x 0.9 to 1.1V	1.8H x 1.1V	1.5H x 0.9 to 1.5V
Nail Length (m)	3.0 to 5.5	3.7 to 7.6	3.0 to 6.7	3.7 to 4.6
Nail Design Load (kN)	NP	NP	NP	NP
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	42 ult.	69 ult.	69 ult.	69 ult.
Corrosion Protection	Epoxy coated bars	Epoxy coated bars	Epoxy coated bars	Epoxy coated bars
Shotcrete Thickness (mm)	100	100	100	130
Permanent Facing	150mm CIP.	200mm CIP with fractured fin texture.	200mm CIP with fractured fin texture.	180mm CIP with simulated rock texture.
Face Batter	Vertical	Vertical	Vertical	1 on 10 (5.7 degrees)
Total Area (m^2) Nail Wall Face (bid)	169 (one wall)	3234 (three walls)	8909 (four walls)	604 (two walls)
Cost/m^2	$520	$390	$330	$420
Remarks				

NP = Not Provided in Information Made Available

Table 1.5 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Route 400 Atlanta, GA	Tonawanda Dr. Route 54 San Diego, CA	Route 85 Los Gatos/Saratoga, CA	Route 680 Walnut Creek, CA
Bid	1991	1992	1992	1992
Wall Application	Cut retention freeway	Cut retention.	Cut retention – new depressed freeway.	Cut retention – freeway widening.
Soil/Rock Type	Granite gneiss and saprolite.	Clayey sand and silty sand.	Sandy silt to silty sand with some gravel and clay.	Silty clay, clayey silt and silty clayey sand, with minor siltstone and claystone.
Field SPT (blowcounts)	30 to refusal	NA	24 to 100+	14 to 100+
Max. Wall H (m)	7.6	7.9	6.6	7.0
Wall Length (m)	98	154	551	365
Nail Spacing (m)	1.5H x 1.5V	1.8H x 1.6V	1.2 to 1.8 H 0.5 to 1.5V	0.8 to 2.1 H 0.6 to 1.8V
Nail Length (m)	12.2	5.5 to 6.1	3.7 to 7.0	3.0 to 7.9
Nail Design Load (kN)	222	NP	NP	NP
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	98 Design	103 ult.	62 ult.	42 ult.
Corrosion Protection	Fully encapsulated	Epoxy coated bar	Epoxy coated bar	Epoxy coated bar
Shotcrete Thickness (mm)	100	100	100	100 to 130
Permanent Facing	305mm CIP	200mm CIP with textured surface.	200mm CIP with fractured fin surface.	150mm to 250mm CIP with fractured fin texture.
Face Batter	Vertical	Vertical	Vertical	Vertical
Total Area (m^2) Nail Wall Face (bid)	502	975	1732 (six walls)	1863 (three walls)
Cost/m^2	$364	$422	$380	$300
Remarks	Wall added by supplemental agreement and price negotiated.	Soil nail wall VE substitute for tieback wall. Cost savings $268/$m^2$		Tieback slide retention wall on same project. Bid at $830/$m^2$

NP = Not Provided in Information Made Available

Table 1.6 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Interstate 80 Berkeley, CA	Route 121 Napa County, CA	Route 85 San Jose, CA	Route 85 San Jose, CA
Bid	1992	1992	1992	1992
Wall Application	Cut retention – freeway widening.	Cut retention for climbing lane and saving heritage oak tree.	Cut retention – new depressed freeway.	Cut retention – new depressed freeway.
Soil/Rock Type	Silty clay and silty sand.	Very dense decomposed volcanic tuff and conglomerate.	Silty clay and sandy clay to clayey sand with minor gravel.	Clay and silty gravel to silty sand with gravel.
Field SPT (blowcounts)	2 to 70+	50 to 100+	13 to 50	4 to 100+
Max. Wall H (m)	4.8	9.9	6.1	7.3
Wall Length (m)	110	62	1159	939
Nail Spacing (m)	0.6 to 1.5H x 1.2V	1.5 to 1.8H 0.9 to 1.8V	1.6H x 0.6 to 1.2V	0.8 to 1.5H 0.6 to 1.1V
Nail Length (m)	4.3 to 4.9	3.7 to 6.7	3.0 to 6.1	4.9 to 9.8
Nail Design Load (kN)	NP	125 to 160	NP	NP
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	28 to 55 ult.	NP	NP	14 to 48 ult.
Corrosion Protection	Epoxy coated bar	Epoxy coated bar	Epoxy coated bar	Epoxy coated bar
Shotcrete Thickness (mm)	130	100	100	100
Permanent Facing	150mm CIP with fractured fin texture.	170mm CIP with brush hammered texture.	200mm CIP with fractured fin texture.	200mm CIP with fractured fin texture.
Face Batter	Vertical	1 on 6	Vertical	Vertical
Total Area (m^2) Nail Wall Face (bid)	314	301	4591 (two walls)	3434 (four walls)
Cost/m^2	$495	$536 $601*	$270	$230
Remarks			* Includes brush hammer and stain finish.	

NP = Not Provided in Information Made Available

Table 1.7 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Quioccasin Road Richmond, VA	Route 2 Dixon IL	Interstate 80 Elmswood Park, NJ	Route 217 Cedar Hills Interchange Portland, OR
Bid	1992	1993	1993	1993
Wall Application	Temporary cut retention for road widening.	Cut retention – freeway widening.	Widening under existing bridge abutment on piles.	Cut slope retention for light rail.
Soil/Rock Type	Silty sand weathered granite.	Sand, highly fractured and sound sandstone.	Silty, gravelly sand	Clayey silt (Portland Hills Silt) over cobbly gravel.
Field SPT (blowcounts)	6 to refusal	7 to 80	6 to 75	4 to 49
Max. Wall H (m)	7.0	7.6	4.6	8.1
Wall Length (m)	335 (2 walls)	84	78	335
Nail Spacing (m)	1.5H x 1.5V	1.5H x 1.5V	1.2H x 1.5V	1.8H x 1.8V
Nail Length (m)	3.0 to 6.1	5.5	6.4	3.7 to 7.0
Nail Design Load (kN)	90 to 180	156	53	12 to 23 (Portland Hills Silt) 42 to 81 (Rubble basalt)
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	60 Design	69 ult.* 345 ult.**	48 Design	14.6 Design (Portland Hills Silt) 51.1 Design (Rubble basalt)
Corrosion Protection	None	Epoxy coated bar	Galvanized	Fully encapsulated epoxy coated bars
Shotcrete Thickness (mm)	100	100	76	102
Permanent Facing	None	200mm CIP with texture.	305mm CIP	203mm CIP with random board finish.
Face Batter	Vertical	Vertical	Vertical	1:12
Total Area (m^2) Nail Wall Face (bid)	1285	446	316	1958
Cost/m^2	$234	$431	$1242	$411
Remarks	Temporary cut for permanent CIP wall.	* Weakly cemented highly fractured sandstone. ** Sound sandstone.		

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Table 1.8 Cost Data for Soil Nail Walls U.S. Transportation Projects

Step 9 - Check the Upper Cantilever

In accordance with section 6 Commentary, division I-A of the Standard Specifications for Highway Bridges, 15th Edition [30], equation C6-4, the approach developed by Mononobe and Okabe for a free-standing retaining structure is used to develop the active seismic loading on the upper cantilever. For a soil friction angle of 34°, a soil-to-wall interface friction angle of (2/3)34° = 22°, a vertical wall, a soil profile behind the wall as shown on figure 5.13, and a horizontal pseudo-static seismic coefficient of 0.32g (i.e. $0.8A_{PK}$), the combined static and dynamic active earth pressure load P_{AE} determined from a Mononobe-Okabe type computer solution is 9.14 kN/m length of wall. This can be considered to consist of a static earth pressure load of 2.95 kN/m (see step 9 for the static loading condition) and a dynamic earth pressure load of 6.19 kN/m. The load components normal to the wall are (2.95)cos(22°) = 2.74 kN/m (static) and (6.19)cos(22°) = 5.74 kN/m (dynamic).

Project	Route 101 Olympia, WA	Route 167 Seattle, WA	Interstate 90 Seattle, WA	Interstate 90 Seattle, WA
Bid	1993	1993	1993	1993
Wall Application	Roadway widening cut retention	Cut retention under bridge abutment for highway widening.	New depressed off-ramp.	Replacement of an old depressed concrete gravity wall.
Soil/Rock Type	Consolidated sand, silt; silty sand; silty gravelly sand.	Dense silty sand with layers of sandy silt.	Dense sandy gravel (fill) or clayey silt, clay or sandy clay (native ground).	Dense silty sand (backfill and glacial outwash)
Field SPT (blowcounts)	6 to 100+	21 to 100+	4 to 77	24 to 100+
Max. Wall H (m)	7.3	5.3	12.5	4.8
Wall Length (m)	236	121	187.4	57
Nail Spacing (m)	1.2 to 1.8H 0.9 to 1.2V	1.8H by 1.2V 1.6H by 1.2V*	1.2 to 1.8H 0.6 to 1.2V	1.5 to 1.8H 1.5 to 1.8V
Nail Length (m)	2.7 to 7.3	7.9 to 15.5	4.6 to 11.6	4.6 to 5.5
Nail Design Load (kN)	9 to 89	93 to 262	61 to 160	58 to 125
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	57 Design	69 Design	58 Design	71 Design
Corrosion Protection	Epoxy coated bar	Epoxy coated bar	Epoxy coated bar, fully encapsulated below footing.	Fully encapsulated
Shotcrete Thickness (mm)	150	150	150	250
Permanent Facing	180mm CIP with fractured fin finish.	180mm CIP with fractured fin finish.	180mm CIP with board finish and parapet.	140mm finished shotcrete over 110mm shotcrete.
Face Batter	Vertical	Vertical	1 on 10	1 on 12
Total Area (m^2) Nail Wall Face (bid)	1429	552	1414	214
Cost/m^2	$393	$336 w/ traffic barrier $334 w/o	$163	$462
Remarks		* Under bridge footing		Nails were installed through old wall prior to removing old wall face.

NP = Not Provided in Information Made Available

Table 1.10 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Interstate 5 Seattle, WA	Interstate 5 Tukwila, WA	Fort Baker – Fort Barry Tunnel Marin County, CO	Route 50 Cannon City, CO
Bid	1993	1993	1994	1994
Wall Application	Highway widening for HOV lanes.	Cut retention for highway widening for HOV lanes.	Temporary excavation support for tunnel reconstruction.	Cut retention highway widening and historic site protection.
Soil/Rock Type	Medium dense to very dense silty gravelly sand.	Dense glacial till, silty sand and gravel with cobbles and boulders.	Low to medium plastic clay, sandy clay, clayey gravel and clayey sand.	Sandy gravel with boulders.
Field SPT (blowcounts)	23 to 100+	3 to 42	8 to 50	NA
Max. Wall H (m)	7.8	2.6	7.9	4.9
Wall Length (m)	796 (five walls)	157	158	32
Nail Spacing (m)	1.2 to 1.5H 0.9 to 1.5V	1.8H (only one row of nails)	1.8H x 1.2 to 1.8V	1.2H x 1.2V
Nail Length (m)	1.7 to 11.9	3.9	3.7 to 9.8	4.0
Nail Design Load (kN)	9 to 144 (static) up to 179(dynamic)	58	129	174
Estimated Ground-Grout Bond (Adhesion) (kN/m²)	40 to 86 Design	72 Design	67 ult.	137 ult.
Corrosion Protection	Epoxy coated or fully encapsulated below br. fnd.	Fully encapsulated	None (temporary wall)	Epoxy coated bar
Shotcrete Thickness (mm)	180 or 200	120	80	80
Permanent Facing	150mm CIP with fractured fin texture.	Hand-trowel finished shotcrete 120mm to 200mm as-built.	None	100mm timber.
Face Batter	Vertical	Vertical	1 to 6	1 on 8
Total Area (m²) Nail Wall Face (bid)	3277 (five walls)	407	808	102
Cost/m²	\$341 w/ traffic barrier \$326 w/o	\$718	\$341	\$645
Remarks	Wall 11 has top 3 nail rows prestressed to 50% D.L. under bridge abutment spread foundation to limit deflection.	VE substitution, approximately \$146,000 savings over CIP wall with temporary shoring.	VE substitution for temporary tieback shoring. Cost savings \$112,000 (37%).	Cost includes timber facing.

NP = Not Provided in Information Made Available

Table 1.11 Cost Data for Soil Nail Walls U.S. Transportation Projects

Project	Interstate I-70 St Louis, MO	I-40/Route 220 Interchange Greensboro, NC	Albion Bridge Rehabilitation Lincoln, RI	Interstate 35 at Frio River, Pearsall, TX
Bid	1994	1994	1994	1995
Wall Application	Temporary Excavation support for freeway widening.	Cut retention below highway.	Slope retention at realigned bridge approach.	Retain existing embankment supporting highway.
Soil/Rock Type	Lean clay.	Severely to lightly decomposed granite.	Fine to coarse sand with silt and gravel.	Sand, clay, and gravel.
Field SPT (blowcounts)	NA	4 to 100+	10 to refusal	10 to 40
Max. Wall H (m)	5.5	6.4	5.5	4.6
Wall Length (m)	46	85	49.4	152
Nail Spacing (m)	1.2H x 1.2V	1.5H x 0.6 to 1.5V	1.2H x 1.2V	1.3H x 1.1V
Nail Length (m)	10.1	3.0 to 5.2	9,2	3.0 to 6.4
Nail Design Load (kN)	28	123 to 150	NP	62
Estimated Ground-Grout Bond (Adhesion) (kN/m^2)	16 ult.	97 to 138 Design	98 Design	62 Design
Corrosion Protection	None	Epoxy coated bar	Fully encapsulated	Epoxy coated bar
Shotcrete Thickness (mm)	100	80	220	76
Permanent Facing	None	200mm CIP	Granite rubble veneer	229mm CIP
Face Batter	Vertical	Vertical	1 on 16	Vertical
Total Area (m^2) Nail Wall Face (bid)	231	400	161	539
Cost/m^2	$459	$777	$1098*	$393
Remarks	VE substitution for tieback wall.	VE substitution for anchored shaft. Cost savings of $53/m	*Soil nailed wall only. First soil nail wall in Rhode Island.	Replacing gravity wall.

NP = Not Provided in Information Made Available

Table 1.12 Cost Data for Soil Nail Walls U.S. Transportation Projects

CHAPTER 2. DESCRIPTION OF SOIL NAILING AND BASIC MECHANISMS

2.1 Background

Soil nailing has been used in a variety of civil engineering projects in the last two decades. The technique originated as an extension of rock bolting and of the "New Austrian Tunneling Method" (NATM) developed by Rabcewicz, which combines reinforced shotcrete and rockbolting to provide a flexible support system for the construction of underground excavations. In North America, the first recorded application of the system was in Vancouver, B.C. in the early 1970s for temporary excavation support. Mason filed for a US Patent for soil nailing on April 1, 1970, and walls were built under his supervision in Vancouver, BC, Washington, DC, and Mexico City in the late 1960s, prior to submission of the patent application. In Europe, the earliest reported works were retaining wall construction in Spain (1972), France (1972/73), and Germany (1976), in connection with highway or railroad cut slope construction or temporary building excavation support.

The French contractor Bouygues, in joint venture with the specialist contractor Soletanche, is credited with the first recorded application of soil nailing in Europe (1972/73) for an 18-m-high 70° cut slope in Fontainebleau Sand, as part of a railway widening project near Versailles. A total of 12,000 m^2 of face was stabilized by over 25,000 steel bars grouted into pre-drilled holes up to 6m long.

The excavation for the foundation of the extension to the Good Samaritan Hospital in Portland, Oregon, executed in 1976 by a joint venture of Kulchin and Associates Inc. and Albert K. Leung and Associates, was the first published application of soil nailing in the United States [6]. The maximum depth of excavation was 13.7 m. The ground consisted of medium dense to dense silty fine lacustrine sands with a friction angle about 36° and cementation giving a "cohesion" of 20 kN/m^2. No groundwater was encountered. It was noted that the work was conducted in 50 to 70 percent of the time required for conventional excavation support and at about 85 percent of the cost. Given the time savings and the facility to cast the final wall straight on, the overall support and wall system costs were claimed to have been reduced by about 30 percent.

The first major research program (Bodenvernagelung) on soil nailing was undertaken in Germany by the University of Karlsruhe and the contractor Bauer (1975-1981). This jointly funded program cost approximately $2.3 million (US$1=DM1.5) and involved full-scale testing of a variety of experimental wall configurations. As a result of the increasing use of soil nailing within France following its initial applications and the perceived lack of a defensible design methodology, the French initiated their own experimental program (Clouterre) in 1986. The Clouterre program was jointly funded by the French government and private industry, with a budget on the order of $5 million and included 21 individual private and public participants. The program involved three large-scale experiments in prepared fill (Fontainebleau sand) and the monitoring of six full-scale, in-service structures. The results of the Clouterre program have recently been published and will form the basis for the future soil nailing design approach to be adopted in France.

Within the 20 years since its first introduction to Europe and the subsequent conduct of the two major national experimental programs, soil nailing has been and is now used very extensively in both France and Germany. The major attractions of the method are its economy, construction flexibility, ability to make use of small construction equipment that is particularly suited for use in urban environments, and its overall adaptability for special applications. Within France, it is reported that over 100,000 m^2 per year of soil nail walls are presently being constructed for public works alone, with perhaps hundreds of smaller undocumented walls constructed for private owners. To date, the great majority of these walls have been temporary in nature and have used shotcrete for the structural facing. The highest vertical soil nail wall in France is 22 meters high (at Montpelier). The highest battered soil nail wall (73 degree face angle) is almost 30 meters high (Dombes tunnel portal, near Lyon). In Germany, over 500 walls are estimated to have been constructed to date, with the majority being temporary basement walls using structural shotcrete facings.

Applications of soil nail walls in Europe and North America include permanent and temporary stabilization of natural slopes, renovation of old failing retaining walls, renovation of mechanically stabilized earth walls that have undergone premature corrosion deterioration of the originally installed <u>stainless</u> **steel** reinforcing elements (France), temporary shoring for basement excavations, foundation walls for buildings, and permanent and temporary support for cuts associated with roads, railways, and tunnel portals. The most common form of nail installation throughout Europe and North America involves grouting the nails into uncased pre-drilled holes. Within Germany, this installation method is used almost exclusively, but there have also been limited applications of a post-grouted driven nail with an oversized head to create an open annulus between the steel bar and the surrounding soil. In France, in addition to the conventional drill and grout installation method, relatively extensive use is also made of percussion-driven closely-spaced nails without post-grouting, together with the specialist technique of jet nailing. Within Great Britain, there is one specialist contractor using explosive injection of nails using compressed air. This last technique appears to have been used almost exclusively for doweling marginally stable slopes.

The major use of soil nailing in the U.S. to date has been for temporary excavation support for building excavations in urban areas. During the past few years, use of soil nailing for temporary excavation support has become common practice in some U.S. cities. For example, Kulchin-Condon Construction Company, a San Francisco-based shoring specialty contractor, has reportedly done over 200 soil nail projects with walls up to 18 m high. Schnabel Foundation Company has constructed soil nail walls in 21 different States, with over one-quarter of these for permanent applications. Several soil nail walls have now been constructed in building excavations in Seattle, one up to 23 m deep, which is the highest in the U.S. to date. In addition, a new soil nailing technology in design and construction was developed in the Seattle area where the permanent walls for basement excavations have been constructed from the top-down during the excavation stage of construction. This has resulted in substantial savings related to schedule time reductions and reduced direct costs in the materials for the wall. Table 1.1 summarizes some representative case history examples illustrating the use of temporary and permanent nail walls on U.S. transportation projects (through the middle of 1994).

There are no proprietary restrictions on the use of the soil nailing concept. However, some specific systems of nails and/or facing are patented. A recently patented (by Soil Nailing Limited, United Kingdom) soil-nailing technique inserts reinforcing nails into the ground by means of a compressed-air "launcher." Various nail installation techniques such as the French-developed "HURPINOISE" and "Jet Nailing" techniques, are patented. One U.S. specialty contractor has taken out a patent on the use of soil nails and tie-backs for repair of existing walls such as corrugated metal bin walls (Schnabel Foundation Co. Patent No. 4, 911, 582 March, 1990).

2.2 Construction Sequence

The following is the typical sequence to construct a soil nail wall using the drill and grout method of nail installation, which is the most common method used in North America (Figure 2.1).

A. Excavate Initial Cut

Before commencing excavation, it is necessary to ensure that all surface water will be controlled during the construction process. This is usually done by the use of collector trenches to intercept and divert surface water before it can impact the construction operations. The initial cut is excavated to a depth slightly below the first row of nails, typically about 1 to 2 m depending on the ability of the soil to stand unsupported for a minimum period of 24 to 48 hours. Where face stability is problematical for these periods of time, a stabilizing berm can be left in place until the nail has been installed and final trimming then takes place just prior to application of the facing. Another method of dealing with face stability problems includes placing of a flash coat of shotcrete. It is generally the case that face stability problems are likely to be most severe during the first one or two excavation stages, because of the presence of near-surface weathered and weakened materials or, in urban environments, the presence of loose fills or voids often associated with buried utilities.

Mass excavation is done with conventional earth moving equipment. Final trimming of the excavation face is typically done with a backhoe or hydraulic excavator. Usually, the exposed length of the cut is dictated by the area of face that can be stabilized and shotcreted in the course of a working shift. Ground disturbance during excavation should be minimized and loosened areas of the face removed before shotcrete facing support is applied. The excavated face profile should be reasonably smooth and regular in order to minimize subsequent shotcrete quantities.

A level working bench on the order of 10 m width is typically left in place to accommodate the drilling equipment used for nail installation. Smaller tracked drills are available that can work on bench widths as narrow as 5 m and with headroom clearance as low as 4 m. Larger bench widths may be necessary depending upon the equipment to be used during nail installation.

B. Drill Hole for Nail

Nail holes are drilled at predetermined locations to a specified length and inclination using a drilling method appropriate for the ground. Drilling methods include both uncased methods for more competent materials (rotary or rotary percussive methods using air flush, and dry auger methods) and cased methods for less stable ground (single tube and duplex rotary methods with air or water flush, and hollow stem auger methods). Typical nail spacings are 1 to 2 m both vertically and horizontally. Typical nail lengths

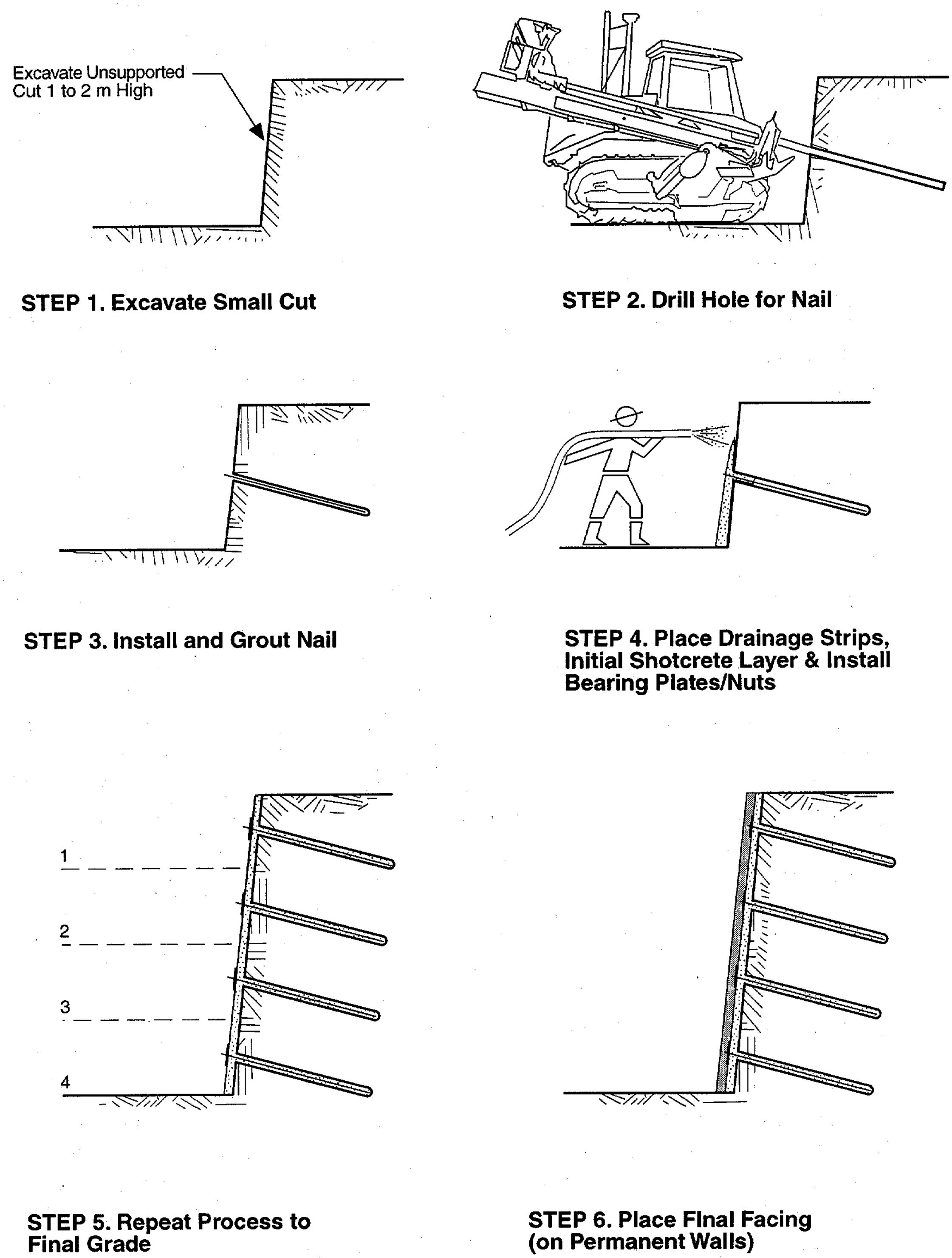


Figure 2.1 Typical Nail Wall Construction Sequence

are 70 to 100 percent of the wall height and nail inclinations are generally on the order of 15 degrees below horizontal to facilitate grouting.

C. Install and Grout Nail

Plastic centralizers are commonly used to center the nail in the drillhole. However, where the nails are installed through a hollow stem auger, centralizers are generally ineffective and a stiffer (200 mm or lower slump) grout mix is used to maintain the position of the nail and prevent it from sinking to the bottom of the hole. The nails, which are commonly 19 to 35 mm bars (yield strength in range of 420 to 500 N/mm^2), are inserted into the hole and the drillhole is filled with cement grout to bond the nail bar to the surrounding soil. However, nail sizes smaller than 25 mm can cause installation problems for moderate-long nail lengths due to their low stiffness. Grouting takes place under gravity or low pressure from the bottom of the hole upwards, either through a tremie pipe for open-hole installation methods or through the drill string (or hollow stem) or tremie pipe for cased installation methods.

For permanent nails, the steel bar is typically protected against corrosion damage with a heavy epoxy coating or by encapsulation in a grout-filled corrugated plastic sheathing.

D. Place Drainage System

A 400 mm-wide prefabricated synthetic drainage mat, placed in vertical strips between the nail heads on a horizontal spacing equal to that of the nails, is commonly installed against the excavation face before shotcreting occurs, to provide drainage behind the shotcrete face. The drainage strips are extended down to the base of the wall with each excavation lift and connected either directly to a footing drain or to weep holes that penetrate the final wall facing. These drainage strips are intended to control seepage from perched water or from limited surface infiltration following construction. If water is encountered during construction, short horizontal drains are generally required to intercept the water before it reaches the face.

E. Place Construction Facing and Install Bearing Plates

The construction facing typically consists of a mesh-reinforced wet mix shotcrete layer on the order of 100 mm thick, although the thickness and reinforcing details will depend on the specific design. Following placement of the shotcrete, a steel bearing plate (typically 200 mm to 250 mm square and 19 mm thick) and securing nut are placed at each nail head and the nut is hand wrench tightened sufficiently to embed the plate a small distance into the still plastic shotcrete.

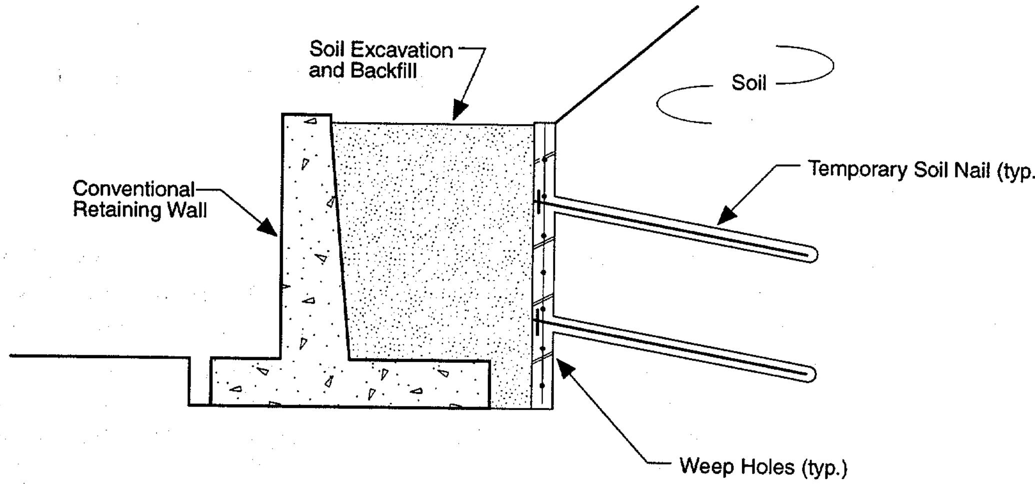
F. Repeat Process to Final Grade

The sequence of excavate, install nail and drainage system, and place construction facing is repeated until the final wall grade is achieved. The shotcrete facing may be placed at each lift prior to nail hole drilling and nail installation, particularly in situations where face stability is a concern.

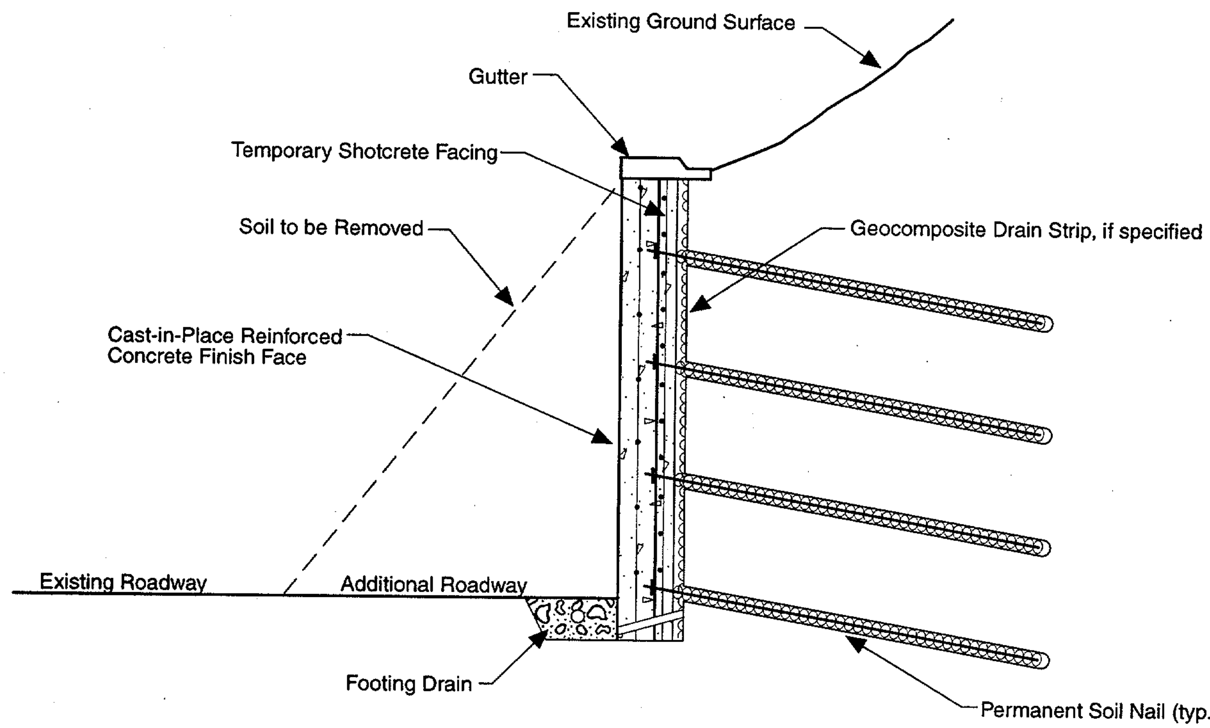
G. Place Final Facing

For architectural and long term structural durability reasons, a CIP concrete facing is the most common final facing being used for transportation applications of permanent soil nail walls. The CIP facing is typically structurally attached to the nail heads by the use of headed studs welded onto the bearing plates. Under appropriate circumstances, the final facing may also consist of a second layer of structural shotcrete applied following completion of the final excavation. Pre-cast concrete panels may also be used as the final facing for soil nail walls.

Typical examples of permanent and temporary soil nail wall sections for a cut slope stabilization and for a road widening under an existing bridge, are shown on figures 2.2 and 2.3 respectively.



Temporary Shoring



Highway Widening and Traffic Lane Additions

Figure 2.2 Permanent and Temporary Cut Slopes

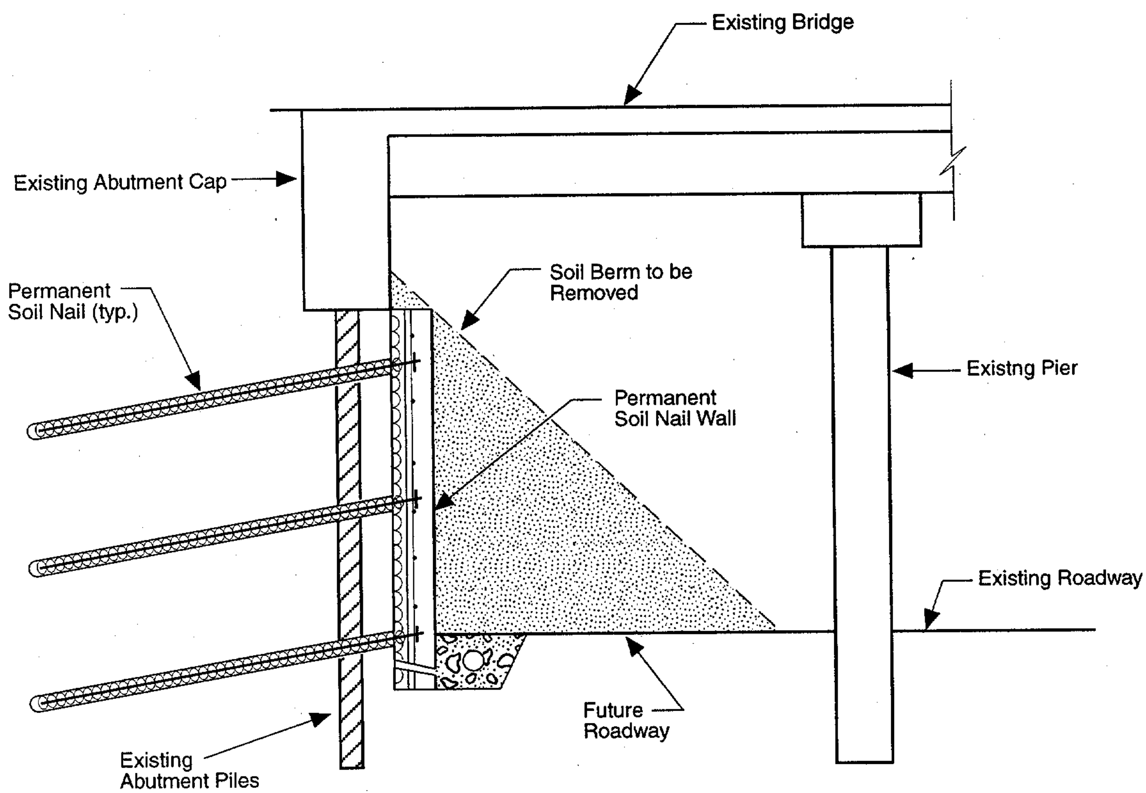


Figure 2.3 Widening Under Existing Bridge

2.3 Construction Equipment and Materials

The construction equipment required for soil nail walls includes the usual excavating and earth moving equipment, drilling equipment for the installation of the nails, shotcreting equipment for the application of the construction facing, and the normal equipment used for the placement of cast-in-place concrete facings if such facings are used for the permanent wall. The materials required include the nails themselves and the associated corrosion protection systems, the nail grout, the drainage materials, the shotcrete/concrete facing materials, and the system for providing a connection between the nail head and the facing.

A. Drilling Equipment and Methods

To date, the principal method of nail installation in North America has been drilling and grouting and, because of corrosion protection considerations, this method is likely to be the predominant method of nail installation for permanent nails on transportation projects in the United States. Drillhole sizes range from about 100 mm to 300 mm in diameter. This usually permits a grout annulus of at least 30 mm thickness around the reinforcement. Typical drilling equipment and methods are summarized in table 2.1.

Most soil nailing is undertaken using small hydraulic, track-mounted drill rigs. These rigs are mostly of the rotary/percussive type that use sectional augers or drill rods. Large hydraulic-powered track-mounted rigs with continuous flight augers have been used to install nails up to 28 m in length. These rigs have the advantage that they can drill the entire length of the nail in a single pass without having to stop to add additional sections of auger. Production rates can therefore be very high. Their main disadvantages are the high mobilization costs and the large space requirements for their operation.

For ground conditions that are well suited to soil nailing, the rate of nail installation will be increased (and the costs correspondingly reduced) by drilling drillholes without casing. This is commonly referred to as the "open-hole" method. Open hole methods suitable for soil nailing are augering, rotary drilling with a drag bit or roller-cone bit, and rotary-percussive drilling. Air is the preferred medium to remove drill cuttings, since flushing with water in uncased holes can significantly lower the bond stress developed during subsequent grouting, particularly in moisture sensitive soils such as silts and clays. The nail grout is subsequently introduced to the drillhole using a tremie pipe to place the grout from the bottom to the top of the drillhole as the pipe is slowly withdrawn.

Where ground conditions are such that the drillhole wall will not stand unsupported for the length of time required to insert and grout the nail, the drilling method must provide casing support to the drillhole. It should be noted that such conditions will often have serious adverse impacts on the cost effectiveness of soil nailing. Such methods include single tube and duplex rotary methods with air or water flush, and hollow stem augers. Flushing with air should be performed at moderate velocities and volumes to ensure that ground fracturing does not occur. This precaution is especially important in residual soils

TABLE 2.1
DRILLING METHODS AND PROCEDURES

Drill Rig Type	Drilling Method	Open Hole	Cased or Auger-Cast	Drillhole Diameters (mm)	Drill Bit Types	Cuttings Removal	Comments
Auger	Lead Flight Kelley-bar Driven	Yes	No	100-300	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock. Predominant drilling method for soil nail installation work.
	Sectional Solid-stem	Yes	No				
	Sectional Hollow-stem	Yes	Yes			Mechanical (air support)	
	Continuous Flight Solid-stem	Yes	No			Mechanical	
	Continuos Flight Hollow-stem	Yes	Yes			Mechanical (air support)	
Rotary	Single-stem Air Rotary	Yes	No	100-200	Button, Roller, Drag, etc.	Compressed air	Hydraulic rotary methods for drilling competent soils, rock, or mixed ground conditions (pneumatic hammers available).
	Duplex Air Rotary	Yes	Yes				
	Sectional Solid-stem Augers	Yes	No	100-300	Rock, Soil, Drag, etc.	Mechanical	Hydraulic rotary auger methods for drilling competent soils or weathered rock.
	Sectional Hollow-stem Augers	Yes	Yes			Mechanical (air support)	
Air Track	Single-stem Air Rotary	Yes	No	100-300	Button, Roller, Drag, etc.	Compressed Air	Pneumatic rotary methods for drilling non-caving competent soils or rock. Drillholes cannot be cased with this method.

or badly weathered rock where fissures or fracture planes exist. With hollow-stem augers, care must be taken to ensure that "mining" of the ground does not occur. The nail grout is introduced to the drillhole either through a tremie pipe or through the casing, and placed from the bottom to the top of the hole as the pipe and casing are slowly withdrawn. Based on experience with ground anchors, the temporary support of drillholes by bentonite or other mud suspensions is not recommended, as "smear" on the drillhole walls can significantly reduce the grout-to-ground bond.

B. Types of Nails

Soil nails typically consist of steel reinforcement inclusions and may be categorized on the basis of their method of installation and degree of corrosion protection. As noted earlier, the drill and grout method of nail installation has been used almost exclusively in North America. Nails installed by this technique may be further subdivided into 1) grouted nails and 2) corrosion protected nails. The steel bars are typically 25 to 35 mm in diameter, with a yield strength in the range of 420 to 500 N/mm^2, and are typically installed into drillholes having diameters in the range of 100 mm to 300 mm and at a spacing between 1 and 2 m. Steel bar diameters smaller than 25 mm are not recommended due to difficulties associated with placement of such flexible tendons in drilled holes.

Grouted nails are suitable for temporary construction and, where soils are not generally highly corrosive, they are also accepted for permanent applications in some countries. In these circumstances, a sacrificial steel thickness is commonly allowed to account for the corrosion losses that are expected to occur over the life of the structure. Nails protected only by the grout annulus are not generally considered adequate for permanent application on U.S. transportation projects. However a particular nail type called "self-drilling nails" have been used routinely outside of the U.S. where open hole drilling is not possible or practical. These types of nail require special corrosion considerations and testing procedures to be considered for permanent applications on highway projects. In general, the self-drilling nail products should not be used in aggressive ground (as defined later in the FHWA Soil Nail Manual), and coatings (including galvanizing, epoxy and metalizing) should not be considered acceptable corrosion protection. These nails should only be used for permanent applications if corrosion protection can be assured by providing sacrificial steel, ie. using an over-sized tendon. In addition, the proof testing schedule should be changed from testing 5% of the nails to testing every nail as self-drilling nails do not permit the same level of inspection as nails installed in bored holes. Self-drilling nail tendons should be sized to allow for proof testing of the fully grouted nail to the required 125% to 150% of capacity.

Corrosion protected nails are as described above, except that the steel bar is further protected against corrosion by either an epoxy coating or by encapsulation within a cement grout-filled plastic sheathing.

C. Nail Grout

For conventional drill and grout nail installations, the nail grout consists typically of a neat cement grout with a water-cement ratio of about 0.4 to 0.5. Where a stiffer consistency grout is required (e.g., to centralize the nail when no centralizers are used in a hollow stem auger installation or to control leakage of grout into the ground such as in highly permeable granular soils or highly fractured rock), a lower slump sand-cement grout may be used. Sand-cement grout may also be used in conjunction with large nail holes for economic reasons.

D. Drainage Systems

Ground water is a major concern in both the construction of soil nail retaining walls and in their long-term performance. Soil nail walls are best suited to applications above the water table. Excess water at the face can result in face stability problems during construction together with an inability to apply a satisfactory shotcrete construction facing. In addition, long-term face drainage is required to prevent the generation of localized high groundwater pressures on the facing. Section 4.9 contains drainage details.

E. Construction and Final Wall Facings

The facing consists of two component parts and is defined primarily in terms of the timing of construction. The **"construction facing"** is the facing erected during excavation and initial construction of the wall and is most commonly a minimum 100-mm-thick mesh-reinforced wet mix shotcrete, as this system provides a continuous, flexible surface layer over the excavated soil face. The wet mix shotcrete process is preferred to the dry mix process and is used almost exclusively for soil nail wall facings. The advantages of the wet mix process include better quality control of the water content (water-cement ratio of about 0.45 to 0.50), the ability to air-entrain for improved freeze-thaw durability, and ready availability from local ready-mix plants. From a design perspective, the construction facing may or may not be considered to contribute to the structural capacity of the final facing.

Final facings are usually installed following completion of the excavation to final grade, although in some applications (e.g., basement walls where aesthetics are of secondary concern) the final facing, consisting totally of shotcrete, has been constructed full thickness concurrent with the excavation of each lift. The most common final facing used to date on permanent walls is cast-in-place (CIP) reinforced concrete (typically 200 mm minimum thickness), as this type of facing can be readily adapted to satisfy a variety of aesthetic and durability criteria. Less commonly, a second layer of shotcrete has also been used as the final facing. Present technology is such that the final shotcrete layer can be controlled to close tolerances and, with the shotcrete application and nominal hand finishing performed by experienced structural shotcreters, an appearance similar to a CIP wall can be obtained, although the architectural possibilities are necessarily much more limited. In addition, the shotcrete can be colored either by adding coloring agent to the mix or by applying a pigmented sealer or stain over the shotcrete surface.

Precast concrete facing panels have also been used as final facings, and can be attached to the construction facing in a variety of ways. The precast panels can consist of smaller modular units or of full-height tilt-up panels and provide the means of integrating a continuous drainage blanket behind the facing. One disadvantage of this type of system is the difficulty of providing adequate long-term corrosion protection to all the attachment/connection devices. A further disadvantage of the smaller modular panels is the difficulty of attaching the panels to the nail heads. Some systems are also restricted by patent. A disadvantage of the full-height precast panels is that they are practically limited to wall heights of about 8 m because of weight and handling limitations. Galvanized welded wire mesh has also been used as a final facing with cemented and generally non-erodible materials and where the slope being stabilized is on a flatter slope angle, e.g.. 1H:1V or flatter.

Sections 4.10.15 through 4.10.19 contain the following information related to wall facings:

- External loads on wall facings,
- Design of support for facing dead load,
- End of wall transitions,
- CIP concrete form connects for shotcrete facing,
- Aesthetic issues.

2.4 Behavior of Soil Nail Walls

2.4.1 Fundamental Mechanism and Potential Failure Modes of a Soil Nail Wall

The fundamental mechanism of Soil Nail Retaining Structures is the development of tensile forces in the "passive" reinforcements as a result of the restraint that the reinforcements and the attached facing offer to lateral deformations of the structure. In the case of a soil nail wall constructed from the top-down, the lateral expansion of the reinforced zone is associated with removal of lateral support as excavation proceeds following installation of each level of reinforcement. In the case of repair of existing retaining structures or the stabilization of marginally stable slopes, the lateral deformations are associated with ongoing movements of the distressed wall or slope as a result of inadequate support. In either case, however, the reinforcements interact with the ground to support the stresses and strains that would otherwise cause the unreinforced ground to fail. These reinforcements are oriented to correspond in general with the direction of maximum tensile straining within the soil so that the generation of tensile loads is dominant. This distinguishes soil nailing from soil doweling. In soil doweling, the reinforcements tend to be installed across potential slip surfaces so as to derive significant support from the development of shear forces within the dowel.

Loads are developed within the soil nails primarily as a result of the frictional interaction between the nail and the soil, and secondarily by the soil-structure interaction between the facing and the soil. The latter phenomenon is responsible for the development of tensile load at the head of the nail (i.e., at the connection between the nail and facing), and this nail head load is typically some fraction of the maximum nail load. The maximum tensile load within each nail occurs within the body of the reinforced soil at a distance from the facing that depends on the vertical location of the nail within the wall. The line of maximum tension within the nails is often considered as dividing the soil mass into two separate zones [7] - (a) an "active zone", close to the facing, where the shear stresses exerted by the soil on the reinforcement are directed outward and tend to pull the reinforcement out of the ground and (b) a "resistant zone", where the shear stresses are directed inward and tend to restrain the reinforcements from pullout. This behavior is shown conceptually on figure 2.4. It should be noted that the line of maximum tension does not correspond to the conventional critical slip surface, as defined in limiting equilibrium stability analyses, but reflects the results of the soil structure interactions between the ground and the nail/facing reinforcement system.

The reinforcement acts to tie the active zone (that would otherwise fail by moving outwards and downwards with respect to the resistant zone) to the resistant zone. For stability to be achieved, the nail tensile strength must be adequate to provide the support force to stabilize the active block. The nails must also be embedded a sufficient length into the resistant zone to prevent a pullout failure. In addition, the combined effect of the nail head strength (as determined by the strength of the facing or connection system) and the pullout resistance of the length of the nail between the face and the slip surface must be adequate to provide the required nail tension at the slip surface (interface between active and resistant zones).

The above is demonstrated by reference to figure 2.5. Figure 2.5(a) shows a soil nail wall with long nails of high tensile strength. For a modest strength facing (or nail-facing connection) system, the most likely failure mode of the wall is for the facing or connection to fail (e.g., in flexure or shear) and for pullout then to occur along the portion of the nails located within the "active" zone of the reinforced soil mass, i.e., the active zone slides off the front of the nails. The high nail tendon tensile strength prevents nail tensile failure and the long nail length prevents the nails from pulling out of the resistant zone behind the active block.

Figure 2.5(b) shows a soil nail wall with nails of high tensile strength but of more limited length. In this case, the most likely failure mode is pullout of the nails from the resistant zone. The nail force available to support the active block is dependent on the length of nail behind the potential slip surface and on the limiting unit pullout resistance that can be developed between the nail and the soil. In this case, the pullout resistance within the resistant zone is less than the nail tensile strength and is also less-than the facing strength/pullout resistance within the active zone.

Figure 2.5(c) shows a soil nail wall with long nails of more modest tensile strength, and a high capacity facing system. In this case, nail tensile failure will occur before either the face strength is exceeded or pullout from the resistant zone can occur.

All potential failure modes must be considered in evaluating the available nail force to stabilize the active block defined by any particular slip surface.

2.4.2 Nail-Ground Interaction

As noted above, the principal interaction between the nail and the ground is the development of shear stresses along the nail-ground interface as the ground strains laterally toward the excavation face and the nail develops resistance to that expansion. The distribution of tension and interface shear along the nail depends on the unloading stiffness of the ground, the initial stress field, the vertical location of the nail, the nail length, the nail inclination, the tensile stiffness of the nail, and the nail-ground interface stiffness. Local equilibrium of a section of reinforcement within the ground shows that the rate of change of tension along the length of the nail is equivalent to the mobilized shear force per unit length at that point, and can be expressed mathematically as:

$$dT/dL = \pi\, D\, \tau = Q$$

where:

- dT = change of nail tensile force over the length dL
- D = outside diameter of the nail drillhole (steel bar and grout)
- τ = mobilized shear stress at the grout-ground interface
- Q = mobilized shear force per unit length of nail (pullout resistance)

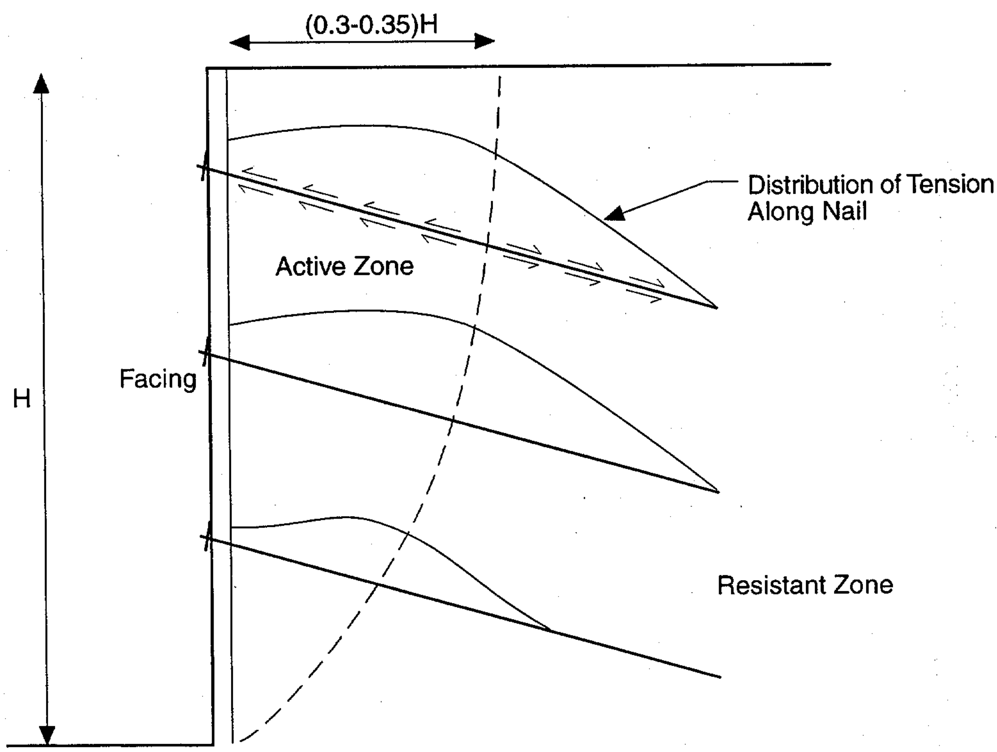
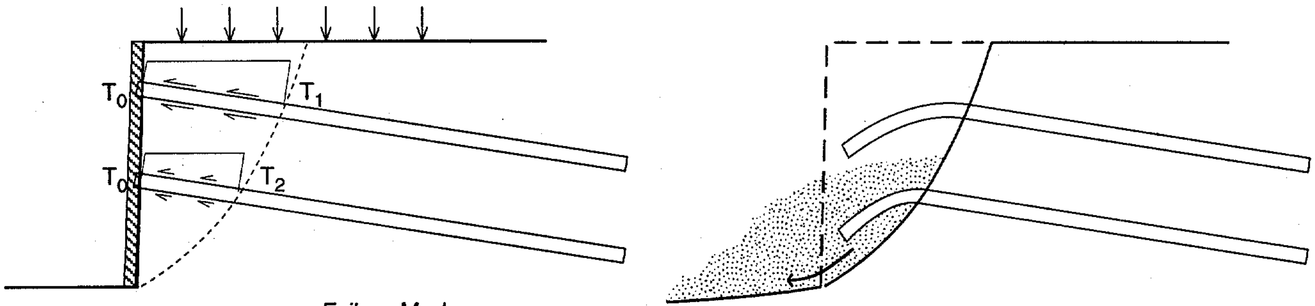


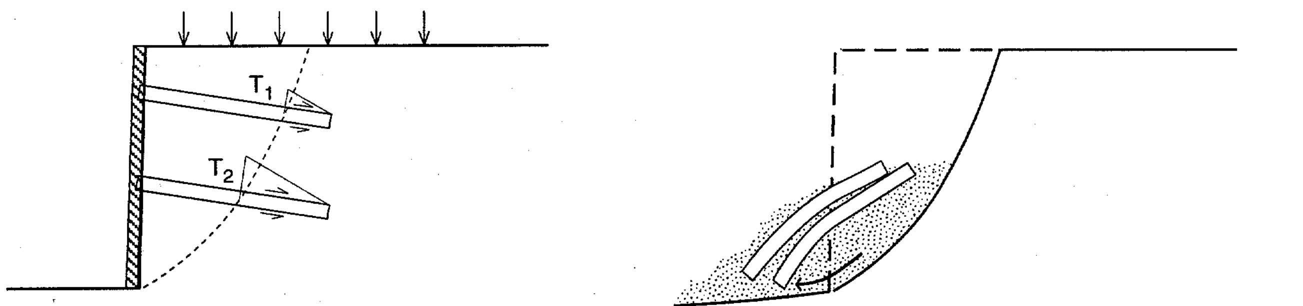
Figure 2.4 Conceptual Soil Nail Behavior



a) Face Failure

Failure Mode -

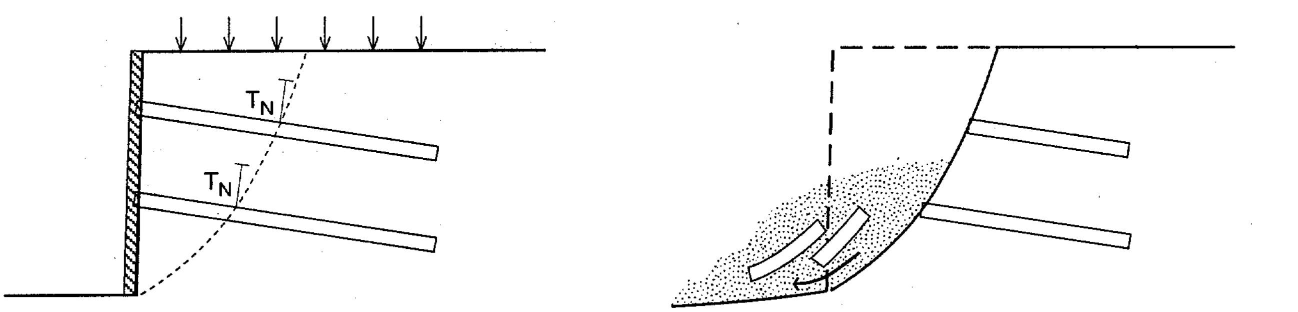
Face failure and "active" zone sliding off front of nails



b) Pullout Failure

Failure Mode -

Nails pull out of resistant zone



c) Nail Tendon Failure

Failure Mode -

Tensile failure of tendon

Figure 2.5 Potential Soil Nail Wall Failure Modes

From a strength perspective, the ultimate pullout resistance (force per unit length) at the nail-ground interface is an important parameter for the performance of a soil nail reinforcement system. The ultimate pullout resistance depends on the nail drillhole diameter (which can be controlled within limits) and the shear stress that can be developed before interface slip occurs, and is a function not only of the ground type but also of the method of nail installation.

The ultimate pullout resistance tends to be relatively independent of depth below surface for a constant soil type and particular installation technique [7]. This relative lack of dependence of the ultimate pullout resistance on the level of overburden pressure has been attributed to the decreasing significance of soil dilation with depth, which tends to offset the effect of increasing overburden pressure with depth. For nails that are installed in predrilled holes and grouted under gravity or low pressure, which is the most common method in North America, the effect of overburden pressure tends to be diminished by arching of stresses around the installation drillhole. This phenomenon also contributes to the observed lack of significant dependence of the ultimate pullout resistance on the overburden pressure.

Finally, only very small displacements between the nail and the adjacent ground are required to mobilize the ultimate bond or adhesion. Numerous pullout tests have established that relative displacements on the order of 1 to 2 mm are often all that are required to achieve the ultimate pullout resistance. This is an important aspect in the performance of soil nail structures. Not only must the reinforcements be sufficiently strong to prevent nail tendon tensile failure, and the ground-nail ultimate pullout resistance sufficiently high to prevent pullout of the reinforcements, but the nails and the nail grout-ground interface must be sufficiently stiff to ensure that the reinforcing loads can be developed without associated excessive deformations.

2.4.3 Nail-Ground-Facing Interaction

Although soil nail reinforcement systems have been used without any structural facing [8], a construction facing is most typically developed concurrently with excavation and nail installation, and the nails are structurally connected to that facing. Hence, as the excavation proceeds and lateral deformations of the ground occur, earth pressures are developed at the ground-facing interface. These face pressures are balanced by equal and opposite nail head tensions. The magnitude of the nail head tensile load depends on the timeliness of the nail installation, the ground stiffness characteristics, the nail tensile stiffness, the nail grout-ground interface stiffness, and the facing stiffness.

Although one of the most attractive features of the soil nailing technique is the ability of the method to provide support with a relatively modest structural facing, this aspect of the system has been one of the least well understood. **It is clear, however, that the construction method and the presence of fully bonded reinforcements carried up to the facing result in lower face pressures than are developed with active support systems (e.g., tiebacks) or with conventional retaining structures.**

The issues for facing design and for assessing the contribution of the facing to the overall soil nail system "support" are therefore a) the magnitude of the facing load developed under the service load condition and how this load is applied (i.e., the pressure distribution at the ground-facing interface) and b) the strengths of the construction and final facings typically applied in soil nail wall construction.

2.4.4 Distribution of Nail Forces

Figure 2.4 showed a typical distribution of nail forces for a soil nail retaining wall with a horizontal backslope. Along any nail, the maximum nail force usually occurs at an intermediate point, whose location depends, for any given geometry, on the location of the nail within the wall. For a near-vertical wall with a horizontal backslope, the line of maximum tension within the reinforced zone is typically curvilinear and intercepts the surface at about 0.3H to 0.35H back from the wall (figure 2.4), where H is the height of the wall. Considering that nail lengths are typically on the order of 0.6H to 0.8H, this implies that in the upper part of the reinforced zone, the maximum nail force tends to occur at about the mid-length of the nail or slightly on the facing side of the mid-point. In the lower portions of the reinforced zone, the point of maximum tension moves closer to the wall face, as the zone of soil deformation is constrained by the close proximity of the foundation materials. The nail tension at the face is generally less than the maximum nail tension, with the ratio of nail head load to maximum nail load tending to decrease as the nail loads are progressively developed during construction.

The nail tensions are developed gradually as the excavation proceeds following nail installation. Observations during construction have indicated that most of the nail tensile loadings are developed within any nail during the first three excavation steps immediately following nail installation [7]. Consequently, the nails in the lower portion of the wall are usually more lightly loaded at the completion of construction because there has been insufficient soil deformation/straining in the vicinity of these lower nails to fully develop the nail-ground interface shear forces that mobilize the nail tensions. The bottom row of nails is completely unloaded immediately following construction, as it is installed following the last excavation step and may develop tensions only because of long-term deformations that might occur within the ground.

A summary of maximum measured nail loads (t_N), as a function of the nail depth within the wall, is shown plotted on figure 2.6. The nail loads were estimated from the results of strain gauge monitoring of test sections at a variety of sites described in table 2.2. The nail load estimates were made using a technique that attempted to account for the portion of the nail load supported by tensile stresses within the nail grout annulus [9], and considered the effects of grout aging, creep, and cracking. Much previous reporting of calculated nail loads ignores the contribution of the nail load carried by the grout, tending to underestimate the nail loads unless the grout in the vicinity of the strain gauge is extensively cracked.

The information presented in figure 2.6 is given in dimensionless format, in which the nail load is normalized with respect to the soil unit weight γ, the nail vertical and horizontal spacings, the wall height H, and the calculated active earth pressure coefficient K_a. The active earth pressure coefficient was calculated for the geometry and boundary loading conditions appropriate to the problem and for the soil friction angle reported by the wall designers (cohesive components of soil strength were such that $c/\gamma H<0.05$, and were ignored in the calculation of the K_a value). The interpreted information suggests a pattern for the maximum nail loads as a function of the depth within the reinforced zone. Within the upper two-thirds to three-quarters of the wall height, the maximum nail loads appear to be relatively constant with depth, with the normalized nail load ($t_N/(K_a\gamma HS_HS_V)$) within the range of 0.4 to in excess of 1.0. This rather large range in the normalized maximum nail loads probably reflects natural variability, the inaccuracies inherent in estimating nail grout loads using generic material property data and highly simplified material models, together with possibly inappropriate soil strength parameter estimates used and reported by the designers. The mean normalized maximum nail load in the upper portion of the wall appears to lie in the range of 0.75.

Within the lower one-quarter to one-third of the wall height, the maximum nail loads decrease significantly and appear in general to trend from the upper wall maximum values noted above to a value of zero at the base of the wall.

These observations are consistent with those reported by others. The "Recommendations Clouterre" [7] report states that the equivalent earth pressure distribution corresponding to the maximum measured nail loads corresponds to an at-rest K_o condition near the top of the wall to less than K_a conditions near the base of the wall.

2.4.5 Face Loading Magnitude and Distribution

In general, our understanding of the magnitude of the face loadings developed in soil nailing applications is not as good as is our knowledge of the maximum loads developed within the nails. The main reason for this is that the quality of field monitoring data is poor. The results from strain-gauged nails are more difficult to assess in the vicinity of the facing, where bending effects tend to be more significant. The most reliable information on nail head loads is likely to be obtained from load cells installed at the head of the nails, but there has been relatively little of this type of information collected to date.

Figure 2.7 shows the nail loads extrapolated to the facing, for the instrumented walls of table 2.2. The data are presented in the same dimensionless format as are the maximum nail loads given on figure 2.6. The nail head loads, or equivalent face pressures, are seen to follow a similar pattern to that observed for the maximum nail loads, i.e., fairly uniform in the upper two-thirds of the wall and decreasing in the lower part of the wall. The normalized nail head loads in the upper part of the wall vary in general from about 0.3 (excluding those walls with no facing, where the nail head loads were zero) to about 0.7 (excluding one or two high readings for the IH30 wall that appeared to be atypical in a number of respects, including the CH soils and the very close nail spacing). The mean normalized nail head load in the upper part of the wall is in the range 0.4 to 0.45, or less than 60 percent of the maximum nail loads observed under the service condition.

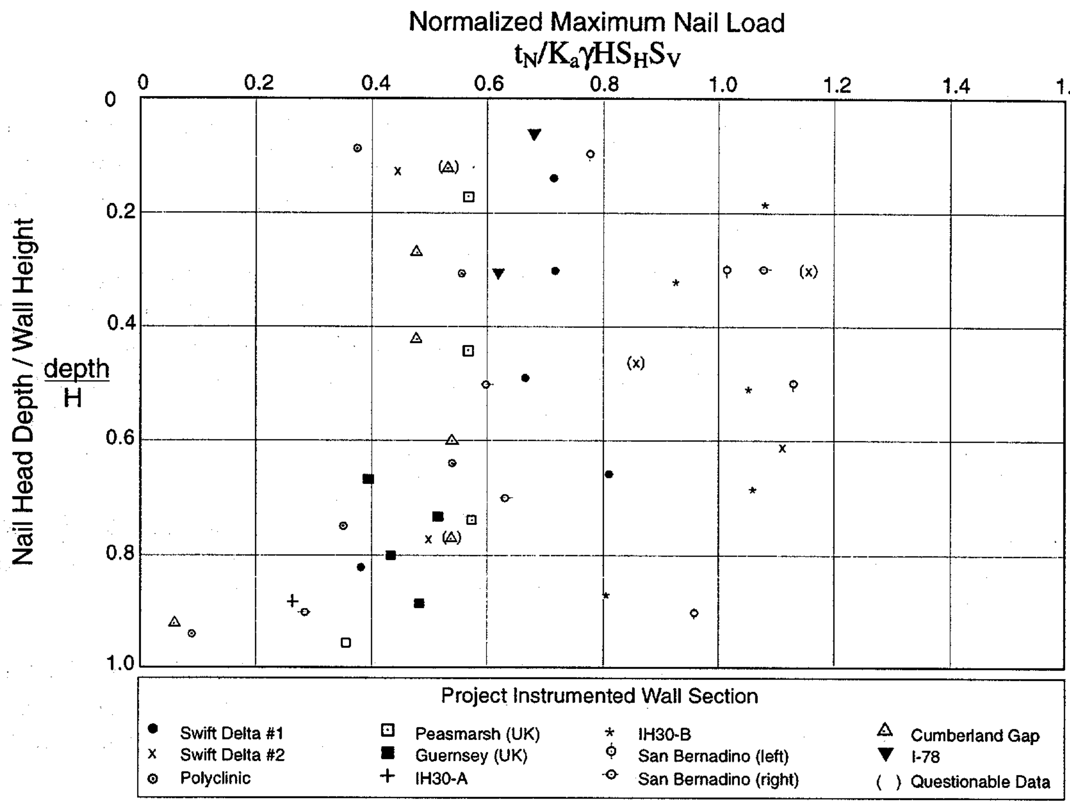


Figure 2.6 Normalized Maximum Measured Nail Loads

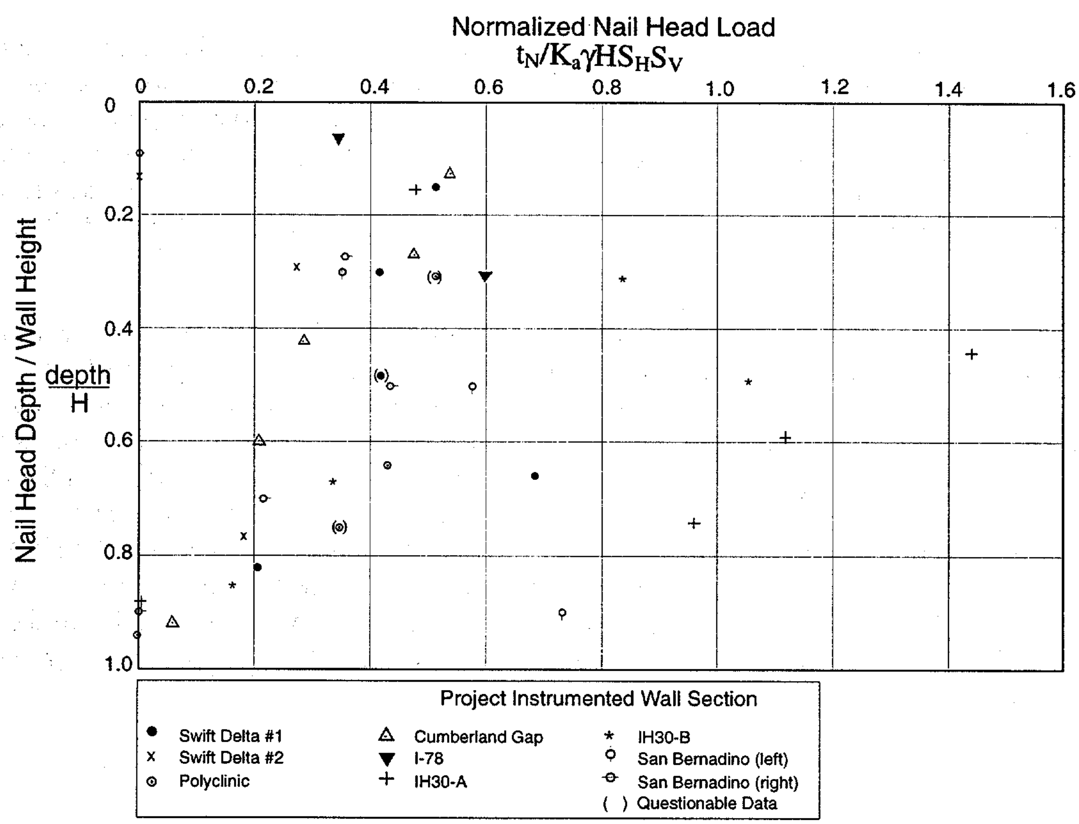


Figure 2.7 Normalized Measured Nail Head Loads

TABLE 2.2
CHARACTERISTICS OF MONITORED SOIL NAIL WALLS

	Swift-Delta Station 1	Swift-Delta Station 2	Polyclinic	Peasmarsh	Guernsey
Height (m)	5.3	5.6	16.8	11	20
Face slope (deg)	0	0	0	20	30
Back slope (deg)	55 kN/m^2 surcharge	27	0	0	0
Type of Facing	shotcrete	shotcrete	shotcrete	geogrid	geogrid
Nail Length (m)	6.4	5.2	10.7	6 – 7	10
Nail Inclination (deg)	15	15	15	20	20
Steel Diameter (mm)	29	29	36	25	25
Spacing, H x V (m)	1.4 x 1	1.4 x 1	1.8 x 1.8	1.5 x 1.5	1.5 x 1.25

	IH-30, Rockwall Section A	IH-30, Rockwall Section B	San Bernadino	Cumberland Gap 1988	I-78, Allentown
Height (m)	5.2	4.3	7.6	7.9	12.2
Face slope (deg)	0	0	6	0	3 m bench
Back slop (deg)	0	75 kN/m^2 surcharge	5	33	33
Type of Facing	shotcrete	shotcrete	shotcrete	shotcrete	concrete panels
Nail Length (m)	6.1	6.1	6.7	13.4	6.1-9.2
Nail Inclination (deg)	5	5	12	15	10
Nail Diameter (mm)	152	152	203	114	89
Steel Diameter (mm)	19	19	25	29	25-32
Spacing, H x V (m)	.75 x .75	.75 x .75	1.5 x 1.5	1.5 x 1.2	1.5 x 1.5

Gässler et al [10] reports the results of earth pressure measurements by means of Glötzl cells located at the shotcrete-soil interface for an experimental wall studied as part of the Bodenvernagelung program. Under self weight loading, the equivalent face pressure was reported to be about 50 percent of the Coulomb active value, with a distribution closer to uniform than triangular and a marked reduction close to the toe. When surface surcharge was added, the resultant earth pressure reached about 70 percent of the Coulomb active value. Furthermore, equivalent face pressure loads of 60 to 70 percent of the Coulomb active earth pressure were observed repeatedly, and it was recommended that the shotcrete facing and the nail heads should be dimensioned for these reduced earth pressures.

Similar behavior was observed during the investigations carried out as part of Clouterre. Under service conditions, the ratio of the nail head load to the maximum nail load was always less than one and was typically in the range of 0.4 to 0.5 for the most heavily loaded nails in the upper portion of the walls where the nail-soil interaction behavior was fully developed [7]. On the basis of this information, the Clouterre design recommendations are for facing service loads of 60 percent of the maximum nail service loads for a nail head spacing of one meter, increasing to 100 percent of the maximum nail service loads for a nail head spacing of three meters. For a typical nail head spacing of 1.5 m, this corresponds to a recommended facing service load of about 0.7 times the maximum nail service load.

Each of the above studies indicates that the nail head loads mobilized under service loading for typical nail spacings and relatively flexible construction facings of the mesh reinforced shotcrete type are significantly less than the maximum nail loads developed within the reinforced soil mass away from the facing. The ratio of the nail head load to the maximum nail load is defined as the nail head service load factor (F_F). A value of F_F equal to 0.5 should be adopted for design purposes unless the designer has site specific information from locally monitored nail walls. In absence of other data a nail head service load t_F, of $0.5K_a\gamma HS_HS_V$ is recommended.

2.4.6 Deformation Behavior

During construction of a soil nail wall from the top-down, the reinforced soil zone tends to rotate outwards about the toe of the wall as part of the process of mobilizing tensile loads within the nails. Hence, maximum horizontal movements occur at the top of the wall and decrease progressively towards the toe of the wall. Settlements at the facing also occur, and these tend to be on the same order of magnitude as the horizontal movements at the top of the wall. Displacements of the facing depend on the following factors [7]:

- Construction rate;
- Nail spacing and excavation lift heights;
- Nail and Soil stiffness;
- Global factor of safety;

- Nail inclination (greater displacements for greater inclinations because of less efficient reinforcing action);
- Bearing capacity of the foundation soils; and
- Magnitude of any surcharge loadings.

For vertical soil nail walls with typical nail-length-to-wall-height ratios, negligible surcharge loadings, and designed with reasonable factors of safety, the peak wall displacements at the top of the wall tend to vary from 0.1%H or less for weathered rocks and very competent and dense soils (e.g., glacial tills), to 0.2%H for granular soils, and up to 0.4%H for fine-grained clay type soils. These movements are considered to be relatively small and generally correspond to those expected for well braced systems and for tieback walls.

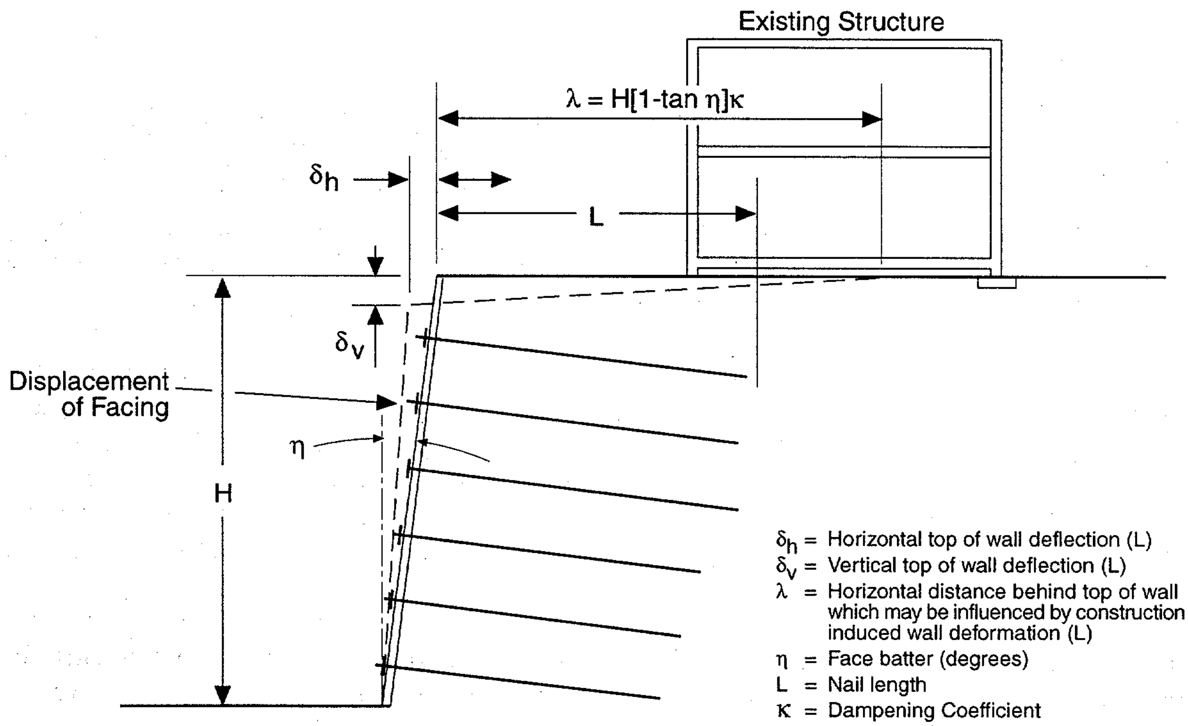
The construction displacements tend to decrease in magnitude with distance back from the wall facing and are typically on the order of several tenths of the wall facing displacements at a surface location over the ends of the nails. Figure 2.8 presents the Clouterre recommendations [7] for estimating the manner in which surface displacements decrease with distance from the facing for soil nail walls in various soil types. The displacements experienced by soil nail walls are generally comparable to those experienced by other types of retaining walls (see figure 2.9).

Post-construction monitoring of wall displacements indicates that some ongoing movements may tend to occur with time, depending on the ground type, and that some additional tensions in the nails, particularly those near the base of the wall, may develop. In many instances, however, ongoing straining of the nails is associated principally with a redistribution of nail load from the creeping grout annulus to the steel tendon, as inclinometer or wall survey data may indicate zero to minimal time dependent deformations of the wall.

2.4.7 Role of Nail Bending and Shear

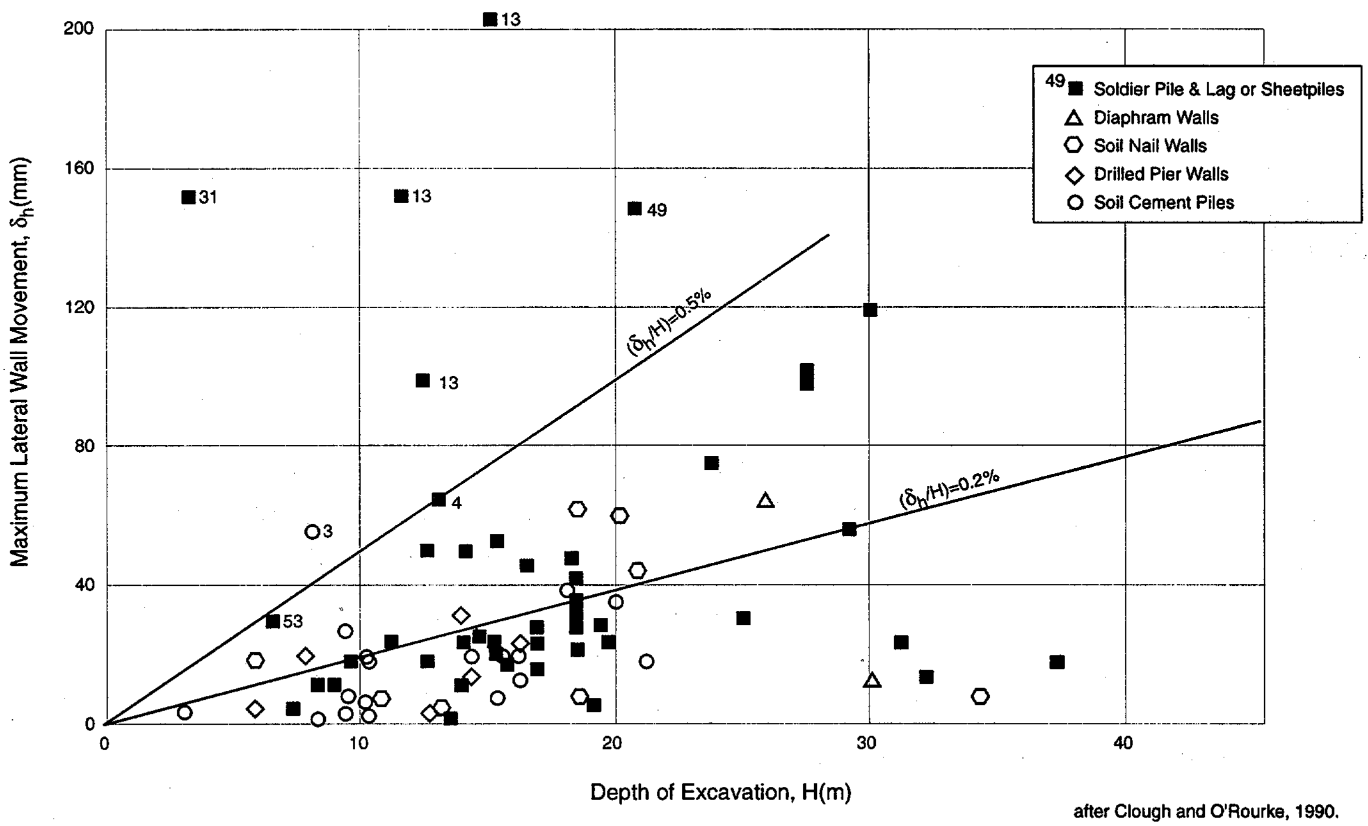
Under service load conditions, soil nails provide their reinforcing action almost exclusively through the development of tensile nail loadings associated with shear stresses developed at the interface between the nail and the ground. For the relatively slender dimensions typical of soil nails and the nature of the materials usually reinforced by the emplacement of nails, the reinforcing contribution of the nails by the development of shear forces within the nails is limited by their small flexural or bending strength. In addition, the minimum displacement required to mobilize the full passive bearing pressure beneath a laterally loaded nail is about one order of magnitude greater than the minimum displacement required to mobilize the nail grout-ground ultimate adhesion. Shear and bending of the nails are not therefore considered to be important for design.

For typical soil nail construction facings consisting of approximately 100 mm of shotcrete, the soil nails will generally be able to support the self weight of the facing through a combination of interface friction between the facing and the soil and bearing of the nails against the soil



Type of Soil	Weathered Rocks Stiff Soils	Sandy Soils	Clayey Soils
δ_h and δ_v coefficient κ	H / 1000 0.8	2 H / 1000 1.25	3 H / 1000 1.5

Figure 2.8 Deformation of Soil Nail Walls (Clouterre 1991)



after Clough and O'Rourke, 1990.

Figure 2.9 Observed Maximum Lateral Movements for Insitu Walls in Stiff Clays, Residual Soils and Sands [11]

surrounding the individual nails. However, for relatively thick shotcrete facings (such as are typically employed in permanent top-down construction) with less competent ground, it may not be possible to support the weight of the facing without unacceptable downward and outward movement of the wall. In such cases, additional vertical support may be obtained by installing "strut" nails or "dowels." Strut nails are short, highly inclined nails, typically placed between the primary nail locations and act primarily in compression. Dowels are short supplementary nails that provide additional vertical support to the facing by acting in shear/bending. Determination of an adequate pattern of struts or dowels is discussed in section 4.10.16.

2.5 Comparison with MSE and Tieback Walls

As a means of reinforcing/supporting in-situ ground to enable the development of steep slopes or walls within that ground, soil nail walls are often in direct competition with the more widely known tieback walls. In order to better define those applications in which soil nail walls offer a viable alternative to tiebacks, it is necessary to understand the fundamental differences between the two systems.

Although soil nailing is not a direct equivalent of MSE in that the former reinforces in-situ ground and the latter is associated with the construction of reinforced fills, the two techniques share the common feature of being passive reinforcement methods. However, there are also a number of fundamental differences between soil nailing and MSE, and these must be understood to appreciate why somewhat different design approaches have been adopted for the two techniques.

Table 2.3 provides a summary comparison of soil nail, tieback and MSE wall types, discussed further below.

2.5.1 Tieback Walls

There are a number of significant functional distinctions to be made between tiebacks and soil nails. The following comparisons (adapted from Bruce and Jewell [12], and the Clouterre program [7]) may be drawn:

- Tiebacks do not interact directly with the soil (except in the anchor zone) and the supporting force is constant for all potential slip surfaces contained within the "no load zone." The tensile reinforcement load varies along the length of a soil nail, however, and the total supporting force will therefore change from one potential slip surface to another (figure 2.10).
- Tieback anchors are stressed after installation. In contrast, soil nails are not prestressed, except for a small seating load, and require a small but finite relative movement between the nail and the ground in order to develop their supporting tensile loads.
- Because of the installation of soldier piles prior to excavation, face stability and stand-up time are much less critical concerns for tieback systems than for soil nail systems.

Nails are installed at a far higher density than tiebacks, and the consequences of a unit failure are therefore much less severe because of the higher redundancy of nail installations. However, the load carrying capacity of each tieback is verified by testing whereas typically only about 5 percent of nails are load tested.

TABLE 2.3
CONCEPTUAL COMPARISON OF WALL TYPES

Wall Type	Soil Nail Wall	Tieback Wall	MSE Wall
Wall Construction	Top down (cut in-situ soil/rock)	Top down (cut in-situ soil/rock)	Bottom up (in fill)
	Reinforcements are drilled and grouted in place full length in both active and resistant zone	Reinforcements are drilled and grouted in place, but such that all reinforcement capacity is obtained behind active zone	Reinforcement strips or sheets are sandwiched between compacted fill in both active and resistance zone
	Reinforcements are passively stressed as cut excavation progresses	Reinforcements are actively stressed at each excavation stage	Reinforcements are passively stressed as fill construction progresses
Wall Behavior	Reinforcement loads decrease top to bottom (see figure 2.12)	Near uniform stress distribution (see figure 2.12)	Reinforcement loads increase top to bottom (see figure 2.12)
	Facing carries only soil pressure not taken up by nails	Facing carries full soil pressure	Facing carries only soil pressure not taken up by reinforcement layers
	Load transfer between soil and reinforcement occurs along full length (see figure 2.10)	Load transfer between soil and reinforcement occurs only in resistant zone (no load transfer allowed in active zone - see figure 2.10)	Load transfer between soil and reinforcement occurs along full length (see figure 2.10)
	Tensile forces in nail due to gravity forces on soil in active zone	Tensile forces in anchor result from equilibrium between tensile force applied to anchor head during stressing and self weight of soil in active zone	Tensile forces in reinforcement due to gravity forces on soil in active zone
	Soil is partially confined by nails	Soil is confined by facing only	Soil is mostly confined by reinforcement, depending on reinforcement type and spacing
	Maximum wall face deflection is at top of wall (see figure 2.11)	Maximum wall face deflection is generally at midlevel, depending on how anchors are stressed	Maximum wall face deflection is usually at lower third point (see figure 2.11)

Wall Type	Soil Nail Wall	Tieback Wall	MSE Wall
Design Approach (Internal)	Limit equilibrium using allowable capacity of facing and nail (strength and pullout resistance) as added force in combination with available soil strength to achieve force and/or moment equilibrium along critical slip surface	Uses empirical earth pressure distribution to calculate reinforcement stress, which is equated to reinforcement capacity to determine safety factor against reinforcement rupture and pullout	Uses empirical earth pressure distribution (different than that used for tiebacks) to calculate reinforcement stress, which is equated to reinforcement capacity to determine safety factor against reinforcement rupture and pullout
	Ratio of resisting forces/moments to driving forces/moments is FS		
	Reinforcement strength and pullout resistance required is calculated globally for entire wall or for intermediate construction stages (limit state conditions)	Reinforcement strength and pullout resistance required is calculated at each reinforcement level based on the anchor capacity required for the facing to structurally resist the lateral earth pressure applied (working stress conditions)	Reinforcement strength and pullout resistance required is calculated at each reinforcement level to resist the local lateral earth pressure in the tributary area of the reinforcement (working stress conditions)
	Facing and facing/nail connection system checked for possible failure modes (e.g., flexure, shear) using empirical earth pressure distribution at the facing	Facing (e.g. soldier piles) and facing-tieback connection system checked for possible failure modes under the installed tieback pre-stress	Facing-reinforcement connection system checked for possible failure modes using empirical earth pressure distribution at the facing.
Design Approach (External)	Treat reinforced zone as rigid body, evaluating overturning and bearing capacity. Slope stability behind wall is also evaluated	Only slope stability behind wall is typically evaluated, but bearing failure of the gravity wall should be evaluated under adverse foundation conditions	Treat reinforced zone as rigid body, evaluating sliding, overturning, and bearing capacity. Slope stability behind wall is also evaluated

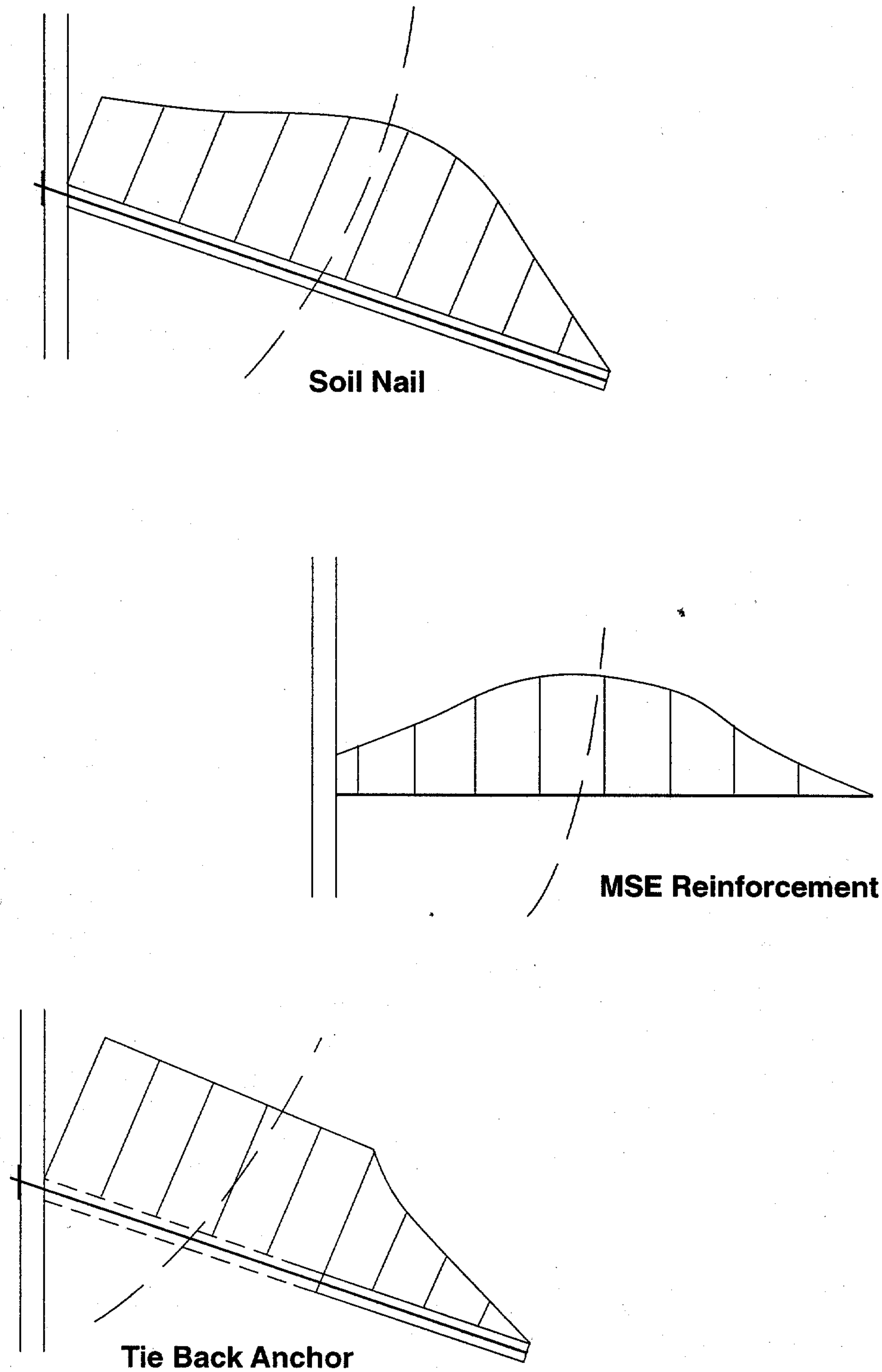


Figure 2.10 Typical Tensile Force Distribution Along Reinforcement Length

- As the maximum reinforcing load is taken to the face with tiebacks, appropriate bearing support must be provided at the face to eliminate the possibility of "punching through" the facing of the retained structure. Because only a portion of the maximum nail load is taken to the facing with soil nailing, and the individual reinforcing loads are smaller because of the closer spacing of nails versus tiebacks, substantial bearing support at the face is not required with nails. Typically, nail face loads are satisfactorily accommodated by small steel plates bearing directly on the shotcrete construction facing.
- Individual tieback anchors tend to be longer than individual nails, and may therefore require larger installation equipment. In addition, a tieback anchorage system is often used to retain a substantial structure such as a diaphragm wall or bored pile wall, which will itself require large scale construction equipment.

In general, if stability problems are shown to be deep seated with marginal soils adjacent the wall facing and more competent materials at depth, then ground anchorages will probably be required. Conversely, in the appropriate soil types, soil nailing has frequently proved preferable to other methods of lateral support that incorporate prestressed ground anchors (e.g., soldier pile wall).

2.5.2 MSE Walls

MSE walls and soil nail walls share certain features in common. These are summarized as:

- The reinforcement is placed in the soil unstressed and the reinforcement forces are mobilized by subsequent deformation of the soil.
- The reinforcement forces are sustained by frictional bond between the soil and the reinforcing element. The reinforced zone performs as a gravity retaining structure and resists the thrust from the unreinforced soil it retains.
- The facing of the structure is thin–prefabricated elements in the case of mechanically stabilized earth and usually shotcrete in the case of soil nails.

There are, however, some fundamental differences that can be summarized as:

- *The construction procedure:* Although the two structures may appear similar at the end of construction, the construction sequences are radically different. Soil nailing is constructed by staged excavations from the "top down" whereas MSE walls are constructed by placing reinforced fill from the "bottom up." This different construction sequence results in a different wall deformation pattern (figure 2.11) and a different distribution of loads within the reinforcement (figure 2.12). Maximum deformations in a soil nail wall occur at the top of the wall, whereas in an MSE wall the maximum deformations occur near the base (figure 2.11). Also, with mechanically stabilized earth, the reinforcing loads generally increase from the top of the wall towards the base, whereas with soil nailing the reinforcement loads tend to be more uniform with depth, similar to a braced excavation, and decrease in the lower portion of the wall (figure 2.12).

- *The nature of the soil:* Soil nailing is an in-situ reinforcement technique and the properties of the soils cannot be pre-selected as they are for MSE walls. MSE walls utilize clean, low water content granular backfills that are predominantly frictional in nature. On the other hand, nails are installed into soil and rock whose strength properties and water content can vary through a wide range.
- *The soil-reinforcement bond:* Grouting techniques are usually (although not exclusively) employed to bond the nail to the surrounding ground, and load is transferred along the grout-soil interface. In MSE structures, friction is generated directly along the reinforcement to soil interface.

From a design perspective, the locations of the maximum tensile forces for MSE walls are generally well known because of the simplicity of the geometry and the standardization of the materials. This is not the case for soil nail structures, and simplified empirical "earth pressure" or "local equilibrium" design models are therefore more difficult to apply with soil nailing than with mechanically stabilized earth.

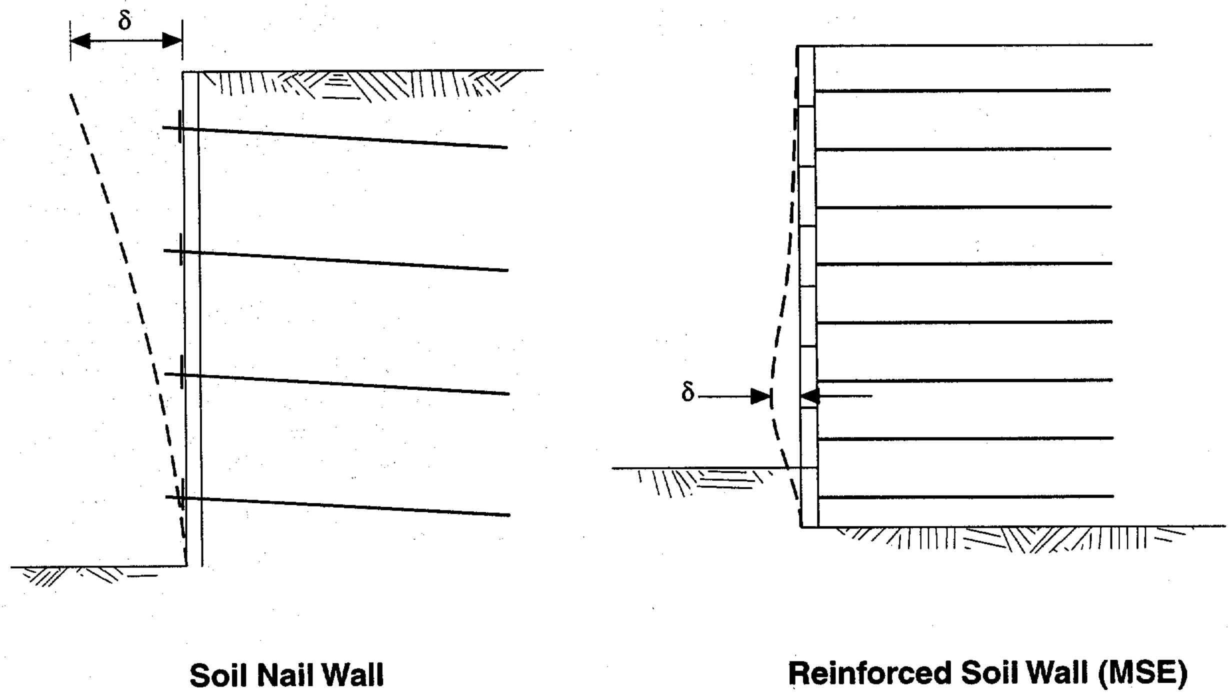


Figure 2.11 Differences in Wall Deformation Between Soil Nail Walls and Reinforced Soil Walls (Clouterre 1991)

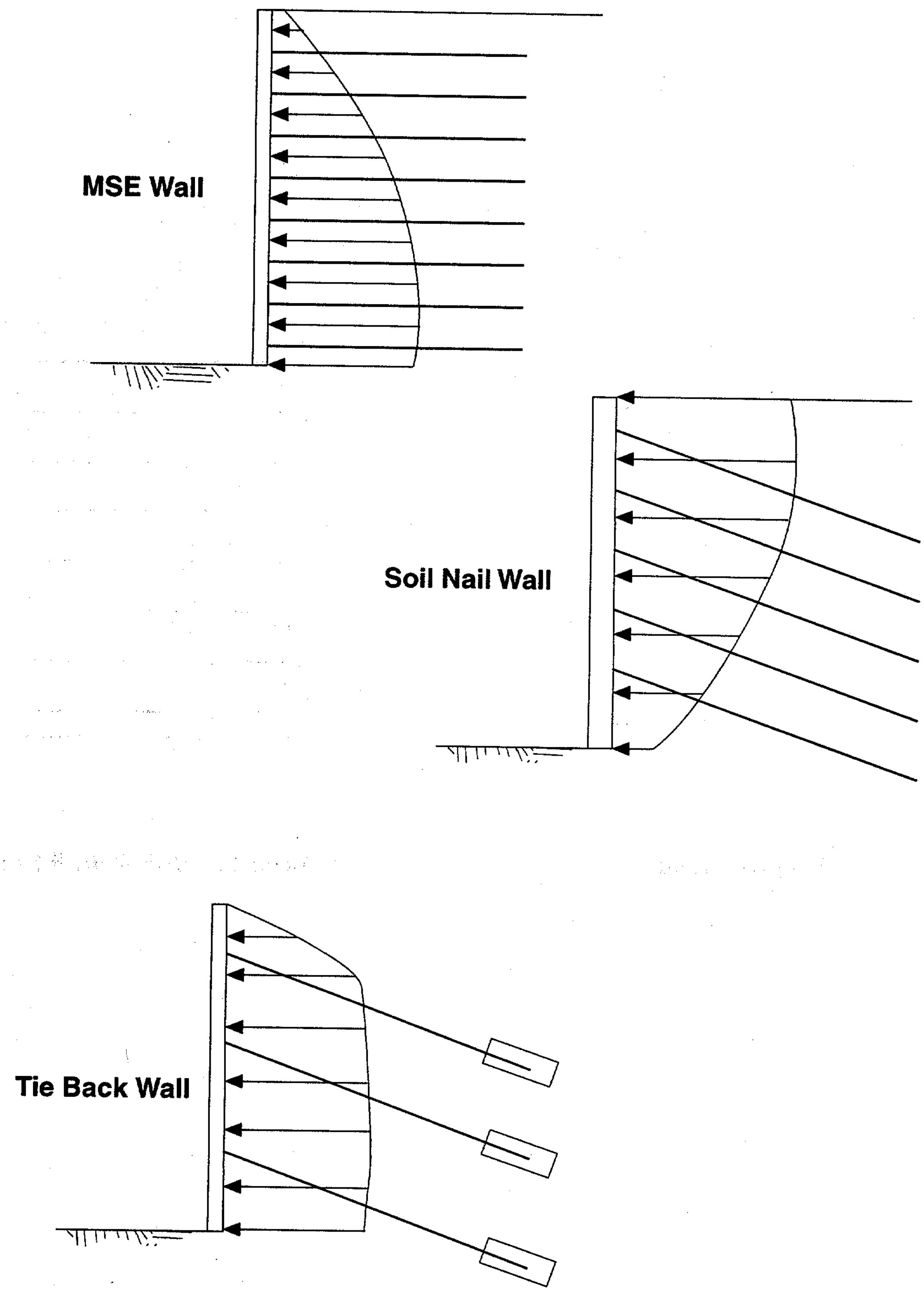


Figure 2.12 Typical Stress Distribution Vertically in Reinforcements for Soil Nail, Tie Back, and MSE Walls

CHAPTER 3. SITE INVESTIGATION AND TESTING

3.1 Ground Characterization

The feasibility of an economical and reliable design for soil nailing depends on existing site physical features, subsurface stratigraphy, groundwater, and soil/rock properties. Subsurface investigations must explore not only the location of the nail wall face, but also the region of the nails themselves. These investigations should determine the nature, strength, and corrosive potential of the ground in which the nails are to be placed. Each project must be treated individually as both soil conditions and related risks may vary widely. The basic ingredients for a rational subsurface investigation program include a background study of geologic conditions, field reconnaissance, explorations, and laboratory testing. The information obtained from one phase is used to determine the scope of work in the next. The aim of the investigation is to determine, by the most economical means, adequate information about the block of ground in which nails will be installed to permit safe, economical design and construction. This includes information on groundwater conditions and an assessment of excavation face stability.

The primary design considerations for soil nail walls are adequate stability, durability, and limited wall deflections. The most critical component in the design and construction of a soil nail wall is an adequate design phase site investigation. Further, design of soil nail walls, like any major retaining wall system, should be performed only by well-qualified and experienced geotechnical and structural engineers.

The recommended site investigation phases for a soil nail wall are discussed below.

A. Site Geology Review

A review of available geologic and groundwater information should be performed initially. This information may be contained in sources such as geologic maps, air photos, surveys, and other databases including geologic/geotechnical reports prepared in association with prior site investigations in the project area.

B. Field Reconnaissance

The minimum elements of a complete field reconnaissance for permanent ground anchor installations presented in publication FHWA DP-68-1R [13] are consistent and applicable for soil nail structures, since soil nailing will often be considered as an alternate to ground anchors for cut retention.

Early in project planning, sites that appear suitable for soil nail structures should be inspected in the field by the geotechnical engineer, preferably accompanied by the project roadway design and structural engineers. On rehabilitation or reconstruction projects, a cooperative geotechnical - structural inspection should be mandatory. Such a "team" approach has been proven to result in better design decisions and improved constructability. Major items to be accomplished during field reconnaissance include:

1. Selecting limits and intervals for topographic cross sections.

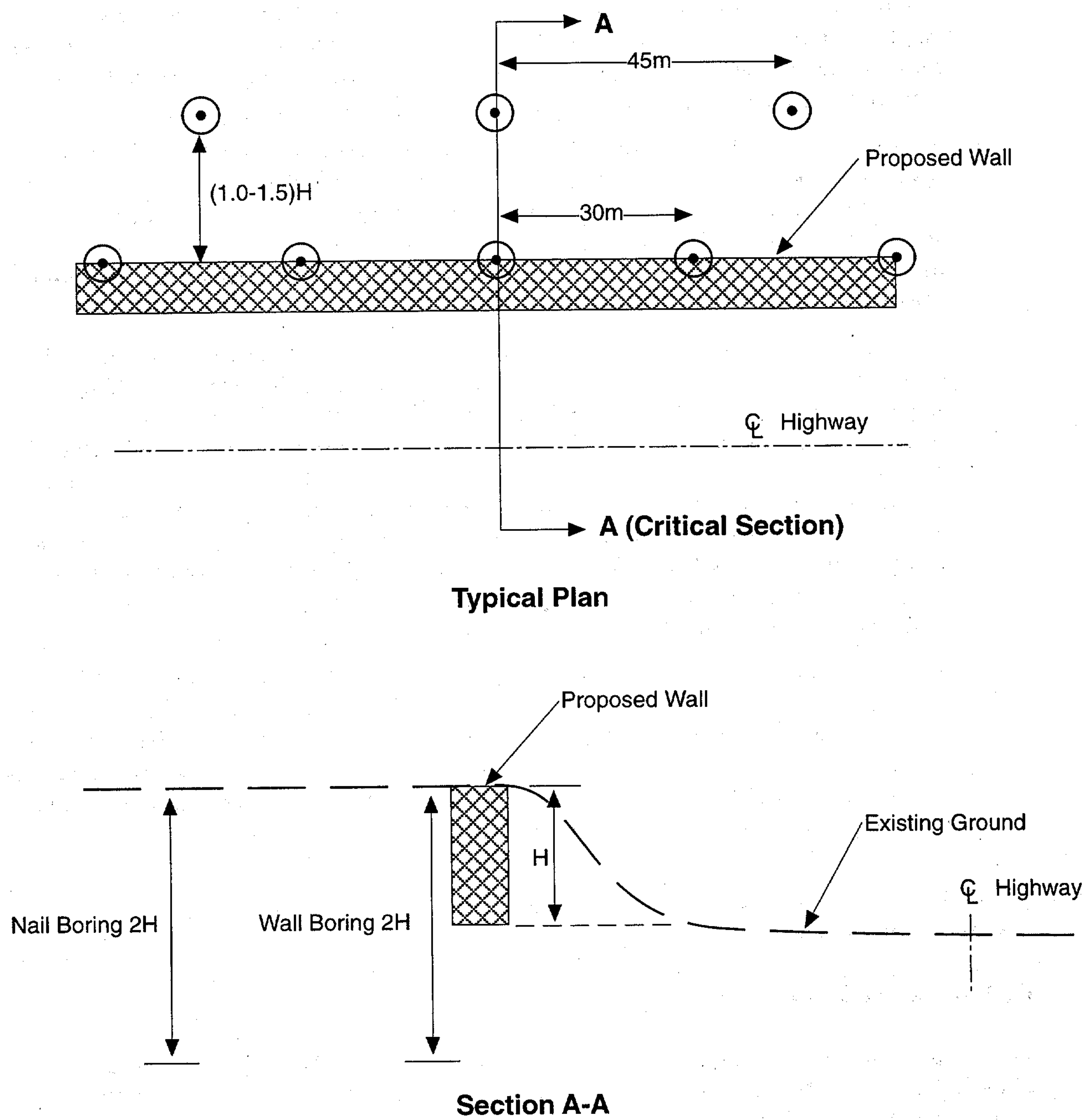
2. Recording site access conditions for work forces and equipment (both for explorations and construction).
3. Observing surface drainage patterns, seepage and vegetative characteristics to estimate groundwater conditions and structural drainage requirements. Corrosion of existing drainage structures should be noted to identify if a corrosive environment may exist for concrete and/or steel materials.
4. Studying surface geologic features including rock outcrops and landforms. Existing cuts or excavations can be used to help identify stratification.
5. Determining the extent, nature, and situation of any existing or abandoned, above- or below-ground utilities (electric, water, sewer, gas, etc.), basements, and substructures of adjacent structures which may be impacted by explorations or subsequent construction.
6. Identifying existing structures or properties not under common ownership with the new project for possible future right-of-way acquisition or legal temporary or permanent underground easement.
7. Reviewing files for existing subsurface data in the vicinity of the project site.

C. Subsurface Exploration

Subsurface explorations should be sufficiently detailed to determine soil/rock stratigraphy in the zones affected by the proposed soil nail wall construction, develop subsurface cross-sections adequate for stability analyses, allow an estimate of the pullout capacity of the nails, and develop sufficient information to design an efficient internal drainage system. **Whenever site access allows, backhoe test pits or small test cuts are recommended as part of the exploration program to help assess whether or not the excavation face will stand while temporarily unsupported during the staged lift excavations. Such test pits or cuts provide a quick, inexpensive, and very valuable exploration method for soil nail walls, particularly for assessing constructibility.**

The number, type, and locations of subsurface explorations are usually determined by the geotechnical engineer or engineering geologist and are based on the results of the field reconnaissance and available existing subsurface data. There is no "cookbook" exploration program that will fit all sites. Engineering judgment must be used on a project-by-project basis to determine the final subsurface program. The following are recommended general guidelines.

1. Wall borings spaced at approximately 30 m intervals along the structure alignment (figure 3.1). Nail retaining structures (similar to tieback walls) require an additional line of borings (nail borings) to provide subsurface information for design of the nail zone. In flat or gently sloping ground, the nail borings are recommended to lie on approximately 45 m centers at a distance behind the wall equal to approximately 1.0 to 1.5 times the wall height. For sloping ground conditions, the distance behind the wall



Notes:
(1) Nail borings located (1.5-2.0)H behind facing for sloping ground conditions.
(2) Test cut or test pit recommended, when feasible, to evaluate excavation "Stand-up" time.

Figure 3.1 Site Exploration Guideline for Soil Nail Walls

of the nail borings may be increased up to approximately 1.5 to 2.0 times the wall height, depending on the backslope. Static cone penetrometer tests may be substituted for up to half of the borings in a line. At critical sections, borings may be added in front of the proposed wall line to better define the soil/rock stratigraphy.

2. Boring depths will be a function of the encountered subsurface conditions, but where bedrock is within a reasonable depth, extract a minimum rock core length of 3 m. The core is used to distinguish between boulders and bedrock and to identify the rock type. Wall borings and nail borings are usually extended to a minimum depth equal to at least the proposed wall height below the wall base, or 3 m into rock if rock is encountered at a lesser depth.

3. Standard penetration tests (SPT) should be performed at 1.5 m intervals and the soil samples sent to the soils laboratory for visual identification, classification, and testing. In ground that may contain thin weak soil layers, continuous SPT sampling is recommended. Undisturbed tube samples or in-situ strength testing should be taken in cohesive soil deposits at 1.5 m to 3 m depth intervals in sufficient borings to determine the characteristics and variations of the soil deposit. Careful static water level determinations must be made on completion of the boring. A notation should be made on removal of tools and/or casing as to whether the hole stayed open or of the depth of collapse. At least one nail and one wall boring should be converted to a water observation well for long-term water level readings.

4. Test cuts or pits are recommended to be approximately 6 to 8 m long and 2 to 2.5 m deep and left open for at least 3 to 4 days. A daily inspection is recommended, with "stand-up" conditions documented and a photographic record prepared. The long axis of the cut or pit should be parallel to, and located in front of, the proposed wall face. In residual soils, a joint survey should be made to determine the major joint systems and their orientations and joint surface characteristics.

Exploratory work should be carried out in accordance with Agency standards and/or the AASHTO Subsurface Investigation Manual.

D. Laboratory Testing

The focus of the testing program is to obtain reliable estimates of the unit weight and strength of the soil or rock. Properties such as Atterberg limits and grain size distribution will help in establishing general suitability of soil nailing as a structural support system. Soil tests to determine the corrosion potential of the soils should also be conducted.

Minimum test requirements for soil and rock sites are outlined below. However, the geotechnical engineer must decide the type and amount of testing to be done based on the importance of the structure, the site conditions, and local experience.

1. Soil: All soil samples extracted from borings should be visually identified and described in both the field and the laboratory. Moisture contents should be taken on representative samples. In addition, for cohesionless soils, the gradation of the soil should be determined, including the fraction and characteristics of the fines. Unit

weights of granular materials can be estimated from correlation to SPT values or be established in the field by conventional sand cone (ASTM D-1556) [14] or rubber balloon (ASTM 2167) [15] methods. Granular soil properties are often determined from correlation's with standard penetration test or cone penetration test results. Undisturbed samples of granular soils are difficult to obtain and test in the lab. Direct shear tests can be used to investigate the variation in shear strength of granular soils with increasing strains. Triaxial tests are not usually performed in production testing on granular soils.

For cohesive soils, the total unit weight may be determined from undisturbed thin-walled tube samples. The soil strength properties should be determined for both drained (long-term) and undrained (short-term) loading conditions, irrespective of the planned life of the structure. Local experience should be used, where available, to assist in estimating the shear strength of cohesive soils. The effective stress (long-term) strength parameters can be determined from CU triaxial testing with pore pressure measurements, and the undrained strength values estimated from UU triaxial testing. In-situ vane shear tests can also be used for estimating the undrained shear strength of clayey soils. In residual soils, direct shear testing should be performed on critically oriented joint surfaces. Fine-grained soils should be evaluated for their susceptibility to frost (silts) or swelling (clays).

Creep Potential

As noted previously, the Atterberg limits can be used to identify clay soils that should be considered as either non-applications for soil nailing or as potentially problematical with respect to long-term creep. Nails should not be located in organic soils or cohesive soils with a Liquidity Index greater than 0.2 and an undrained shear strength less than 50 kN/m^2 without evaluating the long-term creep behavior of the soil nails by performing creep tests. The Liquidity Index (LI) is defined as:

$$LI = \frac{W - WP}{WL - WP}$$

WL = Liquid Limit Water Content
WP = Plastic Limit Water Content
W = Natural Water Content

Long-term static loading may cause creep of the nail bond zone with time. The nail must be designed such that this deflection will not produce objectionable movements of the structure or the adjacent property. Creep may theoretically develop in three components of the nail: creep of the ground surrounding the anchor bond zone due to time-dependent secondary compression of the soil structure, creep of the grout in the bond zone, and creep of the steel comprising the tendon and/or connections. From a practical view-point, the latter two factors have little significance at typical design loads while the former is only of importance in fine grained plastic soils or soft rocks. Still, the concept of creep is important to be understood by the designer as creep measurements will be recommended for routine inclusion on all projects in later sections of this manual.

In practice, a plot of cumulative nail displacement versus the log of time produces a linear relationship for individual loads. As the total load, which is applied and held constant, increases, so does the slope of the plot relative to the slope at lower loads. A total creep rate exceeding 2 mm per log cycle of time (6-60 minutes) on a short-term field test has been shown to be unacceptable in practice. Such excessive creep rates are usually associated with soft cohesive soils and tend to increase with time until failure occurs. Longer term creep tests may be necessary in soils exhibiting borderline creep rates to determine long-term behavior and permit prediction of service life deflection. Detailed guidance for longer term creep tests are given in AASHTO Task Force 27 Report, August 1990, Pages 185-188 [16].

Corrosion Potential

Soil tests may be performed to measure the aggressiveness of the soil environment, especially if field observations indicate corrosion of existing structures. The most common and simplest tests are for electrical resistivity, pH, chloride, and sulfate. In general if the electrical resistivity of the soil is greater than 5000 ohm-cm and pH between 5 and 10 the soil may be considered to be non-aggressive and additional corrosion testing is unnecessary. If the electrical resistivity is between 2000 and 5000 ohm-cm, sulfate and chloride tests are required. The designations for these tests and the critical values defining whether an aggressive soil environment exists, are as shown below. The ground is considered aggressive if any one of these indicators show critical values.

TABLE 3.1
GROUND AGGRESSIVENESS INDICATORS

Property	Test Designation	Critical Values
Resistivity	ASTM G 57 [17], AASHTO T-288 [18]	below 2,000 ohm-cm
pH	ASTM G 51 [19], AASHTO T-289 [20]	below 5
Sulfate	ASTM D516M [21], ASTM D4327 [22]	above 200 ppm
Chloride	ASTM D512 [23], ASTM D4327 [22], AASHTO T-291 [24]	above 100 ppm

Ref. FHWA-RD-89-198 [25]

Note: User should check test standard for latest updates and individual transportation agencies may have limits on critical values different than tabulated above.

2. Rock: Analysis of rock properties is more field oriented as the presence and location of fissures, joints, or other discontinuities will control the overall strength of the rock mass. Determination of rock properties (mass strength, bond) is based on information from both laboratory and field testing:

   a. Form of the rock mass and the depth of overburden
   b. Rock type
   c. Rock quality designation (RQD)
   d. Joint spacing and orientation
   e. Stratification
   f. Rock material strength
   g. Water pressure in joints

E. Final Feasibility Evaluation

Based on the results of the subsurface exploration and subsequent laboratory testing program, a final feasibility evaluation can be made to determine if a successful design can be implemented with a relatively high degree of confidence. **This requires an understanding of ground conditions for which nailing is and is not well suited**. These conditions are discussed in chapter 1. In addition, anticipated displacements for the ground conditions (see section 2.4) should be estimated and checked for compatibility with adjacent structures.

3.2 Estimating Nail Pullout Resistance for Design

Verification of the ultimate soil-nail pullout resistance, Q_u, assumed in design is essential to ensure structure safety. It should be considered an extension of design. Further, the actual pullout resistance achieved can be affected by:

- Soil or rock type and shear strength.
- Roughness of drillhole wall (will vary with drilling method used).
- Final drillhole diameter.
- Loose drill cuttings left along the bottom of the drillhole (can occur particularly with auger drilling or when air is used to remove drill cuttings if air compressor capacity is not large enough).
- Contractor drilling and grouting techniques, expertise, and workmanship.
- Amount of time hole left open before grouting.

Nail pullout resistance estimates should be based on experience with open hole methods of construction if soil conditions allow. If inadequate experience exists to provide a conservative design value, then a pre-contract test nail program should be considered to determine the appropriate design values, particularly on large projects.
It is imperative that field pullout testing be done during construction to verify the estimated pullout resistance used in design. Field pullout tests should be conducted as recommended in FHWA-SA-93-068, "Soil Nailing Field Inspectors Manual [26]." In contrast to ground anchorages, it is not normal to test each individual production soil nail.

Guidance for estimation of ground anchor and nail pullout resistance has been previously summarized in FHWA reports titled "Tiebacks," Report No. FHWA/RD-82/047 [27]; "Permanent Ground Anchors," FHWA-DP-68-1 R [13]; and "Soil Nailing for Stabilization of Highway Slopes and Excavations," FHWA-RD-89-198 [25]. Local experience and practice can also be used to estimate nail pullout resistances for use in design. Guideline unit ultimate grout-ground bond stress values, are given below.

A. Cohesionless (Granular) Soils

For tremie or low pressure grouted nails in dry cohesionless soils, data reported in the literature suggest the following ranges of ultimate pullout resistance.

TABLE 3.2
ULTIMATE BOND STRESS - COHESIONLESS SOILS

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Open Hole	Non-plastic silt	20 -30 (3.0-4.5)
	Medium dense sand and silty sand/sandy silt	50 - 75 (7.0-11.0)
	Dense silty sand and gravel	80 - 100 (11.5-14.5)
	Very dense silty sand and gravel	120 - 240 (17.5-34.5)
	Loess	25 – 75 (3.5 – 11.0)

B. Cohesive Soil

For tremie grouted nails, the ultimate pullout resistance can be estimated as 0.25 to 0.75 times the average undrained shear strength, with the lower factors associated with the stiffer or harder clays. For augered holes, a lower factor may be warranted because it is influenced by the care taken in cleaning the drillhole. For sandy and silty clays, the factor is somewhat higher than the range shown above. Typical values of ultimate pullout resistance for cohesive soils are indicated below.

TABLE 3.3
ULTIMATE BOND STRESS - COHESIVE SOILS

Construction Method	Soil Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Open Hole	Stiff Clay	40 - 60 (6.0-8.5)
	Stiff Clayey Silt	40 - 100 (6.0-14.5)
	Stiff Sandy Clay	100 - 200 (16.5-29.0)

C. Rock

The ultimate pullout resistance for tremie grouted nails in competent massive rock may be taken as 10 percent of the uniaxial compressive strength of the rock up to a maximum value of 4000 kN/m^2. Estimated ultimate pullout resistance for different rock types are given below.

TABLE 3.4
ULTIMATE BOND STRESS -ROCK

Construction Method	Rock Type	Unit Ultimate Bond Stress kN/m^2 (psi)
Rotary Drilled	Marl/Limestone	300 - 400 (43.5-58.0)
	Phillite	100 - 300 (14.5-43.5)
	Chalk	500 - 600 (72.0-86.5)
	Soft Dolomite	400 - 600 (58.0-86.5)
	Fissured Dolomite	600 - 1000 (86.5-144.5)
	Weathered Sandstone	200 - 300 (29.0-43.5)
	Weathered Shale	100 - 150 (14.5-21.5)
	Weathered Schist	100 - 175 (14.5-25.5)
	Basalt	500 - 600 (72.0-86.5)

Additional information on the ultimate pullout resistance of anchors installed in rock is given in "Rock Anchors: State of the Art" [28].

Note: In the design procedure and guide construction specifications set forth in this manual, the nail pullout resistance is expressed in terms of force per unit length of nail, kN/m. The way to compute pullout resistance in terms of force per unit length of nail using the unit bond stresses tabulated above is:

Ultimate Pullout Resistance, Qu (kN/m) = (Unit Ultimate Bond Stress (kN/m^2))(π)(D(m))

Where:

πD = Nail Drillhole Circumference
D = Nail Drillhole Diameter (m)

Example:

Assume nail drillhole dia. = 150 mm = 0.15 m.
Assume soil is dense to very dense silty sand.
From table 3.2 estimate unit Ultimate Bond Stress = 130 kN/m^2.
Ultimate Pullout Resistance, kN/m = 130 kN/m^2 x 3.14 x 0.15 m = 61.2 kN/m.
Use 60 kN/m.

The Clouterre experimental program [7] has summarized the results of nail pullout tests as a function of the material type and installation technique. The unit ultimate bond stress is presented as a function of the limit pressuremeter pressure, for each material type and construction method. The pressuremeter is widely used in France for obtaining an initial estimate of the ultimate ground-nail pullout resistance, but this technique has not found widespread acceptance in U.S. practice to date.

CHAPTER 4. SOIL NAIL WALL DESIGN

4.1 Introduction

FHWA SOIL NAIL DESIGN METHOD

The recommended design method provides a complete and rational approach towards soil nail wall design, incorporating the following elements:

1. Based on slip surface limiting equilibrium concepts.

2. Incorporates the reinforcing effect of the nails, including consideration of the strength of the nail head connection to the facing, the strength of the nail tendon itself, and the pullout resistance of the nail-ground interface.

3. Provides a rational approach for determining the nominal strength of the facing and nail/facing connection system, for both temporary shotcrete facings and permanent shotcrete or concrete facings. These strength recommendations are based on the results of both full-scale laboratory destructive tests to failure and detailed structural analysis.

4. Recommends design earth pressures for the facing and nail head system, based on soil-structure interaction considerations and monitoring of in-service structures.

5. Addresses both Service Load Design (SLD) and Load and Resistance Factor Design (LRFD) approaches.

6. For SLD, provides recommended allowable loads for the nail tendon, the nail head system and the pullout resistance, together with recommended factors of safety to be applied to the soil strength. Recommendations are separately provided for regular service loading, for seismic loading, for critical structures, and for temporary construction conditions.

7. For LRFD, provides recommended load factors and design strengths (i.e., resistance factors to be applied to the nominal or ultimate strengths) for the nail tendon, the nail head system, the nail pullout resistance, and the soil strength. Recommendations are separately provided for regular service and extreme event (seismic) loading, for critical structures, and for temporary construction conditions.

8. Recommends procedures for ensuring a proper distribution of nail steel within the reinforced block of ground to enhance stability and limit wall deformation.

9. Identifies the facing reinforcement details to be considered, together with the facing and overall soil nail wall structure serviceability checks to be performed.

10. Designs the soil nails and wall facing as a combined integrated soil-nail-wall "system".

4.1.1 Limit States

To provide for an acceptable level of safety, including considerations of loss of function, the design procedure for soil nail retaining walls addresses the following important limit states:

Strength Limit State

The strength limit state is that limit state that addresses potential failure mechanisms or collapse states of the soil nail wall system. *Strength limit states* address stability under expected forces. *Extreme limit states* address survival under extreme loads, e.g., seismic loading.

Service Limit State

The service limit state is that limit state that addresses loss of service function resulting from excessive wall deformation and is defined by restrictions on stress, deformation, and facing crack width under regular service conditions.

4.1.2 Design Approaches

Two different design approaches are presented in this manual: Service Load Design (SLD), and Load and Resistance Factor Design (LRFD). Both design approaches are discussed in the following sections. Design of soil nail retaining walls may use either approach. It is not required that both approaches be used.

(a) Load and Resistance Factor Design (LRFD)

This design approach is defined in the AASHTO LRFD Bridge Design Specifications, 1st Edition (AASHTO, 1994) [29]. LRFD of soil nail retaining walls considers the strength limit state by ensuring that the design strengths of the nails and of the soil exceed the applied loads, multiplied by load factors that are appropriate to the level of uncertainty associated with the loads. Design strengths are determined by applying appropriate resistance factors to the nominal or ultimate strengths, to account for the variability of actual strengths. The resisting capacity of the nails is determined by both structural (i.e., design strengths of the tendon) and geotechnical (i.e., design pullout resistance) elements. The resisting capacity of the soil is determined by applying a resistance factor to the ultimate soil strength. Several combinations of loading are applied to capture the maximum potential destabilizing effect of the loads, in order to define the maximum demand on the resisting elements.

The service limit state is investigated by addressing the overall displacements of the wall and of the reinforced and retained ground, and by applying limitations on crack widths (steel stresses) in the wall facing, in certain cases.

(b) Service Load Design (SLD)

This design approach is defined in the Standard Specifications for Highway Bridges, 15th Edition (AASHTO, 1992) [30]. SLD of soil nail retaining walls is generally similar to the LRFD approach and requires that the allowable nail loads and the factored soil strengths exceed the applied loads. The allowable nail loads are determined by both structural (i.e., allowable tendon stresses or loads) and geotechnical (i.e., allowable pullout resistance) elements. The factored soil strength is determined by applying a factor of safety to the ultimate soil strength. In order to define the maximum demand on the resisting elements, several combinations of loading are applied to capture the maximum potential destabilizing effect of the loads.

The service limit state is addressed with SLD, as described for LRFD.

4.2 Soil Nail Wall Stability Considerations

To address the strength limit state condition for a soil nail retaining wall, all potential failure modes must be considered. These failure modes include *external* modes that do not specifically intersect the reinforcements themselves, *internal* modes that involve failure of either the reinforcing tendons or the facing or both, and so-called *mixed* failure modes that involve internal failure of the reinforced zone and which extend beyond the physical limits of the reinforced block of ground (figure 4.1). Both internal and mixed failure modes involve consideration of yield or rupture of the nail, pullout of the nail, and failure of the wall facing or of the facing's connection to the nail.

Local stability of the facing during excavation is one of the most important considerations in soil nail wall construction. This failure mode is not amenable to conventional stability analysis and is typically addressed during design by a field test cut to demonstrate that the face can stand unsupported for sufficient time to allow nail and construction facing installation. Local sloughing of the face, possibly extending through to the surface, can be relatively sudden and is most prevalent at shallow depths where loose fill/highly weathered material is more likely to be encountered.

4.2.1 Basic Concepts

The limiting equilibrium approach to soil nail wall strength limit state design is summarized on figure 4.2. The method is demonstrated for a potential planar slip surface in which a global factor of safety is defined as the ratio of the resisting to driving forces along the potential slip surface. Figure 4.2 presents, in simplified form, the limiting equilibrium strength limit state design concept for both SLD and LRFD.

The equilibrium of an unreinforced block of ground is initially addressed in figure 4.2. Figure 4.2 (a) shows a free body diagram on the left, acted upon by the self weight of the block of soil located above the slip surface, and by the normal and shear forces along the slip surface. Considering force equilibrium of the block enables calculation of the normal and shear forces on the potential sliding plane. The factor of safety can then be defined as the ratio of the resisting forces to the driving forces, as shown. The expression for the global factor of safety F is a conventional factor of safety for an unreinforced slope. Shown next to the free body diagram is a conventional force polygon in which the factor of safety F is that factor that, when applied to both the cohesive and frictional components of the soil shear strength, will close the force polygon and satisfy limiting equilibrium. For the planar slip surface considered, the same expression for the global factor of safety F, as derived from considering equilibrium of the free body diagram, can be derived from the force polygon.

Next, a single nail reinforcing element is introduced to examine the manner in which the reinforcement improves the factor of safety or the stability of the sliding block of ground (figure 4.2 (b)). The global factor of safety F can again be derived from a consideration of either the free body diagram or the force polygon. **The effect of the reinforcement is to improve stability by both a) increasing the normal force and hence the shear resistance along the**

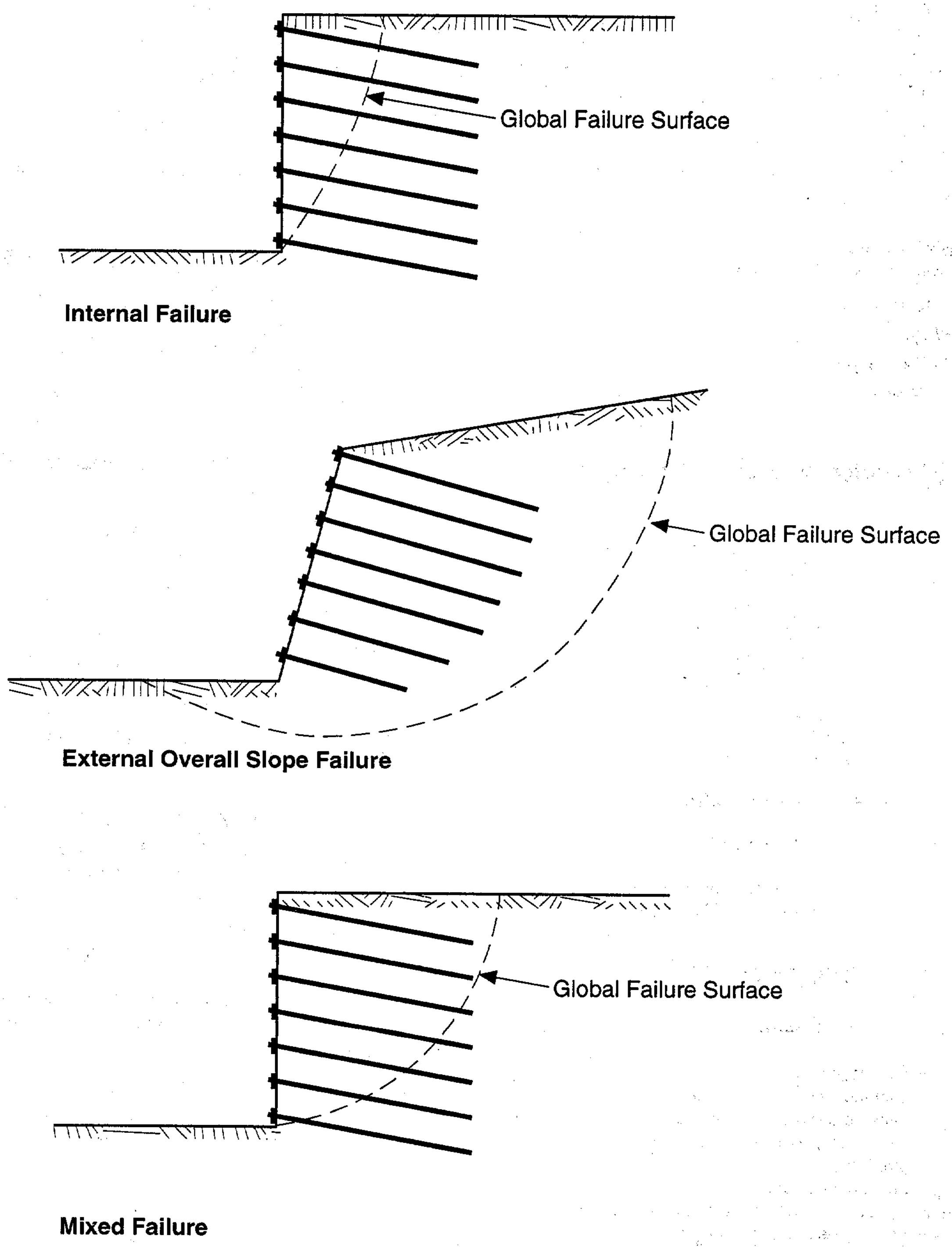
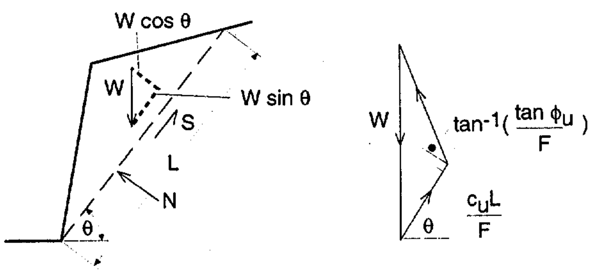


Figure 4.1 Potential Failure Modes to be Analyzed for Soil Nail Walls

(a) Unreinforced Slope

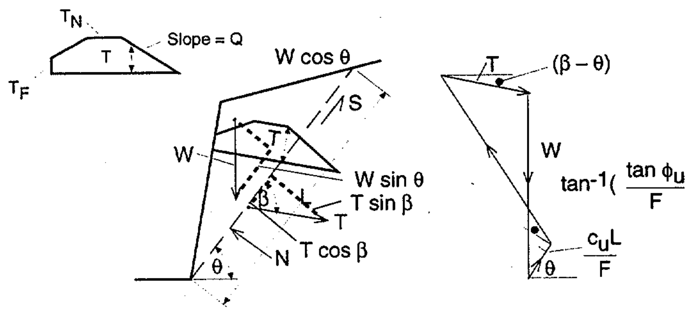


$$F = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{W \sin\theta}$$

$$= \frac{c_u L + N \tan\phi_u}{W \sin\theta}$$

$$= \frac{c_u L + W \cos\theta \tan\phi_u}{W \sin\theta}$$

(b) Reinforced Slope - Single Nail

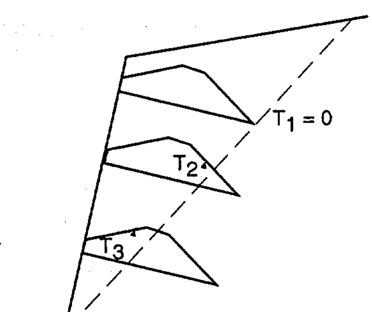


$$F = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{(W \sin\theta - T \cos\beta)}$$

$$= \frac{c_u L + N \tan\phi_u}{(W \sin\theta - T \cos\beta)}$$

$$= \frac{c_u L + (W \cos\theta + T \sin\beta) \tan\phi_u}{(W \sin\theta - T \cos\beta)}$$

(c) Reinforced Slope - Multiple Nails



For slip surface shown

$T = T_2 + T_3$

Expressions for Factor of Safety as given in (b).

TERMINOLOGY

Service Load Design

- W = service load (F)
- C = ultimate soil cohesion (F/L^2)
- φ = ultimate soil friction angle (°)
- F = global factor of safety applied to soil shear strengths
- T_{FN}= nominal nail head strength (F)
- $T_F = \alpha_F T_{FN}$ = allowable nail head load (F)
- T_{NN}= nominal nail tendon strength (F)
- $T_N = \alpha_N T_{NN}$ = allowable nail tendon load (F)
- Q_u = ultimate pullout resistance (F/L)
- $Q_d = \alpha_Q Q_u$= allowable pullout resistance (F/L)
- T = allowable nail load (F)
- S = resisting shear force (F)
- N = normal force (F)

(d) Demonstration Example (SLD)

Unreinforced Wall [see (a)]

$$F = \frac{c_u L + W \cos\theta \tan\phi_u}{W \sin\theta}$$

for

$cu = 5\ kN/m^2$
$L = 5\ m$
$\phi_u = 30°$
$W = 115$ kN/m of wall length
$\theta = 60°$

$$F = \frac{5 \times 5 + 115 \cos 60° \tan 30°}{115 \sin 60°}$$

$= 0.58$

Reinforced Wall [see (b)]

$$F = \frac{c_u L + (W \cos\theta + T \sin\beta) \tan\phi_u}{(W \sin\theta - T \cos\beta)}$$

for

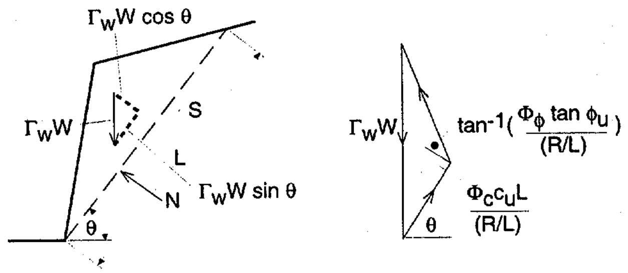
$\beta = 75°$
$T = 80$ kN/m of wall length

$$F = \frac{5 \times 5 + (115 \cos 60° + 80 \sin 75°) \tan 30°}{(115 \sin 60° - 80 \cos 75°)}$$

$= 1.3$

Figure 4.2A Soil Nail Design - Basic Concepts and Terminology (SLD)

(a) Unreinforced Slope

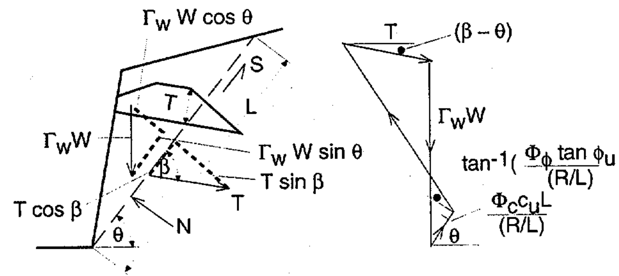


$$(R/L) = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{\Gamma_w W \sin\theta}$$

$$= \frac{\Phi_c c_u L + N\,\Phi_\phi \tan\phi_u}{\Gamma_w W \sin\theta}$$

$$= \frac{\Phi_c c_u L + \Gamma_w W \cos\theta\ \Phi_\phi \tan\phi_u}{\Gamma_w W \sin\theta}$$

(b) Reinforced Slope - Single Nail

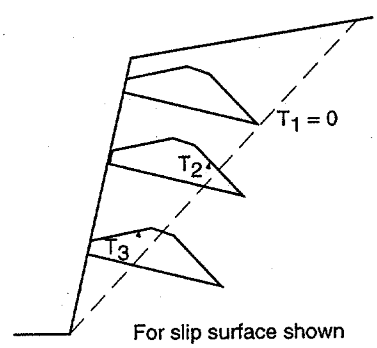


$$(R/L) = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{S}{(\Gamma_w W \sin\theta - T\cos\beta)}$$

$$= \frac{\Phi_c c_u L + N\,\Phi_\phi \tan\phi}{(\Gamma_w W \sin\theta - T\cos\beta)}$$

$$= \frac{\Phi_c c_u L + (\Gamma_w W \cos\theta + T\sin\beta)\,\Phi\phi \tan\phi_u}{(\Gamma_w W \sin\theta - T\cos\beta)}$$

(c) Reinforced Slope - Multiple Nails



For slip surface shown

$T = T_2 + T_3$

Expressions for Factor of Safety as given in (b).

TERMINOLOGY

Load and Resistance Factor Design

W = service load (F)
Γ_w = load factor
C_u = ultimate soil cohesion (F/L^2)
ϕ_u = ultimate soil friction angle (°)
Φ_c = cohesion resistance factor
Φ_ϕ = friction angle resistance factor
c = $\Phi_c c_u$ = design soil cohesion (F/L^2)
ϕ = $\tan^{-1}(\Phi_\phi \tan\phi_u)$ = design soil friction angle (°)
(R/L) = global resistance to load ratio
T_{FN} = nominal nail head strength (F)
Φ_F = nail head resistance factor
T_F = $\Phi_F T_{FN}$ = design nail head strength (F)
T_{NN} = nominal nail tendon strength (F)
Φ_N = nail resistance factor
T_N = $\Phi_N T_{NN}$ = design nail tendon strength (F)
Q_u = ultimate pullout resistance (F/L)
Φ_Q = pullout resistance factor
Q_d = $\Phi_Q Q_u$ = design pullout resistance (F/L)
T = design nail strength (F)
S = resisting shear force (F)
N = normal force (F)

Figure 4.2B Soil Nail Design - Basic Concepts and Terminology (LRFD)

slip surface in frictional soils and b) reducing the driving force along the slip surface in both frictional and cohesive soils. The improvement in the factor of safety as a result of the installation of the reinforcement is demonstrated for the SLD formulations in figure 4.2 (d).

Of particular importance is the shape of the nail strength diagram, indicated in figure 4.2(b) and further presented on figure 4.3 for clarity. Figure 4.3 shows that, for any particular sliding wedge, the reinforcing contribution of the nail is a function of the location at which the associated slip surface intersects the nail. The nail reinforcing strength may be limited by tensile failure of the nail tendon, pullout of the nail, or structural failure of the facing/nail head connection system. The contribution of any nail to the stability of a particular sliding block will be the *least* of a) the tensile strength of the nail, b) the pullout resistance of the length of nail beyond the slip surface, or c) the nail head strength plus the pullout resistance of the length of nail between the slip surface and the face of the wall.

Finally, multiple nails are considered (figure 4.2 (c)) as a simple extension of the single nail problem, to demonstrate the basic design methodology presented in this Manual. Although the methodology is demonstrated for only a single slip surface, all potential slip surfaces must be examined to ensure that the design is complete. Slip surfaces of other than planar shape (e.g., circles, log spirals, bilinear wedges, etc.) are preferred in examining limiting equilibrium states, since a) they generally provide lower calculated factors of safety and b) the planar slip surface can be closely approximated by these more general shapes.

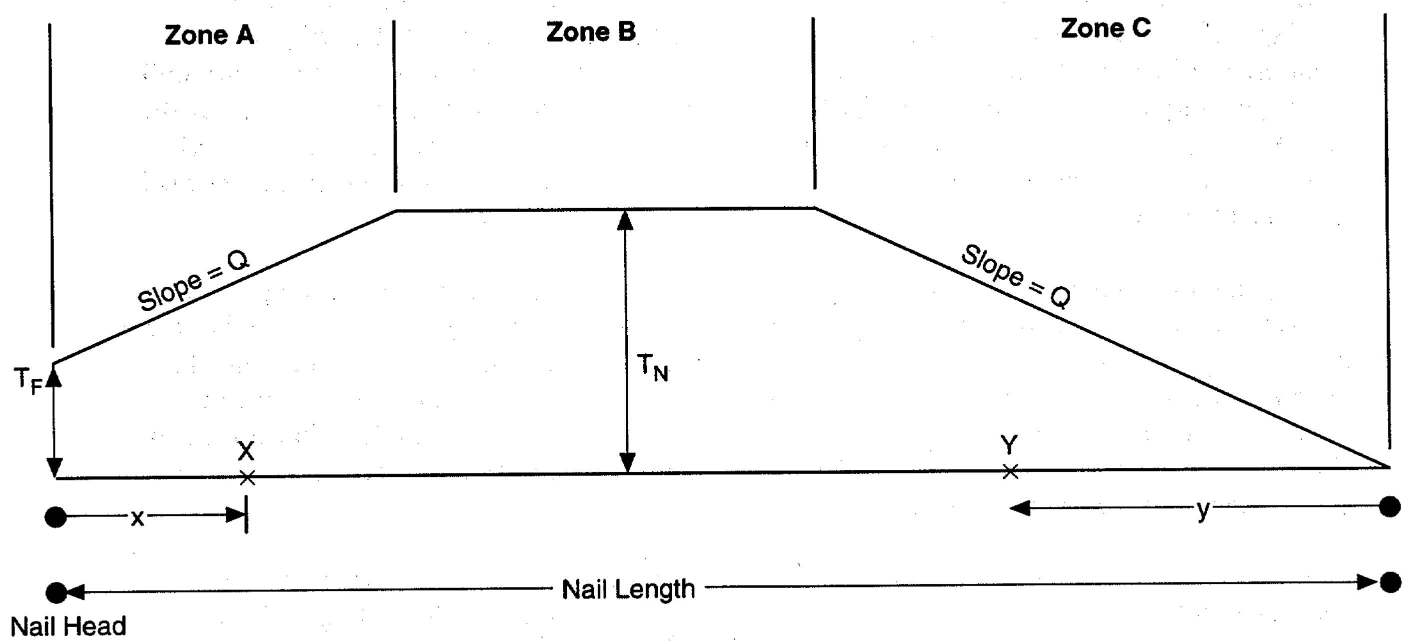
Figure 4.2 (c) also shows how the available design support from any particular nail for any particular sliding block of ground depends on where the nail intersects the sliding surface, as discussed above with reference to figure 4.3. Examining figure 4.2 (c), it can be seen that for the identified slip surface, the upper nail does not intersect the slip surface and therefore does not contribute to its stability. However, the upper nail does contribute to the stability of shallower slip surfaces (closer to the excavation) that intersect the nail. The middle nail provides support T_2 that is equal to the pullout resistance of the length of nail beyond the slip surface. The bottom nail provides support T_3 that is equal to the strength of the nail head together with the pullout resistance of the length of the nail between the slip surface and the facing, at that location. Although not demonstrated by the example in figure 4.2 (c), the structural tensile strength of the nail tendon itself may also limit the available nail support.

4.2.2 Internal Stability - Nail

As discussed above, the maximum reinforcing contribution of individual nails may be limited by any of the following:

- Nail tendon structural tensile strength.
- Ground-grout bond defined by either pullout or unacceptable creep behavior.
- Grout-tendon bond.

These elements are discussed below.



Nail Support to Slip Surfaces intersecting the Nail in Zone A at Point X = T_F +Qx

Nail Support to Slip Surfaces intersecting the Nail in Zone B = T_N

Nail Support to Slip Surfaces intersecting the Nail in Zone C at Point Y = Qy

T_F	= strength of nail head-facing connector	= Allowable Nail Head Load (SLD) = Design Nail Head Strength (LRFD)
T_N	= nail tendon tensile strength	= Allowable Nail Tendon Load (SLD) = Design Nail Tendon Strength (LRFD)
Q	= nail-ground pullout resistance	= Allowable Pullout Resistance (SLD) = Design Pullout Resistance (LRFD)

Figure 4.3 Nail Support Diagram

(a) Nail Tendon Tensile Strength

If the applied nail loading is greater than the structural strength of the nail tendon itself, yield and subsequent rupture may occur. The nominal nail tendon strength, T_{NN}, will be used to define the maximum structural tensile strength of the nail tendon as follows:

$$T_{NN} = A_b F_y$$

Where A_b = nominal area of the bar from table F2.

As noted previously, only the tensile strength of the nail is considered, and shear/bending contributions that may develop following significant deformations of the soil are neglected, which is conservative.

(b) Ground-Grout Bond

Construction of an economical soil nail wall is highly dependent on an ability to develop adequate bond or pullout at the nail grout - ground interface. Low pullout resistance will require long nails relative to the wall height and/or larger diameter nail drill holes to provide a higher pullout resistance per unit length of nail. The factors that exert most control on the ultimate bond that can be achieved at the ground-nail grout interface include the soil characteristics (plasticity, strength, grain size distribution), the drilling method and method of cuttings removal, and the grouting pressure. If inadequate bond is obtained and conventional nail lengths installed, then failure by nail pullout may occur. Such a failure occurred in 1980 at the Eparris wall [7] in France, where a wall constructed in a highly plastic clayey soil failed by nail pullout following a rainy season.

In addition to actual nail pullout by failure of the ground-grout bond, long-term creep of the soil at the ground-grout interface must also be considered. For creep susceptible soils, the rate of creep of a test nail will increase as the test nail load increases. From a practical experience perspective, a creep rate exceeding 2 mm per log cycle of time (6 to 60 minutes) in a short-term field test is considered unacceptable. Unacceptable creep behavior is more likely to be associated with softer, more highly plastic, cohesive soils. Short-term creep tests are a standard part of nail pullout testing and, in marginal soils exhibiting borderline creep rates, longer term tests might be required to assess service life deflections.

(c) Grout-Tendon Bond

For deformed reinforcing bars and continuous threadbars used for nail tendons, the bond between the grout and nail tendon is primarily a result of mechanical interlock, in which the grout mobilizes its shear strength against the bar deformations, and the ultimate strength of the tendon can be developed within a short embedment length in the grout (e.g., 12 to 15 bar diameters). The loose powdery rust appearing on bars after short exposures before installation has no significant effect on the grout-tendon bond.

Grout-tendon bond (in terms of force per unit length of nail) is typically an order of magnitude or more higher than the ground-grout bond and is therefore not critical for soil nailing applications when proper grout mix and installation techniques are used.

4.2.3 Internal Stability - Nail Head

The strength of the nail head may be controlled by the flexural strength of the wall facing, the punching shear strength of the facing and connection system, or the tensile capacity of the headed studs that are typically used in a permanent wall facing connection system. Other potential failure mechanisms do exist for the nail head. However, as discussed later in section 4.5, these modes will usually never control the design or limit the nail head strength for the types of systems commonly employed in soil nail wall construction.

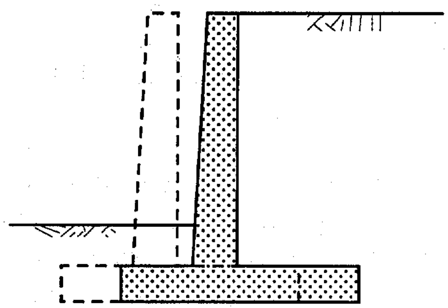
The nail head strength defines the available reinforcement strength at the head of the nail, which is one of the elements required to define the overall reinforcing capacity of the nail (figure 4.3). Analysis of the nail head strength is presented in section 4.5 and demonstrated by example in appendix F.

4.2.4 External Stability

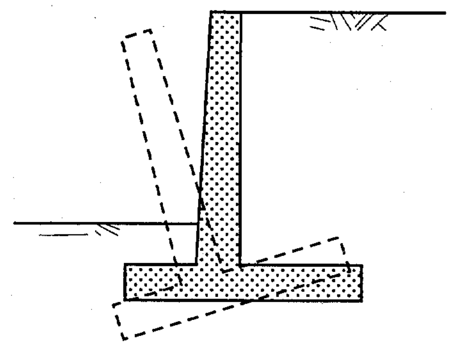
External stability refers to the potential deformation modes typically associated with conventional gravity or cantilever retaining structures and may involve consideration of (figure 4.4):

- Horizontal sliding of the retaining structure along its base, under the lateral earth pressure of the ground retained behind the reinforced mass.
- Foundation bearing failure of the retaining structure, associated with overturning, under the combined structure self weight and lateral earth pressure loading.
- Overall slope stability of the ground on which the retaining structure is located.

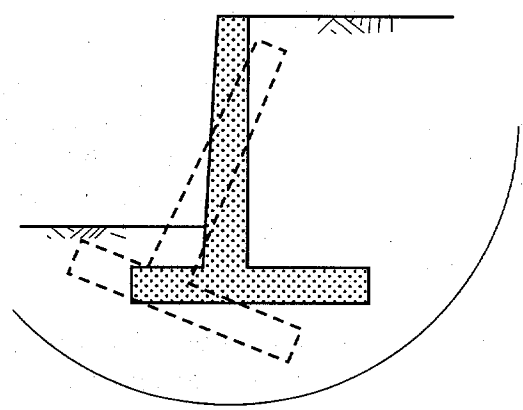
Separate consideration of the external stability modes shown on figure 4.4 is typically undertaken for "earth pressure" limiting equilibrium design methods, in which the reinforcement is apportioned on the basis of design earth pressures within the reinforced zone. To ensure that certain failure modes have not been overlooked when an "earth pressure" method is used, it is necessary to separately consider the possible "external" modes. When a "slip surface" limiting equilibrium design method is used for soil nail wall design, the horizontal sliding mode of deformation will typically be considered by the suite of slip surfaces examined, and does not therefore require separate consideration.



(a) Sliding Failure Along Base



(b) Bearing Failure Associated with Overturning



(c) Deep-seated Overall Slope Sliding Failure

Figure 4.4 "External" Failure Modes for Retaining Walls

Excavations in deep deposits of soft to medium clays can move excessively if the weight of the retained soils exceeds the bearing capacity of the soil at subgrade or a deep seated failure develops. Retained excavations in granular soils are generally not subjected to basal instability since the walls are free-draining and the shear strength is adequate at the base. The exception for granular soils is the case where substantial hydrostatic forces build up behind the wall due to inadequate drainage.

The external stability of soil nail walls which are constructed in clay soils must consider the reduction with time in the factor of safety, excess pore water pressures, and shear strength. For cuts in overconsolidated clays, the long term reduction in shear strength can be appreciable.

Designers should use general bearing capacity theory to check the foundation stability of soil nail walls. The reinforced gravity wall created by soil nailing will be acted on by self-weight together with earth pressure loads from the retained soil. Standard bearing capacity reductions for both inclined and eccentric loading should therefore be considered. The following should also be considered:

- The geometry of the general bearing capacity failure surface extends to a depth of about 1.5 times the width of the footing in relatively homogeneous soils. The typical base width of soil nail walls may greatly exceed typical foundation widths and this requires the designer to consider the soil and groundwater conditions to greater depths than would be common for conventional footings. Changes in soil type or strength and the presence of groundwater in the failure depth can substantially affect results.

- For fine-grained soils, both drained and undrained loading conditions should be evaluated. Construction of a soil nail wall involves unloading of the soil in front of the wall and this can result in long-term degradation of soil strength in this area of the foundation. For these conditions, undrained strength analyses relevant to short-term construction conditions may be less critical than long-term drained strength analyses.

- For depths of clay beneath the wall that are on the order of the width of the nailed block, general bearing capacity methods that account for eccentric and inclined loading should be applied. A minimum factor of safety of 2.5 is required.

- For depths of clay beneath the wall that are significantly less than the width of the nailed block, bearing failure modes may be limited to a portion of the nailed block. Under these conditions, there may be essentially no net lateral loading on the nailed block portion, since the nailed tensile loads may balance the earth pressure loads. In addition, the weight of the block may be partially supported by side shear forces acting along the vertical failure surface that passes through the soil nail block. Under these conditions the following applies:

$$FS = \frac{N_C C_U}{H(\gamma - C_U / y)} \leq 2.5$$

Where: H = height of excavation
y = cohesive soil depth below subgrade << width of nailed block
C_U = ultimate cohesion
γ = unit weight.
N_C = bearing capacity factor

- Overstress of thin soft layers immediately below the assumed footing level are not accounted for in the general bearing capacity approach. Soft soil layers that exist within a depth less than the footing width should be analyzed for overstress. In addition, wedge or other non-circular surfaces through the soft layer should be checked.

In general a rigorous analysis of bearing capacity will only be required in cohesive soils under the following conditions:

- For cohesive soil depth below subgrade equal to the width of the nailed block.

$$FS = \frac{5.14C_u}{\gamma H} \leq 2.5$$

- For cohesive soil depth below subgrade less than the width of the nailed block.

$$FS = \frac{5.14C_u}{H(\gamma - C_u / y)} \leq 2.5$$

Where: H = height of excavation
y = cohesive soil depth below subgrade
C_u = ultimate cohesion
γ = unit weight

Note that the cohesion used in the equations should reflect the average strength of the soil depth under consideration. In the case of a soft soil layer over a stronger cohesive layer, both equations should be used to find the lowest safety factor. Also, not that these analysis are derived for infinitely long, infinitely wide excavations. Narrow excavation widths or excavations of limited length have increased stability as N_C values can increase substantially depending on excavation geometry.

Overall slope stability (i.e. slip surfaces completely external to the reinforced block of ground) may be examined using conventional slip surface limiting equilibrium slope stability models. (e.g. XSTABL or other computer code).

4.3 Design Method Evaluation

The common design approach for assessing strength limit state conditions for earth retaining structures of all types is the limiting equilibrium method. There are two basic variations of the limiting equilibrium method commonly employed - "earth pressure" and "slip surface" methods - and each exhibits certain characteristics. In terms of the calculated total earth loading, each of these methods predicts essentially the same result if the same strength mobilization assumptions are made. In fact, the earth pressure coefficients may be derived directly from consideration of critical slip surface geometries (see figure 4.5). For internally reinforced systems, such as soil nail walls, there are some practical differences in relation to the application of "slip surface" and "earth pressure" limiting equilibrium methods, as discussed below.

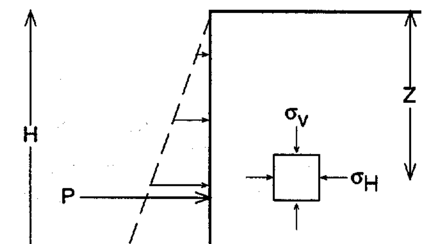
4.3.1 "Earth Pressure" Method

For earth pressure design of conventional cantilever or gravity retaining walls, an equivalent design earth pressure distribution is specified at the contact between the retaining structure and the retained earth. For "internally" reinforced retaining wall structures such as MSE walls, it is also necessary to define the location along each of the reinforcing strips at which the earth pressure should be considered to be applied, in order to design the required length of reinforcement to avoid pullout . This so-called "maximum tension line" is indicated on figure 4.6. The magnitude, distribution, and location of the design earth pressures are based on field experience and the results of experimental programs.

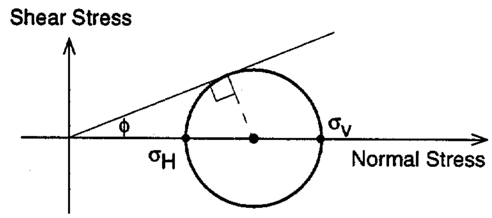
A benefit of the simplified earth pressure method when applied to internally reinforced retaining wall systems is that it accounts for both the local equilibrium of individual reinforcements as well as the overall equilibrium condition, when combined with the appropriate external stability checks. The earth pressure method defines not only the total reinforcement required, but also how that reinforcement should be distributed within the ground.

The earth pressure method is empirically based, both in terms of the magnitude of the design pressure distribution and particularly in terms of the location of the *assumed* maximum tension line. The shape and location of the assumed maximum tension line is dependent on the overall geometry of the system, the character of the reinforcements, and on the distribution of the applied loading. Supporting evidence for the location of the line of maximum tension is generally limited to the simplest geometric configurations and to relatively homogeneous soils and loading conditions. Since all possible failure modes are not explicitly examined with the earth pressure method, it is critical that the maximum tension line be carefully defined. This is possible for the design of MSE type walls where the granular backfill soil is specified and controlled and therefore has reasonably homogeneous and "standardized" properties. Such an approach is more problematical for soil nail wall design because soil nails are installed in possibly heterogeneous in-situ ground with a very wide range of soil shear strengths and ground-grout bond capacities.

Earth Pressure Method



γ = unit weight
ϕ = friction angle



$$\sin \phi = \frac{(\sigma_V - \sigma_H)}{(\sigma_V + \sigma_H)}$$

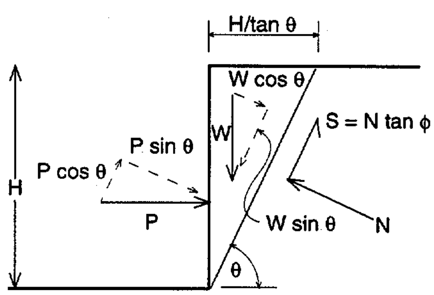
$$\therefore \sigma_H = \left(\frac{1 - \sin \phi}{1 + \sin \phi}\right) \sigma_V$$

$$= K_A \sigma_V$$

$$\text{and } P = (K_A \gamma H)\left(\frac{H}{2}\right) = \frac{K_A \gamma H^2}{2}$$

where K_A = active earth pressure coefficient

Slip Surface Method



γ = unit weight
ϕ = friction angle

Block Weight $W = \gamma H^2/(2 \tan \theta)$

$$\text{Factor of Safety} = \frac{\text{Resisting Force}}{\text{Driving Force}} = 1.0$$

Resisting Force $S = N \tan \phi = (W \cos \theta + P \sin \theta) \tan \phi$

Driving Force $= W \sin \theta - P \cos \theta$

$$\therefore (W \cos \theta + P \sin \theta) \tan \phi = W \sin \theta - P \cos \theta$$

$$P = W \frac{(\sin \theta - \cos \theta \tan \phi)}{(\cos \theta + \sin \theta \tan \phi)}$$

$$= \frac{\gamma H^2}{2 \tan \theta} \frac{(\tan \theta - \tan \phi)}{1 + \tan \theta \tan \phi)}$$

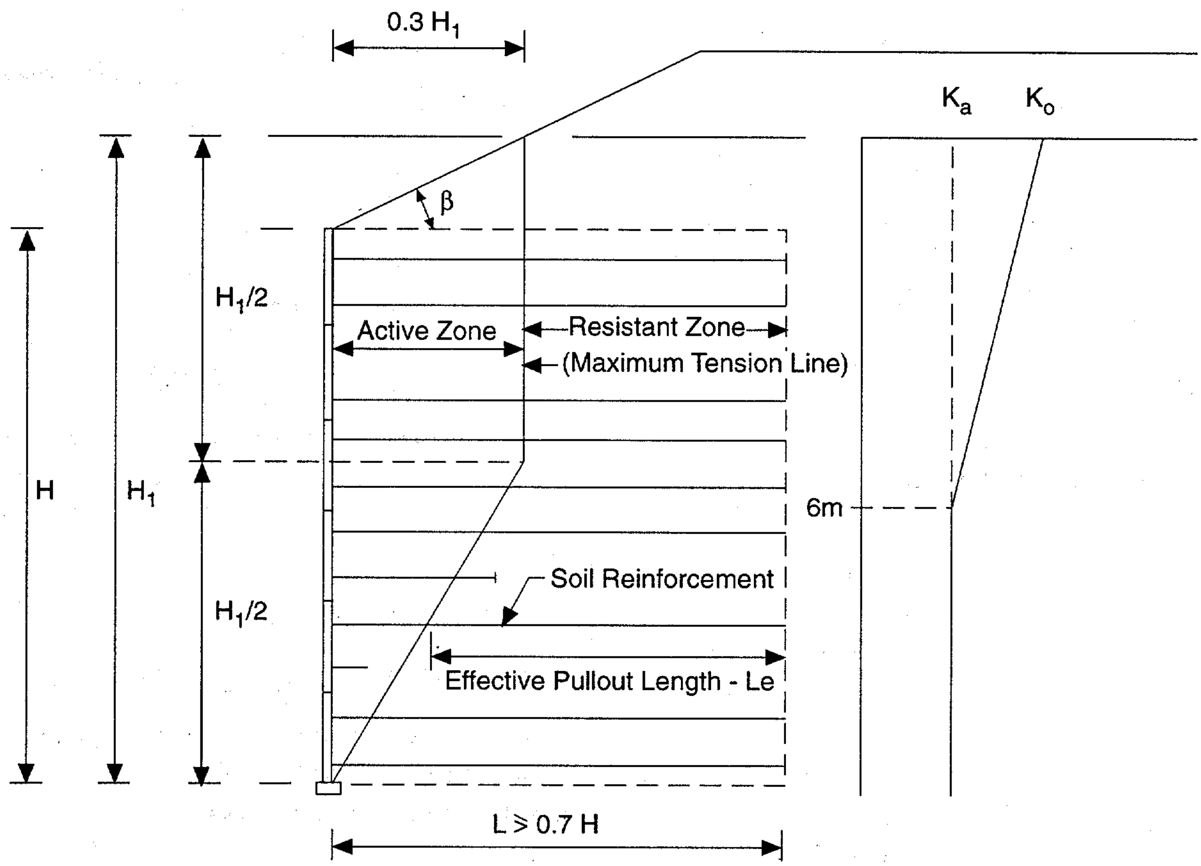
$$= \frac{\gamma H^2}{2} \frac{\tan (\theta - \phi)}{\tan \theta}$$

Critical Slip Surface occurs for $\theta = 45 + \phi/2$

$$P = \frac{\gamma H^2}{2} \frac{\tan (45 - \phi/2)}{\tan (45 + \phi/2)} = \frac{\gamma H^2}{2} \tan^2 (45 - \phi/2)$$

$$= K_A \frac{\gamma H^2}{2}$$

Figure 4.5 Earth Pressure and Slip Surface Methods Comparison



$$H_1 = H + \frac{\tan\beta \times 0.3H}{(1-0.3\tan\beta)}$$

Figure 4.6 Maximum Tension Line and Earth Pressure Coefficients for Inextensible Reinforcements (MSE Walls)

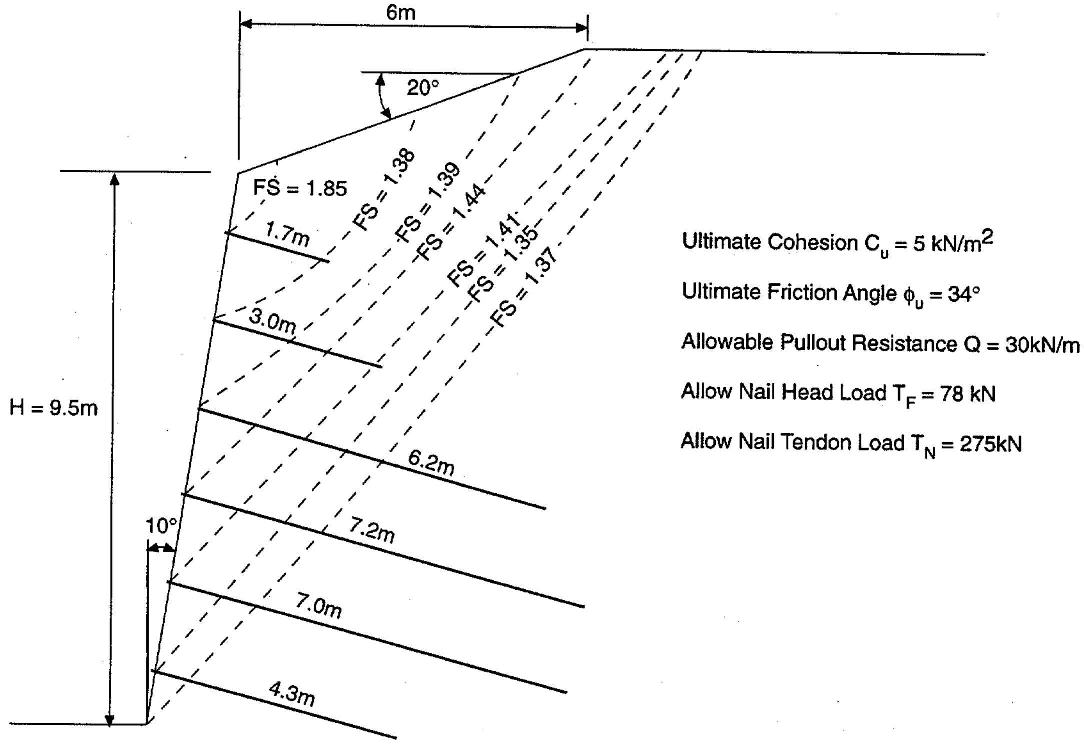
4.3.2 "Slip Surface" Method

Slip surface limiting equilibrium design methods consider the global stability of zones of ground defined by potential failure surfaces. These methods have been widely used in conventional slope stability analyses of unreinforced soil and have been demonstrated to provide good correlations with actual performance in such applications. Furthermore, virtually all current practical design methods for soil nail walls are based on the slip surface limiting equilibrium technique. As with the corresponding slope stability models, a critical slip surface is identified as that yielding the lowest calculated factor of safety, taking into account the support provided by the installed reinforcing. The chosen slip surface may be contained entirely or partially within the reinforced zone or entirely outside the reinforced zone. Hence, as noted in section 4.2.4, it is not necessary to separately consider certain "external" failure modes.

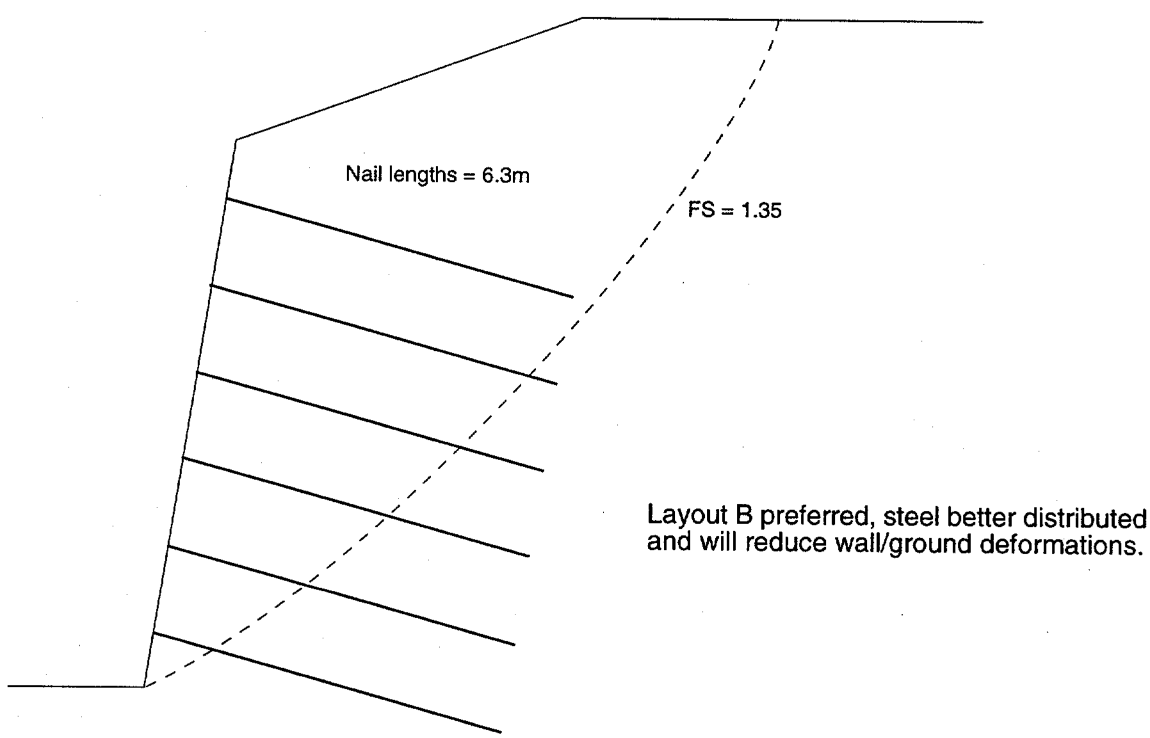
As with the classical slope stability limiting equilibrium models from which the soil nail models have been derived, a variety of slip surface shapes can be analyzed. These shapes include planar, bilinear, and piecewise linear surfaces, together with circles and log spirals. Some of the methods address only force balance, others only moment balance, and still others both force and moment balance.

The most significant benefits of the slip surface limiting equilibrium approach to soil nail wall design are 1) the method considers all internal, external, and mixed potential slip surfaces for the wall (bearing capacity of the nailed mass and overall stability of any slope on which the wall is constructed are typically evaluated separately) and evaluates global stability for each; 2) the method does not require specification of a maximum tension line and 3) the method is more convenient and accurate for heterogeneous geometries, soil types, and surcharge loadings than the simplified earth pressure method.

A limitation of the slip surface method in the design of reinforced soil structures is that it is possible to define a wide variety of reinforcement distributions that satisfy strength limit state requirements but that are not satisfactory from a serviceability perspective (i.e., result in excessive deformations of the reinforced mass). Figure 4.7 shows two fundamentally different nail layouts that result in calculated factors of safety that meet or exceed the minimum specified value for any slip surface examined. Layout A, while satisfying the minimum factor of safety requirements for any potential slip surface, would constitute an unsuitable design because of the deformations likely to be associated with such an arrangement of nails. Layout B represents a more typical design, with the same calculated minimum factor of safety. Clearly, it is not sufficient to simply specify strength limit state design criteria to ensure a satisfactory design. Other rules/constraints are therefore required to ensure a distribution of reinforcement that has been proven by experience to provide adequate performance of the wall under service conditions. These are discussed in section 4.7.



Layout A



Layout B

Figure 4.7 Different Nail Patterns Yielding Same Calculated Minimum Factor of Safety

4.3.3 Current Soil Nailing Design Methods

Nail Design

Most of the current design methods for soil nailed retaining structures are derived from classical slope stability analysis methods modified to incorporate the additional resisting tensile forces provided by the nail reinforcement. These methods of analysis evaluate "global" factors of safety along assumed failure surfaces. They include: German Method [31], Davis Method [32], and French Method [33].

The German Method assumes a bi-linear failure surface passing through the toe of the excavation [10]. The failing soil mass can be broken into two parts. The first part contains the nailed soil mass while the second forms the active earth pressure wedge behind the soil nailed "gravity wall." The analysis also considers the tensile resistance of the nails crossing the failure surface.

The Davis (Original Shen) Method incorporates a parabolic failure surface that also passes through the wall toe. The sliding surface either passes entirely through the nails or intersects the ground surface somewhere beyond the reinforced zone. In the analysis, the tensile and pull out resistance of the nails crossing the failure surface are considered the governing stabilizing forces.

The French Method (Talren) assumes a circular failure surface also passing through the toe. Unlike the previous two methods, this method can also consider the shear/bending contribution of the nails. It is reported that the inclusion of the nail shear/bending capacity provides only a few percent increase in the global factor of safety. Therefore, neglecting shear/bending provides a slightly more conservative design.

More recently, a kinematical limit analysis approach has been proposed for the design of soil nailed structures [25, 34]. It differs from the other analysis procedures in that the developer claims it provides a method for estimating actual mobilized "working' forces. The method assumes that the failure surface is defined by a log-spiral, and that failure occurs by a quasi-rigid rotation along this surface. The method also considers shear/bending in the nails. This method is theoretically and numerically complex, has not yet been presented in a form that can be easily understood or used by practicing engineers, and has been challenged by others as containing questionable theoretical assumptions.

Recently, two other limit equilibrium design methods have been developed in the U.S. These are the SNAIL design method developed by the California Department of Transportation (CALTRANS) and the GoldNail design method developed by Golder Associates of Redmond, Washington. The CALTRANS method uses a bi-linear or linear failure surface. The Golder method can analyze circular failure surfaces. Both methods consider the tensile resistance of the nails crossing the failure surface. These two methods are improvements over the previously mentioned limit equilibrium methods in that they design the soil-nail-wall facing as a system and: 1) Consider the limiting pull-out capacity of the nails on <u>both</u> the wall and non-wall sides of the failure surface; and 2) Allow the structural face capacity of the wall facing to be incorporated into the analysis.

An obvious recommendation is that detailed design of soil nailed walls should only be performed by experienced well-qualified geotechnical and structural engineers.

Facing Design

The facing has several functions in a soil-nailed structure [35]: it provides a lateral confinement to the soil between the nails and may carry external loads such as decorative panels. The first function is the most important one. Locally, between each nail, soil has to be retained. At the extreme, if the number of nails were infinite, i.e. spaced one next to the other, soil would not need to be retained as there would be no soil pressure against the facing. On the other hand it can easily be understood that in soil nail walls with very large spacing between the nails, the facing would play a major role and have to resist full soil earth pressure and would be very thick. For an infinitely large nail spacing, the facing would become a classical gravity retaining wall. Therefore, the nail spacing plays a major role in the facing design.

The density of nails is one major factor, but other factors including the rigidity of the facing itself play a role. The distribution of earth pressures on the facing between the nails is non-uniform. Arching effects tend to develop horizontally and vertically, which results in stress concentrations around the nail-face connections. These effects are particularly important for flexible facing, which is the case for soil nail wall facings.

The facing structural design requires provision of adequate concrete thickness, reinforcement, and moment capacity to resist the earth pressures applied to the facing span between adjacent nail heads, and provision of adequately sized bearing plates to provide adequate punching shear capacity.

4.3.4 Recommended Design Method

The slip surface limiting equilibrium method has been adopted in this Manual, for the strength limit state design of soil nail retaining walls. Theoretically, an earth pressure design approach could be formulated along the lines developed for MSE walls, but the following factors provide practical and strong support for the use of the slip surface method:

- Virtually all designers to date have used the slip surface approach, and there are no current empirical earth pressure recommendations that are sufficiently general to cover the wide variety of conditions (soil types, geometries, loading) typically encountered in soil nailing.

- Factors such as the heterogeneity of native soils makes the development of a general earth pressure specification for soil nail walls relatively complex.

- A critical issue for earth pressure methods is the definition of the locations of maximum tension for each of the reinforcements. These locations have not been defined for the variety of conditions generally encountered with soil nail walls.

Based on current knowledge, the slip surface method of analysis is therefore to be preferred. Structural engineers may be less familiar with the slip surface limiting equilibrium design approach than with the earth pressure approach. With the earth pressure approach, both the self weight of the ground/surcharge loads and the strength of the ground are combined to develop an "equivalent" earth loading or earth pressure. The ground strength is therefore incorporated on the "load" side of the equation, and the resistances of the structural components **only** are considered on the "resistance" side of the equation. Hence the resistances of the structural elements only are explicitly considered, and the structural engineer can execute his structural design analysis without any explicit consideration of the strength of the ground. The earth pressure design approach therefore leads to a de-coupling of the structural and geotechnical elements. The slip surface approach considers the same loads, together with the ground strength, but incorporates loads on the "load" side of the equation and the ground strength on the "resistance" side of the equation (along with the structural resistances). In this case, the structural resistances cannot be considered in isolation, and the design equation must explicitly incorporate not only the resistances of the structural elements but also the resistance of the ground. In the slip surface approach, therefore, the structural and geotechnical design analyses remain coupled.

The detailed design approach recommended for soil nail walls is presented in section 4.7, and demonstrated by worked examples in chapter 5. Before considering the detailed design approach, however, some wall layout and dimensioning issues are briefly addressed.

4.4 Layout and Dimensioning

Before performing detailed design calculations, the overall geometry (location, face batter, height) of the wall is defined which, together with the ground material properties, will determine the critical design sections for analysis. In addition, subsurface restrictions that could affect the nail layout must be identified and a preliminary nail pattern established.

4.4.1 Wall Location and Dimensions

The location of the wall facing will be established by its intended function, any right-of-way restrictions, together with related wall height constraints controlled by site geometry, environmental, economic, aesthetic or technical considerations. Vertical (vs. battered) walls and walls on tangent or circular radius provide for easier constructability, particularly for wall line and nail location field survey control. Use of wall line on spirals is not recommended for soil nail walls as this makes field survey and control of wall excavation and nail locations more difficult. Stepped walls are also possible and have been used for high walls or where desired for aesthetic reasons. **Once the design location of the top of the wall has been established, it is necessary to obtain a detailed topographic survey along the wall line so that the grade at the top of the cut can be precisely determined before the preparation of detailed plans.** This will ensure that 1) the upper nails are not inadvertently specified as being located above the ground surface; 2) local grading requirements can be identified (e.g., for surface water control); 3) the size of any upper cantilevered wall sections can be defined; and d) the quantities and locations of any required backfill can be determined.

If any buried utilities or other subsurface structures are present within the reinforced zone, they must be located so that impacts on design and construction can be identified.

Welded wire reinforcement, commonly referred to as a fabric or mesh, is used for reinforcement of shotcrete facings of soil nail walls. The selected dimensions of this fabric are important for both structural aspects and wall constructibility when shotcrete facings are used for either the construction or permanent facing. The structural issues involve wall flexure and nail head punching; particularly for the relatively thin shotcrete construction facing. The constructibility issue is less obvious but equally important. Wire fabric is manufactured in 1.5 to 2.5 m rolls or sheets. Roll widths of 1.5 m are common on the east coast while roll widths of 2.1 m are common on the west coast. The width of the fabric selected for a given project will vary depending on the height of the designed soil cut and the vertical spacing of the nails. The objective is to select a fabric width that will provide sufficient lap (AASHTO 8.32.6) below the height of the cut to permit splicing the next full width fabric sheet. E.g., if a 1.5 m cut was planned to be shotcreted, a fabric width of 1.5 m plus the lap splice width of about 0.2 m would be required. This fabric width may not be readily available on the east coast which would result in an extra splice in each lift. To avoid such problems the designer should check with local suppliers of wire fabric to determine the availability of different widths of fabric. If the only locally available fabric is 1.5 m, consideration should be given to a cut and shotcrete height of 1.3 m with the vertical nail spacing adjusted accordingly.

Table 4.1 contains the typical sizes of plain wire fabric, "W", which are commonly used for nail walls (as opposed to a designation of D for deformed wire fabric). The "W" number refers to the cross sectional area of the plain wire in hundredths of an inch, e.g., W1.4 has a cross section of .014 square inches. The "MW" designation refers to the metric soft conversion of the plain wire cross sectional area which is rounded to whole numbers in square mm, e.g., W 1.4 would be soft converted to 9.1 square mm and then rounded to 9.0 square mm for a designation of MW9.

TABLE 4.1
COMMON STYLES OF METRIC WELDED WIRE REINFORCEMENT WITH EQUIVALENT US CUSTOMARY UNITS[1]

Metric Styles (MW = Plain Wire)[2]	**A (mm^2/m)**	**Wt. (kg/m^2)**	**Equivalent US Customary Styles (W= Plain Wire)[3]**	**A (in^2/ft)**	**Wt. (lbs/ft^2)**
102x102 - MW9xMW9	88.9	1.51	4x4 - W1.4xW1.4	0.042	3.1
102x102 - MW13xMW13	127.0	2.15	4x4 - W2.0xW2.0	0.060	4.4
102x102 - MW19xMW19	184.2	3.03	4x4 - W2.9xW2.9	0.087	6.2
102x102 - MW26xMW26	254.0	4.30	4x4 - W4.0xW4.0	0.120	8.8
152x152 - MW9xMW9	59.3	1.03	6x6 - W1.4xW1.4	0.028	2.1
152x152 - MW13xMW13	84.7	1.46	6x6 - W2.0xW2.0	0.040	3.0
152x152 - MW19xMW19	122.8	2.05	6x6 - W2.9xW2.9	0.058	4.2
152x152 - MW26xMW26	169.4	2.83	6x6 - W4.0xW4.0	0.080	5.8

[1]Wire sizes may also be deformed, use prefix MD or D, except where only MW or W is required by building codes (usually less than a MW26 or W4). Also wire sizes can be specified in 1 mm^2 (metric) or 0.001 in^2 (US Customary) increments. Areas are based on lower bound tolerances for diameter.
[2]For other available styles or wire sizes, consult other WRI publications or discuss with WWR manufacturers.
[3]Styles may be obtained in roll form. Note: It is recommended that rolls be straightened and cut to size before placement.
Courtesy of Wire Reinforcement Institute (WRI), Findlay Ohio 45839-0450; 419-425-9473

4.4.2 Preliminary Nail Layouts

Definition of a *trial* nail layout pattern, including nail lengths, locations, spacings, strengths and inclinations, is required for design analysis.

Nail Inclinations

For nails installed in predrilled holes and grouted under gravity or low pressure, which is the predominant North American practice, the holes must be inclined downwards. An angle of 15 degrees is a common average declination. It is sometimes required to steepen the nail inclination, particularly with the first row of nails in order to avoid shallow utilities. The local slope stability in the area of these more steeply inclined nails must be carefully considered however, because the reinforcement efficiency (resisting component of the nail forces) decreases significantly with increased nail inclination. Local experience can be an important factor in determining the suitability of more steeply inclined nails. The bottom row of nails is also sometimes inclined at greater than 15 degrees because of drill rig access restrictions. In applications where there are headroom limitations, such as beneath a bridge deck, the upper nails may be installed with a flatter inclination. However, inclination angles of less than 5 degrees may be installed with a flatter inclination. However, inclination angles of less than 5 degrees should not be used since grouting of the nail drill hole will be difficult and there is a significantly increased potential for forming voids in the grout column. Constructability will also be simplified if the nail inclinations are maintained as uniform as possible.

Special attention should be paid to splayed nails that are installed for outside corners, as it is difficult to establish the angle of installation during construction, and the potential for interconnecting soil nail drillholes is high.

Nail Spacings

Nail spacings should be as large as possible for economic reasons and, for constructability reasons, as uniform as possible. Unless the ground is extremely competent, vertical and horizontal soil nail spacings are generally within the range of 1.5 to 2 meters, with a 1.5 meter nail spacing being most common for the drill and grout method of nail installation.

Nail Layout Locations

Nail head locations will be influenced by a number of factors. Nail columns can be vertical or offset row to row. Vertical columns provide for easier field layout and control of nail locations and provide more horizontal space for placement of the vertical geocomposite drain strips. The offset pattern will improve the excavation face stability during construction, through the enhanced development of soil arching. The offset pattern is especially recommended where it is anticipated that the excavation face may be marginally stable. Vertical nail columns may be preferable with some precast panel facing systems to facilitate facing connection and encapsulation of nail heads for corrosion protection. Constructability will also generally be easier if nail rows are laid out 1) parallel to the base of wall grade for longer relatively uniform height walls on steeper grades; 2) horizontally (for easier field survey and layout) for longer relatively uniform height walls with no or very slight bottom of wall grade, with periodic step-ups along the wall length if necessary; 3) top and intermediate nail rows parallel to top of wall profile and bottom row parallel with bottom of wall with transitions between the rows where required, for shorter variable height walls.

The upper row of nails should be placed to limit the height of the construction facing upper cantilever, above the top row of nails, to less than about 1.0 meter. The top row of nails should be approximately centered within the first shotcrete lift of the construction facing to minimize the potential for a toppling failure of the facing during initial construction. If longer upper facing cantilever sections are required, these can be achieved with the installation of the permanent facing following completion of the wall excavation. For top-down construction of the permanent facing, the upper cantilever height should not be increased beyond the above limit until at least the second row of nails and shotcrete have been constructed.

At bridge abutments, it should be verified that the design elevations for the first row of nails allows sufficient head-room for the drilling equipment to access and work beneath the deck. In addition, sufficient clearance from all existing foundations should be ensured.

For sites characterized by an upper soil horizon consisting of loose soil or fill, temporary or permanent flatter cut slopes at the top of the soil nail wall can often be used to allow the installation of the first row of nails at greater depth.

The bottom row of nails should be no closer to the finish grade than is consistent with the ability of the drilling equipment to install the nails. Conversely, the length of the bottom cantilever section of the wall should be generally no greater than about two-thirds of the average vertical nail spacing. Nail heads should also be positioned so as not to coincide with the shotcrete construction facing horizontal joints or the permanent CIP facing vertical expansion joints.

<u>Nail Lengths and Strengths</u>

Nail lengths and required strengths will be influenced by the same factors that affect the nail spacing, and will tend to increase with lower soil strengths , lower nail-ground pullout resistances, steeper face and backslope angles, and higher surcharge loadings. Depending on the combination of these variables, nail lengths can be in excess of the height of the wall. For many common applications, however, such as cut slopes with modest backslopes and minimal surcharge loadings, the nail length will usually be in the range of 0.6 to 1.0 times the height of the wall. A nail-length-to-wall-height ratio of less than 0.6 is rarely employed with steep wall slopes because of concerns about overturning stability. Shorter nail lengths have been used with battered walls in more lithified or rock-like materials. Soil nail wall design and construction practice has also often employed a uniform length of nail over the entire wall height. It is also common to provide a uniform-sized steel tendon over the full height of the wall. It is often possible at the detailed design stage to shorten the length of those nails located in the lower part of the wall. Because of the top down method of construction, the nails located in the upper approximate two-thirds of the wall are most effective in controlling wall displacements, and are also the most heavily loaded.

4.4.3 Caltrans Design Practice

At the time of preparation of this manual, Caltrans has constructed an estimated 75 soil nail walls. The following summarizes current Caltrans design practice for soil nail wall layout, nail layout pattern and wall contraction/expansion joints (per personal communication from Mr. Jim Moese, Caltrans Office of Structures, to R. Chassie/FHWA, August 1996):

For horizontal alignment, tangent and circular curvature alignment is preferred.

For top of wall profile, a smooth line is provided consisting of a smooth splined line or consisting of a series of tangents with parabolic curves transitioning between tangents.

Vertical facing is preferred for urban walls with CIP concrete facing (placing and vibrating concrete for battered walls is difficult). Battered facing is acceptable for all shotcrete facing for rural walls where alignment is not as critical.

Typically, the vertical column of soil nails pattern is used more than the staggered or offset pattern. The offset pattern should be recommended for sites where it is anticipated that the excavation face will be marginally stable.

Typically, the top and intermediate rows of soil nails are parallel with the top of wall profile and the bottom row parallel with the bottom of the wall, with transitions between the rows where required.

For facing with uniform thickness and an architectural finish with vertical relief such as fractured fin texture, no joints are required.

For facing with uniform thickness and uniformly smooth surface finish, expansion joints are used at approximately 30 meter maximum spacing and intermediate contraction joints at approximately 7.5 meter maximum spacing.

For facing with intermittent thicker vertical sections used for architectural relief, either expansion joints or contraction joints are placed each side of these thickened sections. Placement of joints between these thickened sections would depend on the surface finish as noted above.

4.4.4 Schnabel Foundation Company Design Practice

At the time of preparation of this manual, Schnabel Foundation company has constructed over 190 soil nail walls across the United States (per personal communication from Mr. David Weatherby, Vice-President, to R. Chassie/FHWA, 1996). The following summarizes current Schnabel design practice for soil nail wall layout for walls on which they control the design, i.e., temporary walls or design-build permanent walls (per personal communication from Messrs. David Weatherby and Claus Ludwig, to R. Chassie/FHWA, August 1996):

Vertical final finish facings are the most common.

The construction facing is frequently battered slightly to help maintain face stability. For example, when working in bouldery ground, the construction facing is battered rather than trying to remove the cobbles or boulders from the facing. Pulling the boulders and cobbles from the face could seriously disturb the facing above.

Cast-in-place faces are easier to place if they are vertical. However, for the construction reasons mentioned above, it may be desirable to batter the face. Aesthetics may lead the owner to batter the finish face. When battering cast-in-place walls, provisions have to be made to enable the concrete to be consolidated (vibrated). This may require special forms (windows), limited pour heights, modified rebar spacings, and thicker walls. Permanent shotcrete wall-facings can be battered easily.

When the batter is on the order of two horizontal to twelve vertical, the driving forces are reduced enough to make a difference in the design.

Walls are not battered to shorten nail lengths. Walls that are high are generally stepped.

Placement of vertical strip drains is more difficult, but still possible, if the nail pattern is staggered.

Generally, the first row of nails is located a maximum of 0.75 meters from the top of the ground surface (of course utilities, for example, may make this difficult). It is important to secure the upper shotcrete lift to the cut face or problems may develop during subsequent excavation stages, particularly if there is sloping ground above the wall.

4.5 Nail Head Strength

As discussed in section 4.2, the reinforcing contribution of the nail-facing system can be defined in terms of the nail tendon strength, the nail grout-ground pullout resistance, and the nail head strength. The first two items are relatively straightforward in concept. Determination of the nail head strength, however, is more complex and depends on several potential failure modes both of the facing and of the nail-facing connection system. These failure modes, were briefly presented in section 4.2.3, and are now considered in more detail in order to define the nominal nail head strength.

4.5.1 Nail Head Failure Modes

There are three critical failure mechanisms for a soil nail wall facing and connection system that must be checked. The shotcrete/concrete facing may fail either in (1) flexure, or (2) punching shear. In a permanent wall facing headed-stud connection system, (3) the headed studs may fail in tension. These failure modes are considered in sections 4.5.2 through 4.5.4.

There exist three other potential failure mechanisms for the wall facing and connection system. These mechanisms are not critical and therefore are not checked in the design procedure. These additional failure modes include one-way shear of the facing, flexure of the connection bearing plate, and shear of the connection bearing plate [36].

One-way shear of the facing is not critical because of the nail patterns typically used and because the deformational response of the facing system is three-dimensional. The nature of the cracking and general deterioration of the system, as it is loaded up to failure, is also three-dimensional. Physical testing and analytical studies performed at U.C. San Diego have confirmed this [36]. Therefore, one-way shear of the facing is not checked in the design procedure.

For typical width-to-thickness ratios of connection bearing plates being used, plate shear has never been found to control the design and is therefore not considered in the design procedure [36]. Laboratory tests have demonstrated that the post-yield flexural response of mild steel connection plates utilized in typical soil nail wall construction involves large deformations and local stress redistribution. Because the bearing plate steel is inherently ductile, outright failure never occurs. Instead, the plate continues to deform and redistributes pressures between the facing and the plate. Thicker plates merely result in decreased plate deformations as failure of the facing is approached.

Therefore, for mild steel plates with a minimum yield stress (F_Y) of 250 MPa, the design recommendation is to provide a minimum plate width of 200 mm and a minimum plate thickness of 19 mm. For plate widths or thicknesses less than the above minimum recommendations, the adequacy of the connection plate in flexure and shear should be demonstrated by hand calculation.

The following paragraphs consider the potential critical failure mechanisms that may control the nail head strength. For each failure mechanism, the procedure for determining the associated nominal nail head strength is presented. The controlling failure mechanism is defined as that which corresponds to the lowest calculated nail head strength.

4.5.2 Flexural Strength of the Facing

The flexural strength of the facing will develop through the formation of a critical pattern of yield lines throughout the facing. The actual form of the yield line pattern will depend on the nail pattern and the relative horizontal and vertical nail spacings. However, for similar horizontal and vertical nail spacings, the critical yield line pattern will tend to include radial negative moment yield lines about each nail head, with positive moment yield lines at locations midspan between nail heads.

The flexural stiffness of the facing increases with increasing thickness and reinforcement ratio and decreases with increasing nail spacing. As the facing flexural stiffness decreases with respect to the soil subgrade reaction modulus, the pressure distribution behind the facing will become highly non-uniform, with large pressure concentrations occurring behind the nail heads. The non-uniformity of the facing pressure distribution, as illustrated on figure 4.8, should be considered in design because the actual available nail head strength will generally be significantly larger than the strength that would be computed based on the conservative assumption of a uniform pressure distribution.

It has been found that for the common facing configurations employed in soil nail wall construction, the critical nominal nail head strength, T_{FN}, associated with the flexural capacity of the facing may be represented by the following relationship [36]:

$$T_{FN} = C_F\,(m_{V,NEG} + m_{V,POS})\left(\frac{8\,S_H}{S_V}\right)^{*} \qquad (4.1)$$

where $m_{V,NEG}$ and $m_{V,POS}$ are the vertical nominal unit moment resistances at the nail head and mid-span locations, respectively, and S_H and S_V are the horizontal and vertical nail spacings. The pressure factor for facing flexure, C_F, is determined from table 4.2:

TABLE 4.2
FACING PRESSURE FACTORS
RECOMMENDED VALUES FOR DESIGN

	Temporary Facings		Permanent Facings	
Nominal Facing Thickness (mm)	Flexure Pressure Factor C_F	Shear Pressure Factor C_S	Flexure Pressure Factor C_F	Shear Pressure Factor C_S
100	2.0	2.5	1.0	1.0
150	1.5	2.0	1.0	1.0
200	1.0	1.0	1.0	1.0

* Equation 4.1 is applicable to typical soil nail wall facing construction practice. For facing systems that involve either larger horizontal nail spacings than vertical nail spacings, or when the horizontal unit moment capacities are less than those in the vertical direction, then equation 4.1 should also be checked with unit moment capacities corresponding to the horizontal direction and with the vertical spacing substituted for the horizontal spacing and vice versa.

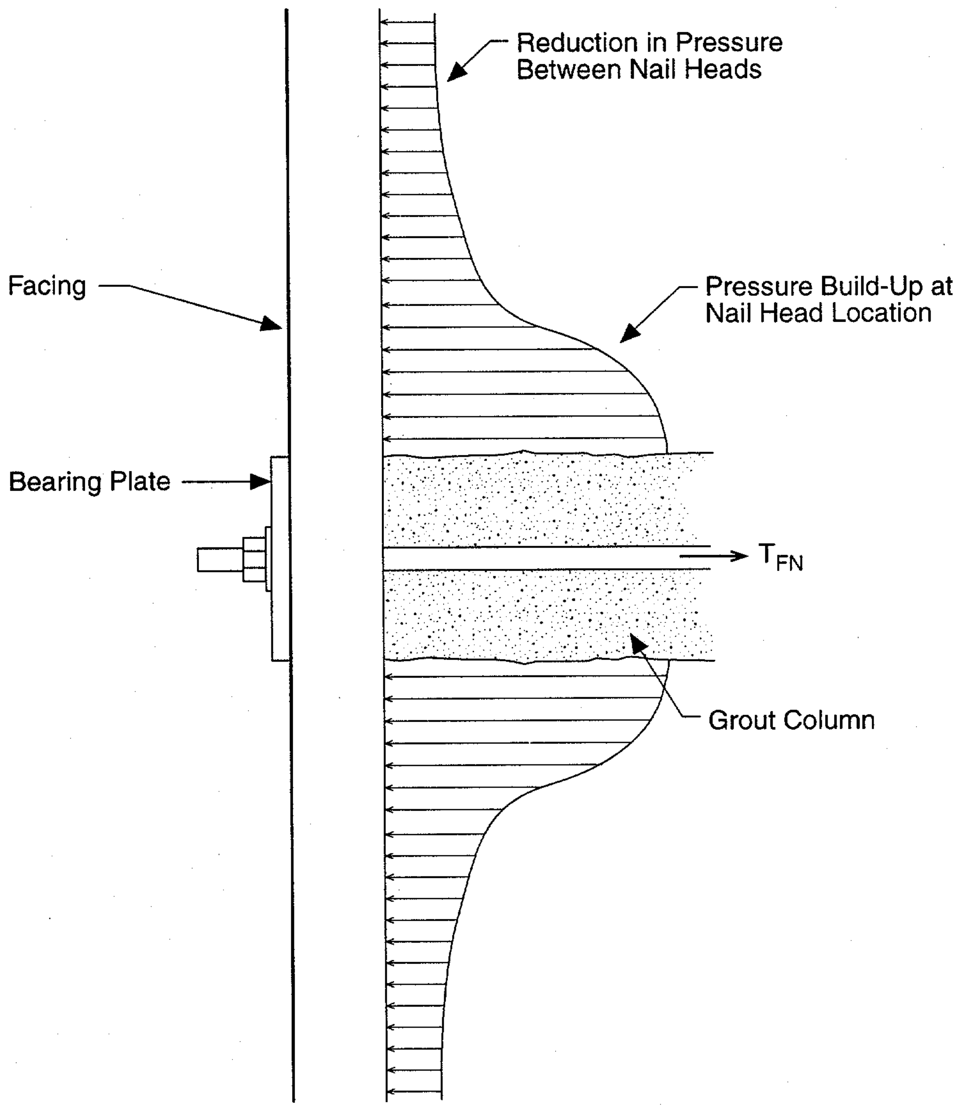


Figure 4.8 Typical Facing Pressure Distribution

The vertical nominal unit moment, m_v, may be found as follows:

$$m_v = \frac{A_S F_y \gamma}{b}\left(d - \frac{A_S F_y}{1.7 f'_c\, b}\right) \qquad (4.1A)$$

Where: A_s = area of tension reinforcement in facing panel width 'b'
b = width of unit facing panel (equal to S_H)
d = distance from extreme compressive fiber to centroid of tension reinforcement.

In general, all reinforcing steel in the construction facing is assumed to be located at the center of the section and the shotcrete thickness is assumed to be uniform.

Equation 4.1 may be derived by considering force and moment equilibrium of a typical interior facing panel loading by the soil, with point supports at the nail head locations and full plastic moment capacity developed at all applicable yield lines. The factor "C_F" may be considered to account for the non-uniformity of the contact pressure between the facing and the subgrade, with a value of 1.0 corresponding to a uniform contact pressure, and increasing values being reflective of increasing concentrations of contact pressure in the vicinity of the nail head

The recommendations for C_F in the above table are based on back analysis of case histories and full-scale laboratory tests, calibrated finite-element modeling, experience, and judgment [36].

Historically, full plastic moment development in slabs at the ultimate state has been considered only when the reinforcing steel is distributed according to where the moments develop under an elastic condition. For soil nail wall facings, one method of achieving that distribution is through the placement of horizontal waler bars and vertical bearing bars. Because soil nail wall construction practice in many parts of the country includes only waler bars in temporary facings and neither waler nor bearing bars in permanent facings, the C_F factor has been reduced to 1.0 for permanent wall facings. It is required that continuous horizontal waler bars be incorporated at each row of nails in a temporary shotcrete facing as the walers passing beneath the nail head bearing plate provide an element of ductility in the event of a punching shear type of failure. Vertical bearing bars at each nail head are optional, and may be used as an alternative to increasing the mesh size if additional vertical moment resistance is required from the facing.

In addition, for uncommonly large reinforcement percentages, the available analytical data at this time indicate that equation 4.1 may over-predict the available nail head strength [36]. As the amount of flexural reinforcement is increased, the actual value for C_F tends to decrease because the facing is actually more stiff with respect to the soil. With the unit moment capacities increasing and C_F decreasing, the net result may be little change in the nail head strength. It is therefore recommended that until further information becomes available, a reinforcement ratio (based on gross area) of no more than 0.35 percent should be considered when calculating the nail head strength using equation 4.1 and using a C_F value greater than 1.0 (i.e., for temporary facings with a thickness of less than 200 mm).

In instances other than the above, and where less common facing designs are employed, all potential yield-line mechanisms should be investigated to determine the critical nail head strength.

4.5.3 Punching Shear Strength of the Facing

(a) General

Punching shear failure of the facing involves the punching of a cone-shaped block of shotcrete or concrete, centered about the nail head, through the facing. The shape of the facing rupture surface depends slightly on the type of connection between the nails and the facing.

Primarily two types of connection systems are currently used in soil nail wall construction (figure 4.9). Only these two systems will be addressed here. For temporary shotcrete construction facings, the bearing-plate connection is most commonly used, wherein a steel plate bearing against the front or excavation side of the facing is connected to the nail tendon using a washer and nut. For permanent CIP concrete facings cast over the temporary shotcrete construction facing or for full-thickness permanent shotcrete facings, a headed-stud connection is most commonly used. As shown on figure 4.9, the headed-stud connection consists of four headed studs welded to the bearing plate, with the entire connection system embedded within the permanent wall facing. In either connection system, failure develops by the punching of a truncated cone through the facing.

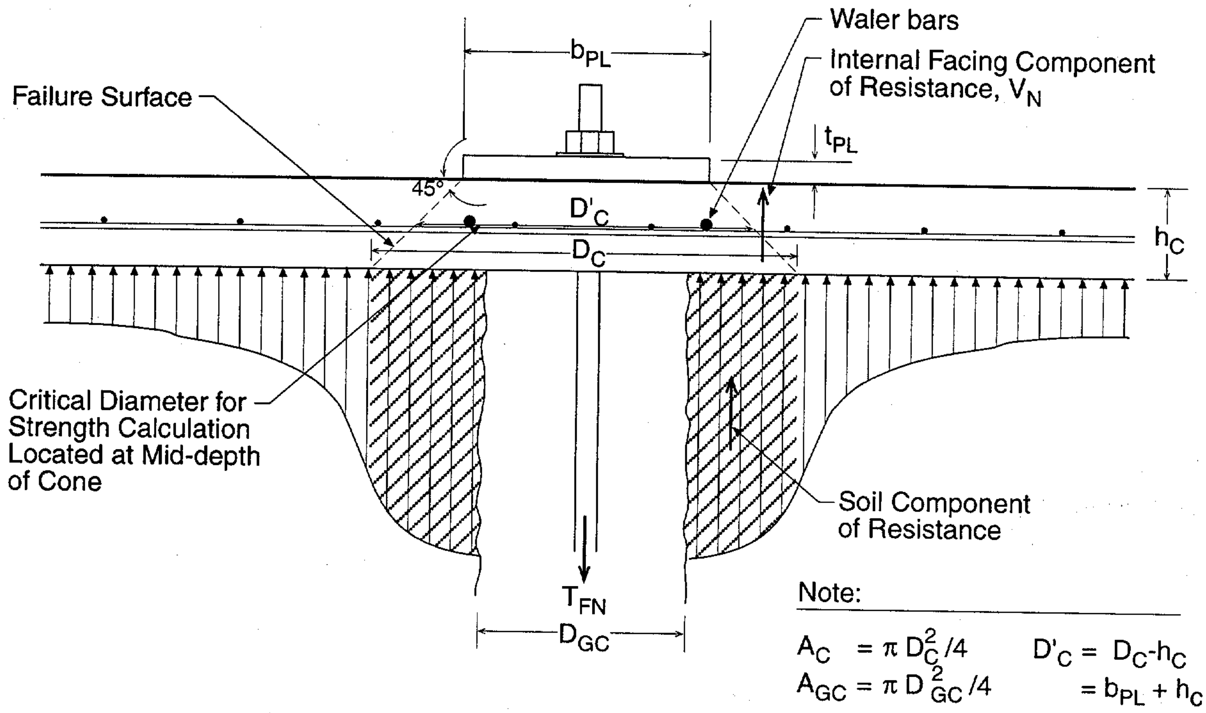
In addition to the strength afforded to the nail head by the internal resistance of the facing, there exists a soil reaction component of resistance. The soil reaction component of the punching shear strength develops because 1) the base diameter of the punching cone can be significantly larger than the nail grout column diameter and 2) as indicated on figure 4.9, the soil pressures that develop in the vicinity of the nail head can be quite large, depending on the flexural stiffness of the facing [36].

(b) Punching Shear of Bearing-Plate Connections

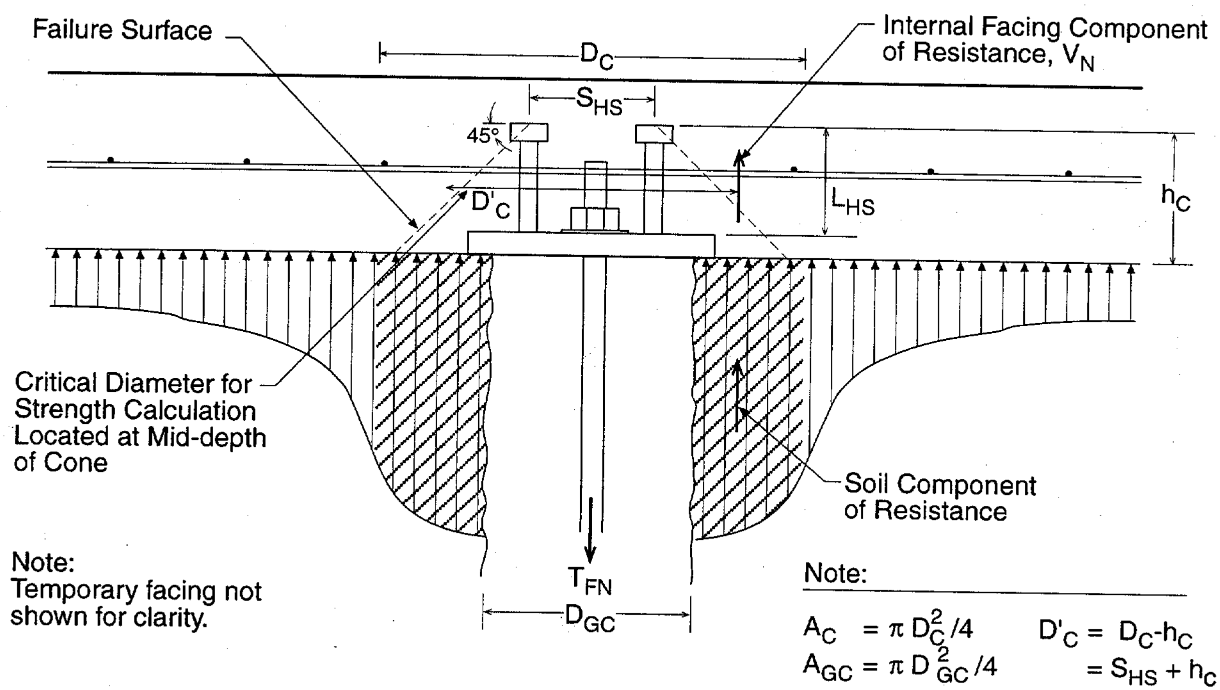
A conservative idealization of the punching shear failure surface geometries observed in recently conducted laboratory tests of bearing plate connections is illustrated on figure 4.9 [36]. Based on this model, the nominal internal punching shear strength of the facing, V_N, is computed by considering a nominal shear stress acting across an effective perimeter area. The perimeter is defined by an effective punching cone diameter D'_C and an effective cone depth h_C as shown on figure 4.9. The complete relationship is indicated below:

$$V_N = 0.33\sqrt{f'_C\,(\mathrm{MPa})}\,(\pi)(D'_C)(h_C) \qquad (4.2)$$

The effective cone diameter is defined by a rupture surface that begins at the edge of the plate and extends toward the soil side of the facing at an angle of 45° from the plane of the facing (figure 4.9). Therefore, at the center of the facing, the effective cone diameter is defined to be



Temporary Bearing-Plate Connection



Permanent Headed-Stud Connection

Figure 4.9 Punching Shear of Nail Head Connections

$D'_C = b_{PL} + h_C$. For a bearing plate type of connection, the cone height h_C is equal to the full facing thickness. This type of failure geometry is most consistent with laboratory tests [36].

As a conservative approximation, the punching shear strength (T_{FN}) of a bearing plate connection can be taken as the internal punching shear strength (V_N) of the facing. If such a conservative approach does not provide sufficient capacity, then the soil reaction contribution to nail head resistance can be included as described below. This will usually result in increases in the calculated punching shear strength of less than 20 percent, for typical shotcrete construction facings.

The soil reaction component of nail head resistance is computed by considering force equilibrium, the failure cone diameter illustrated on figure 4.9, and an increased pressure behind the nail head. The resulting expression for the nominal nail head strength associated with punching shear of the bearing plate connection with the soil reaction contribution included is indicated below [36]:

$$T_{FN} = V_N\left(\frac{1}{1 - C_S(A_C - A_{GC})/(S_V S_H - A_{GC})}\right) \tag{4.3}$$

Equations 4.2 and 4.3 are based on back analysis of case histories, full-scale laboratory testing, calibrated finite-element modeling, experience, and judgment [36]. The pressure factor for punching shear C_S is given in table 4.2 . Equation 4.3 requires that the nail hole diameter be known in order to calculate A_{GC} and the nominal nail head strength. At the design stage, it may be necessary to assume the nail hole size. The nail head strength is relatively insensitive to A_{GC} for the range of C_S values recommended, and hence it will not generally be necessary to revisit the design after the nail hole size has been determined.

(c) Punching Shear of Headed-Stud Connections

The approach for computing the internal punching shear strength of headed-stud connections is similar to that for temporary construction facings. The same nominal shear stress is applied over a slightly different perimeter area. The cone diameter is defined by extension of a line at 45° from the centers of the stud heads (figure 4.9). Furthermore, the effective cone depth h_C is defined from the top of the headed studs as shown on figure 4.9. The resulting expression is the same as equation 4.2, where the effective punching cone diameter $D'_C = S_{HS} + h_C$, where S_{HS} is the stud spacing as indicated on figure 4.9.

As with a bearing-plate connection, the soil reaction component is computed by considering an increased pressure beneath the punching cone, outside the perimeter of the grout column (figure 4.9). The resulting expression for the nominal nail head strength is the same as equation 4.3. As for the bearing plate connection, the punching shear strength of the headed-stud connection can be taken as only the internal punching shear strength of the facing (equation 4.2). Because of the relative stiffness of typical permanent facings with headed stud connections, inclusion of the soil-reaction contribution to the nail head resistance will generally increase the calculated punching shear strength by only a few percent.

For headed-stud connections in which the length of the headed-stud is short in relation to the stud spacing (e.g., on the order of half the stud spacing), the pullout of individual studs may govern the strength. In such cases, the capacity of the individual studs should be evaluated (ref. ACI-349, appendix B, section B.4.2) [37] and the lesser strength used in the design. In addition, it is recommended that the headed stud lengths extend to at least the mid-depth of the permanent facing and that the stud heads be anchored beyond one of the permanent facing reinforcement planes.

4.5.4 Headed-Studs Tensile Strength

In a headed-stud connection system, four headed studs are welded to the steel plate and the entire connection is fully embedded in the wall facing. Because the headed studs resist the nail head force through direct tension, the strength criterion corresponding to ultimate tensile stress for bolts is most applicable, as indicated by the following expression:

$$T_{FN} = 4\, A_{HS}\, F_U \tag{4.4}$$

The headed studs must also be checked for bearing on the concrete beneath the heads. The report of ACI Committee 349 [37] requires that either the computed bearing stress not exceed a prescribed value or that the heads meet two special dimensional requirements. For headed-stud connections that are commonly employed in soil nail wall construction, the stress check can never be met. Therefore, the two geometric criteria must be checked: (1) the cross-sectional area of the stud head must exceed 2-1/2 times the cross-sectional area of the stud body; and (2) the head thickness must exceed 1/2 the difference between the head diameter and the body diameter. The stock sizes of headed shear studs are shown in table 4.2A.

TABLE 4.2A
DIMENSIONS OF STOCK SIZE HEADED STUDS

(1.) Anchor Size - in	(2.) A.W. Length -in (mm)	Head Diameter - in (mm)	Head Thickness -in (mm)
$^1/_4$ x $2^{11}/_{16}$	$2^9/_{16}$ (65)	.500 (12.7)	.187 (4.7)
$^1/_4$ x $4^1/_8$	4 (102)	.500 (12.7)	.187 (4.7)
$^3/_8$ x $4^1/_8$	4 (102)	.750 (19.1)	.281 (7.1)
$^3/_8$ x $6^1/_8$	6 (152)	.750 (19.1)	.281 (7.1)
$^1/_2$ x $2^1/_8$	2 (51)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $3^1/_8$	3 (76)	1.00 (25.4)	.312(7.9)
$^1/_2$ x $4^1/_8$	4 (102)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $5^5/_{16}$	$5^3/_{16}$ (132)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $6^1/_8$	6 (152)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $8^1/_8$	8 (203)	1.00 (25.4)	.312 (7.9)
$^5/_8$ x $2^{11}/_{16}$	$2^1/_2$ (64)	1.250 (31.8)	.312 (7.9)
$^5/_8$ x $6^9/_{16}$	$6^3/_8$ (162)	1.250 (31.8)	.312 (7.9)
$^5/_8$ x $8^3/_{16}$	8 (203)	1.250 (31.8)	.312 (7.9)
$^3/_4$ x $3^3/_{16}$	3 (76)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $3^{11}/_{16}$	$3^1/_2$ (89)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $4^3/_{16}$	4 (102)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $5^3/_{16}$	5 (127)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $6^3/_{16}$	6 (152)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $7^3/_{16}$	7 (178)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $8^3/_{16}$	8 (203)	1.250 (31.8)	.375 (9.5)
$^7/_8$ x $3^{11}/_{16}$	$3^1/_2$ (89)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $4^3/_{16}$	4 (102)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $5^3/_{16}$	5 (127)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $6^3/_{16}$	6 (152)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $7^3/_{16}$	7 (178)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $8^3/_{16}$	8 (203)	1.375 (34.9)	.375 (9.5)

NOTES: (1.) Stock Anchor Sizes
(2.) A.W. - Length overall after welding to plate

4.5.5 Selecting Nominal Nail Head Strengths

Table 4.2B summarizes nominal nail head strengths for facing flexure and facing punching shear failure modes, for common temporary and permanent facing designs.

For the temporary shotcrete construction facing, the flexural failure mode considers standard 100 mm thick facing of shotcrete compressive strength equal to 28 Mpa, with two No. 13 continuous waler bars at each row of nails, and various nail head spacings and sizes of steel mesh reinforcement with and without vertical bearing bars at each nail head location. For the punching shear failure mode of a bearing plate through the shotcrete construction facing, both the internal and total nominal nail head strengths are given for different sizes of bearing plate (nail spacing and drill hole diameter are fixed at typical values as the results are relatively insensitive to these parameters).

For permanent CIP or shotcrete facing, the flexural failure mode considers a standard fixed pattern of facing reinforcement (No. 13 bars at 300 mm spacing each way) and two facing thicknesses of 200 mm and 150 mm that represent the practical minimum facing thickness that can be constructed for CIP facings and permanent shotcrete facings respectively. For the punching shear failure mode of a headed stud wedge pulling out of the permanent facing, both the internal and total nominal nail head strengths are given for different depths of stud embedment (determined by the bearing plate thickness and overall stud length) and different representative stud spacings.

TABLE 4.2B
NOMINAL NAIL HEAD STRENGTH

Temporary Shotcrete Construction Facing

Facing Flexure:

Facing Thickness:	100 mm
Steel Yield:	420 MPa
Shotcrete Comp. Strength:	28 MPa
Walers:	2 X No. 13

Nail Spacing (m)	**WW Mesh**	**Vert. Bearing Bars**	**T_{FN} (kN)**
1.25 X 1.25	152X152 MW13XMW13	-	58
		2 X No. 13	122
	152X152 MW18XMW18	-	81
		2 X No. 13	145
	152X152 MW25XMW25	-	111
		2 X No. 13	166*
	102X102 MW9XMW9	-	59
		2 X No. 13	124
	102X102 MW13XMW13	-	86
		2 X No. 13	149
	102X102 MW18XMW18	-	119
		2 X No. 13	170*
1.5 X 1.5	152X152 MW13XMW13	-	58
		2 X No. 13	112
	152X152 MW18XMW18	-	81
		2 X No. 13	135
	152X152 MW25XMW25	-	111
		2 X No. 13	163
	102X102 MW9XMW9	-	59
		2 X No. 13	113
	102X102 MW13XMW13	-	86
		2 X No. 13	139
	102X102 MW18XMW18	-	119
		2 X No. 13	170*
1.75 X 1.75	152X152 MW13XMW13	-	58
		2 X No. 13	105
	152X152 MW18XMW18	-	81
		2 X No. 13	127
	152X152 MW25XMW25	-	111
		2 X No. 13	156
	102X102 MW9XMW9	-	59
		2 X No. 13	106
	102X102 MW13XMW13	-	86
		2 X No. 13	132
	102X102 MW18XMW18	-	119
		2 X No. 13	164

* Calculated capacity limited by maximum reinforcement ratio (based on gross area) of 0.35%.

TABLE 4.2B (Cont'd)
NOMINAL NAIL HEAD STRENGTH

Facing Punching Shear:

Facing Thickness: 100 mm
Shotcrete Comp. Strength: 28 MPa
Drill Hole Diameter: 200 mm
Nail Spacing: 1.5 m X 1.5 m

Bearing Plate Width (mm)	V_N(kN)	T_{FN} (kN)
200	165	184
225	178	204
250	192	224

<u>Permanent Facing</u>

Facing Flexure:

Steel Yield: 420 MPa
Shotcrete Comp. Strength: 28 MPa
Reinforcement: No. 13 bars @ 300 mm
Nail Pattern: Vertical Spacing = Horizontal Spacing

Facing Thickness (mm)	T_{FN} (kN)
150	206
200	278

Facing Punching Shear:

Shotcrete Comp. Strength: 28 MPa
Drill Hole Diameter: 200 mm
Nail Spacing: 1.5 m X 1.5 m

Pl. Thick.+Stud Lgth. (h_C) (mm)	Stud Spacing (S_{HS}) (mm)	V_N (kN)	T_{FN} (kN)
150	80	189	197
	105	210	219
	130	230	243
145	80	179	185
	105	199	207
	130	219	230
125	80	141	144
	105	158	163
	130	175	182
120	80	132	135
	105	148	152
	130	165	170
100	80	99	100
	105	112	115
	130	126	129
95	80	91	92
	105	104	106
	130	117	120

4.6 Real World Design Process

More than one individual may be responsible for various aspects of a soil nail wall design. For instance, a designer with geotechnical expertise may be responsible for design of the soil nail layout, while a structural engineer may perform the facing design checks. The actual division of responsibility and labor may vary from agency to agency. It is recommended that the division of responsibility be agreed to beforehand to ensure essential design checks are not neglected. Below is a sequence of how a soil nail wall design might progress from start to finish for a typical State Transportation Agency (functions not related to the actual performance of the wall as a structure, such as right-of-way checks, are omitted):

- Project Office contacts geotechnical branch with need for a cut wall design.
- Geotechnical engineer reviews wall site data, collects subsurface design data (borings, lab testing, et.) and selects one or more critical wall design cross-sections.
- Geotechnical engineer develops a scaled drawing of design cross-section with all geometries and external loadings identified.
- Geotechnical engineer selects trial values for allowable nail head loads (appendix F) and reasonable estimated soil nail pullout resistance values for the site ground conditions.
- Geotechnical engineer performs limit equilibrium analyses to optimize the soil nail layout (horizontal and vertical spacing, lengths and bar sizes) and performs external stability checks.
- Geotechnical engineer lays out nails on wall profile according to design section criteria.
- Geotechnical engineer prepares design report to communicate nail layouts, wall facing design assumptions, drainage and specification requirements to structural engineer.

Geotechnical engineer transmits report and nail layout drawings to structural engineer.

- Structural engineer checks punching shear, flexure and shear stud strength assumptions for allowable nail head loads.
- Structural engineer checks cantilever section designs.
- Structural engineer checks shotcrete and/or cast-in-place facing reinforcement requirements.
- Structural engineer prepares final wall plan and detail sheets.
- Structural or specification engineer prepares special provisions.

Geotechnical, structural and project engineers review PS&E package.

Application of the above process is set forth in the following sections in a detailed step-by-step design procedure. The step-by-step design procedure is also summarized in flow chart form on figure 4.10. For the above described design process, steps 1-8 shown on figure 4.10 would typically be performed by the geotechnical engineer and steps 9-11 performed by the structural engineer.

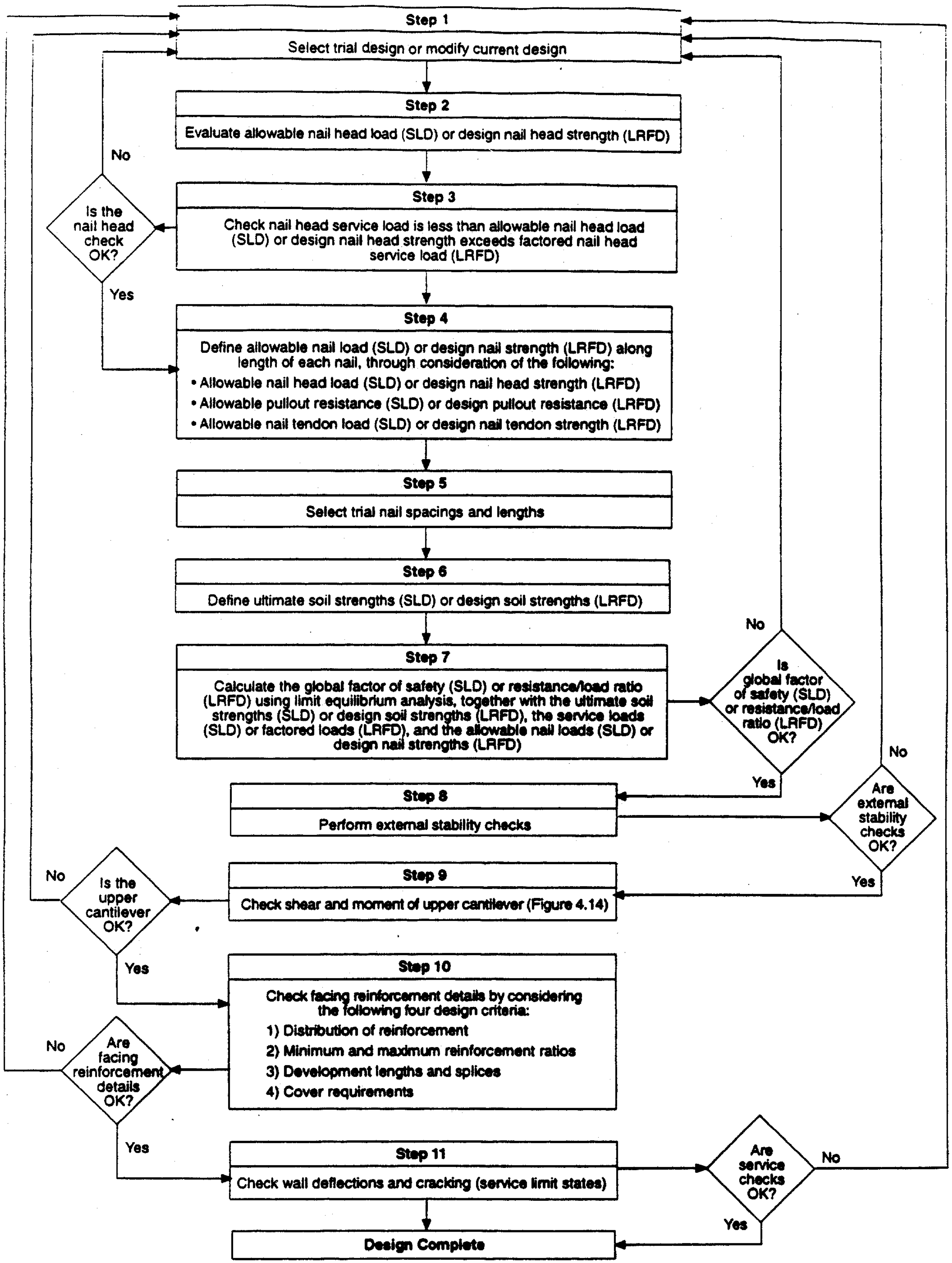


Figure 4.10 Recommended Design Procedure

4.7 Soil Nail Wall Design

The overall soil nail wall design approach is summarized in the flow chart presented on figure 4.10. The design procedure is presented separately for SLD and LRFD in sections 4.7.1 and 4.7.2, respectively. There is repetitive language between sections 4.7.1 and 4.7.2 so that each can be used on a stand-alone basis without the need for cross referencing.

4.7.1 Service Load Design (SLD)

Step 1 - Set Up Critical Design Cross-Section(s) and Select a Trial Design

Select a trial design for the design geometry and loading conditions. The design cross-section should show ultimate soil strength properties for the various subsurface layers and design water table location (should be below wall base) and all applied service loadings such as self weight of the soil and surface surcharges. The load combinations to be considered are summarized in table 4.3 [30]. A proposed trial design nail pattern, including nail lengths, tendon sizes, and trial vertical and horizontal nail spacings, should be developed from section 4.2 and shown superimposed on the design section.

TABLE 4.3
LOAD COMBINATIONS IN AASHTO SPECIFICATIONS (AASHTO, 15th Ed., 1992)

Group	D	L	E	B	RST	EQ	%
I	1	1	1	1	0	0	100
IV	1	1	1	1	1	0	125
VII	1	0	1	1	0	1	133

Notes:
D = dead load.
L = live load.
E = earth pressure.
B = buoyancy.
RST = rib shortening, shrinkage, temperature.
EQ = earthquake.

The loads and load combinations shown in this table are intended to illustrate AASHTO requirements considered most relevant to soil nail wall design. Specific designs may require consideration of additional loads and load combinations.

Step 2 - Compute the Allowable Nail Head Load

Evaluate the allowable nail head load for the trial construction facing and connector design using the following procedure:

a) Determine the nominal nail head strength for each potential failure mode of the facing and connection system using the methods presented in section 4.5 and in appendix F. **As a design aid, the nominal nail head strengths for several soil nail wall typical facing thickness, facing reinforcement and bearing plate connection combinations are tabulated in table 4.2B.**

b) For each possible failure mode in table 4.4, determine the allowable nail head load as a fraction of the corresponding nominal nail head strength. The first column of nail head strength factors of table 4.4 applies to the Load Combination Group I (table 4.3). For other Load Combination Groups, these Group I nail head strength factors of table 4.4 are **increased** in accordance with the percentage factors of the final column of table 4.3. For soil nail wall applications, Load Combination Groups I, IV and VII are likely to control the design most of the time. The corresponding nail head strength factors for Load Combination Groups IV and VII are therefore also shown in table 4.4. The allowable nail head load is the *lowest* calculated value for the various failure modes.

TABLE 4.4
NAIL HEAD STRENGTH FACTORS - SLD

Failure Mode	Nail Head Strength Factor (Group I) α_F	Nail Head Strength Factor (Group IV)	Nail Head Strength Factor (Group VII) (Seismic)
Facing Flexure	0.67[a]	1.25(0.67)= 0.83	1.33(0.67)=0.89
Facing Punching Shear	0.67[a]	1.25(0.67)=0.83	1.33(0.67)=0.89
Headed-Stud Tensile Fracture			
ASTM A307 Bolt Material	0.50[a]	1.25(0.50)=0.63	1.33(0.50)=0.67
ASTM A325 Bolt Material	0.59[a]	1.25(0.59)=0.74	1.33(0.59)=0.78

[a] Obtained by dividing the corresponding AASHTO LRFD resistance factors (table 4.7) by a load factor of 1.35, which is the load factor for the usually dominant self weight load (table 4.6).

Step 3 - Minimum Allowable Nail Head Service Load Check

Perform a minimum allowable nail head load check for the trial facing design. This empirical check is performed to ensure that the computed allowable nail head load exceeds the estimated nail head service load that may actually be developed as a result of soil-structure interaction. With reference to figure 2.7, the nail head service load actually developed can be estimated by using the following empirical equation:

$$t_F = F_F K_A \gamma H S_H S_V \qquad (4.6)$$

Unless the designer has site specific monitoring information from walls constructed in similar soils, it is recommended that a value of the nail head service factor, F_F, equal to 0.50 be adopted for design purposes. (See section 2.4.5).

For simple configurations (i.e., uniform soil conditions, no surcharge, etc.) the active earth pressure coefficient K_A can be determined directly from published equations and charts, neglecting the cohesive component of soil strength in accordance with the discussion in sections 2.4.4 and 2.4.5. For more complex configurations that are not tabulated in the published literature (e.g., variable soil layers, complex wall geometries and surcharge load distributions), the nail head service load can be estimated from

$$t_F = 2 F_F P_A S_H S_V / H \qquad (4.7)$$

The active load P_A can typically be determined using a Coulomb-type slip surface (i.e., slope stability) calculation. If the computed allowable nail head load is less than the empirically estimated nail head service load, the trial facing/connector design should be modified and step 2 repeated.

Note: This check is not applied for seismic loading conditions. Since 1)There is no service load data for seismic conditions 2)There is no evidence of poor permanent facing performance under such loading.

Step 4 - Define the Allowable Nail Load Support Diagrams

Define the allowable nail load for each of the nails as a function of location along the nail length. As shown on figure 4.3, the allowable nail load will vary along the length of the nail and will depend on the allowable nail head load, the allowable nail tendon load, and the allowable nail-ground pullout resistance.

The allowable nail head load is determined as discussed in step 2.

The allowable nail tendon load is taken as the nail tendon strength factor times the nail tendon yield strength, as shown in table 4.5, in accordance with the Standard Specifications for Highway Bridges, 15th Edition [30]. It is recommended that the minimum bar size used for soil nails be No. 19 (Soft Metric designation) - corresponding to a No. 6 standard bar size. However, nail sizes smaller than No. 25 (metric) may cause installation problems for moderate to long nail lengths due to their low stiffness. Bar size designations are provided in Table 4.8A and in appendix F.

The allowable nail pullout resistance will determine the rate at which the allowable nail load can change along the length of the nail and is taken as the nail pullout resistance factor (see table 4.5) times the ultimate ground-grout pullout resistance. The ultimate pullout resistance may be determined from local experience, published data or field testing, and is often expressed in terms of a force per unit length of nail. See section 3.2 for guideline published pullout resistance data.

The first column of the nail strength factors of table 4.5 apply to Load Combination Group I (table 4.3). For other Load Combination Groups, these Group I nail strength factors of table 4.5 are **increased** in accordance with the percentage factors of the final column of table 4.3. As noted previously, for soil nail wall applications, Load Combination Groups I, IV and VII are likely to control the design in most cases. Therefore, the corresponding nail strength factors for Load Combination Groups IV and VII are also shown in table 4.5.

TABLE 4.5
STRENGTH FACTORS AND FACTORS OF SAFETY - SLD

Element	Strength Factor (Group I) α	Strength Factor (Group IV)	Strength Factor (Group VII) (Seismic)
Nail Head Strength	α_F = Table 4.4	see Table 4.4	see Table 4.4
Nail Tendon Tensile Strength	$\alpha_N = 0.55$	1.25(0.55)=0.69	1.33(0.55)= 0.73
Ground-Grout Pullout Resistance	$\alpha_Q = 0.50$	1.25(0.50)=0.63	1.33(0.50)=0.67
Soil	F=1.35 (1.50*)	1.08 (1.20*)	1.01 (1.13*)
Soil-Temporary Construction Condition†	F=1.20 (1.35*)	NA	NA

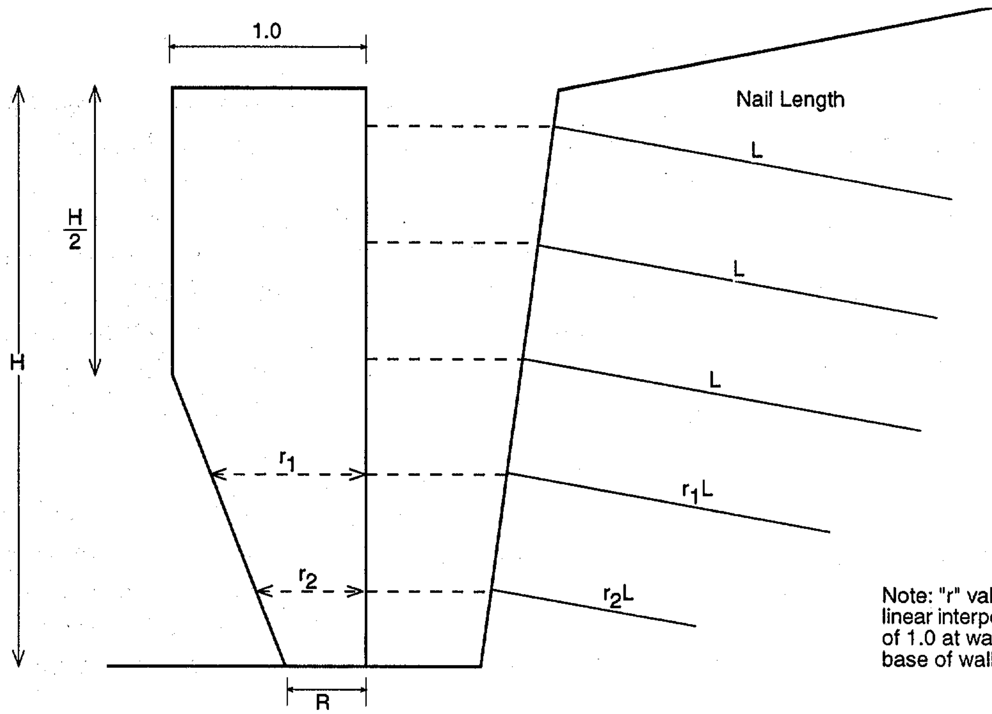
Notes:
Allowable Nail Head Load (T_F) = α_F (Nominal Nail Head Strength) = $\alpha_F\ T_{FN}$
Allowable Nail Tendon Load (T_N) = α_N (Tendon Yield Strength) = $\alpha_N\ T_{NN}$
Allowable Pullout Resistance (Q) = α_Q (Ultimate Pullout Resistance) = $\alpha_Q\ Q_U$
Minimum Required Global Soil Factor of Safety "F" (Group I) = 1.35 (= 1.50 for critical structures).
Minimum Required Global Soil Factor of Safety "F" (Group IV) = 1.35/1.25 = 1.08 (= 1.20 for critical structures.
Minimum Required Global Soil Factor of Safety "F" (Group VII) = 1.35/1.33 = 1.01 (= 1.13 for critical structures).
Minimum Required Global Soil Factor of Safety "F" - Temporary Construction Condition = 1.20 (= 1.35 for critical structures).
* Soil Factors of Safety for Critical Structures.
† Refers to temporary condition existing following cut excavation but before nail installation. Does not refer to "temporary" versus "permanent" wall.

Step 5 - Select Trial Nail Spacings and Lengths

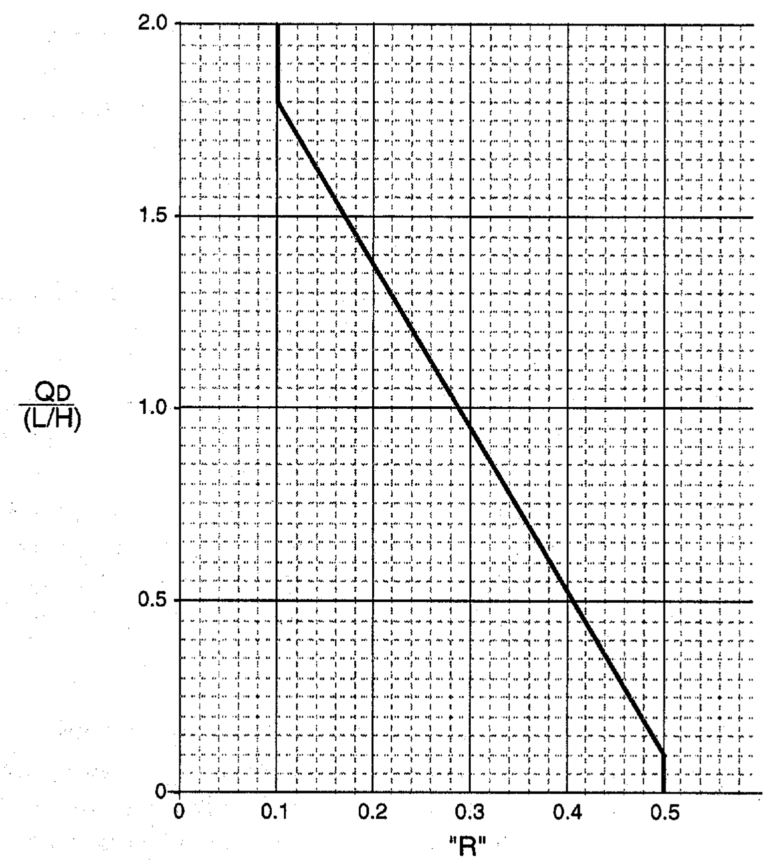
As discussed in section 4.3.2 and as shown on figure 4.7, satisfaction of the strength limit state requirements will not of itself ensure an appropriate design. Additional constraints are required to provide for an appropriate nail layout. The following empirical constraints on the design analysis nail length pattern are therefore recommended for use when performing the limiting equilibrium design calculations:

(a) Nails with heads located in the upper half of the wall height should be of uniform length.

(b) Nails with heads located in the lower half of the wall height should be considered to have a reduced length in accordance with the recommendations given on figure 4.11.



Note: "r" values determined by linear interpolation between a value of 1.0 at wall midheight and "R" at base of wall.



L = Maximum Nail Length

H = Wall Height

Q_D = Dimensionless Pullout Resistance

= α_Q Qu/($\gamma S_H S_v$) (SLD)

= Φ_Q Qu/($\Gamma_W \gamma S_H S_v$) (LRFD)

where

α_Q = pullout resistance strength factor (SLD)

Φ_Q = pullout resistance factor (LRFD)

Qu = ultimate pullout resistance

γ = unit weight

S_H = horizontal nail spacing

S_v = vertical nail spacing

Γ_W = soil weight load factor (LRFD)

Figure 4.11 Nail Length Distribution Assumed for Design

The purpose of these recommendations is to ensure that adequate nail reinforcement (length and strength) is installed in the upper part of the wall. Performance monitoring of several instrumented soil nail walls, in which both nail loads and wall movements were measured, has demonstrated that the top-down method of construction of soil nail walls generally results in the nails in the upper part of the wall being more significant than the nails in the lower part of the wall in developing resisting loads and controlling displacements. If the strength limit state calculation overstates the contribution from the lower nails, then this can have the effect of indicating shorter nails and/or smaller tendon sizes in the upper part of the wall, which is considered undesirable since this could result in poorer in-service performance.

It should be noted that the above recommendation for the nail length pattern to be assumed for design calculation purposes does not imply that the installed nail pattern must correspond exactly to the design nail pattern. It is most common to install nails of uniform length, often to simplify construction. Provided the appropriate external stability checks are performed (see step 8), it may be possible to install shorter nails in the lower part of the wall.

Step 6 - Define the Ultimate Soil Strengths

Define the ultimate soil strengths for analysis. Methods for determining appropriate soil strengths for design are discussed in section 3.1. Reasonably accurate soil strength characterization is an important part of the design process and should be performed only by qualified experienced geotechnical personnel.

Step 7 - Calculate the Factor of Safety

Calculate the limiting equilibrium factor of safety for each potential slip surface, taking into account the additional stabilizing forces provided by the trial pattern of nails. For non-critical structures, a minimum calculated global soil factor of safety of 1.35 (see table 4.5) is recommended, taking account of the allowable nail loads and the ultimate soil strengths. The global soil factor of safety of 1.35 applies to Load Combination Group I of table 4.3. For other Load Combination Groups, the global soil factor of safety of table 4.5 is **decreased** in accordance with the percentage factors of the final column of table 4.3. Table 4.5 shows that the recommended global soil factors of safety for Load Combination Groups IV and VII (seismic loading) are 1.08 and 1.01, respectively. Table 4.5 also indicates that these minimum required factors of safety should be increased for soil nail walls supporting critical structures (e.g. bridge abutments).

It will also be necessary to check the stability of the soil nail wall during its construction. This evaluation should consider temporary construction conditions corresponding to the situation in which the next lift has been excavated, prior to the installation of the nails for that lift. In these circumstances, because of the temporary nature of such conditions, it is recommended that the nail strength factors applied be the same as those shown in table 4.5, but that the required global soil factor of safety be reduced to a value of 1.2 (1.35 for critical structures), as shown in table 4.5. In general, for most applications and typical construction conditions, construction stability requirements will not control the design. However, in certain circumstances such as significant existing surcharge loadings adjacent to the wall during construction, construction conditions may be more critical. This issue is further discussed in section 4.10.2, 4.10.3, and figure 4.18.

Step 8 - External Stability Check

Perform stability analyses for potential "external" failure modes. As noted in section 4.2.4, the potential external failure modes that require consideration with the slip surface method include overall slope failure external to the nailed mass, and foundation bearing capacity failure beneath the laterally loaded soil nail "gravity" wall. The methods of analysis for these failure modes are equivalent to those used for any gravity retaining structure.

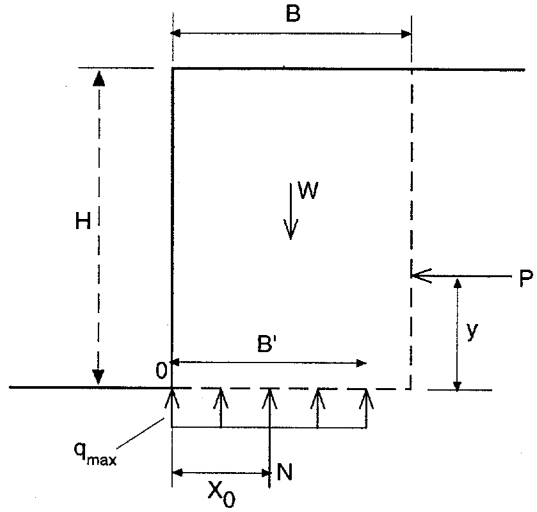
The required factor of safety for overall slope stability is 1.3 (or 1.5 where abutments are supported above the soil nail wall) for Group I loading (ref. section 5.2.2.3, AASHTO, 15th Edition, [30]). When performing overall slope stability checks, consideration should be given to potential slip surfaces that pass beneath the base of the wall and exit downslope or in front of the wall toe. This condition will be more critical if the ground water table is located close to the base of the wall. For this reason, it is recommended that nail lengths should not be shortened in the lower part of the wall (reference step 5, above) unless stability analyses confirm that more deep-seated failure modes will not control.

Although the bearing capacity seldom controls the design, a rough bearing capacity check should be made to insure global stability. In general bearing capacity should be checked in situations where cohesive soils exist within a depth equal to the width of the soil nailed mass. The equations shown in section 4.2.4 should be used to determine if the potential exists for bearing capacity failure or basal heave. If the safety factor from these analyses is less than 2.5, a more rigorous bearing capacity analysis as shown in figure 4.12 should be performed. It is recommended that the factor of safety against bearing capacity failure for Load Combination Group I should have a minimum value of 2.5. For other Load Combination Groups, the required minimum factor of safety for the rigorous analysis should be **decreased** in accordance with the percentage factors of the final column of table 4.3 (i.e., factor of safety of 2.5/1.25 = 2.0 for Load Group IV and 2.5/1.33 = 1.9 for Load Group VII).

Step 9- Check the Upper Cantilever

The upper cantilever section of a soil nail wall facing, above the top row of nails, will be subjected to earth pressures that arise from the self weight of the adjacent soil and any surface surcharge loadings or inertial forces acting upon the adjacent soil. The magnitude of these earth pressures will depend not only on the strength of the soil but also on factors such as the method of fill placement (if any) behind the cantilever. For no fill placement and associated compaction-induced stresses behind the upper cantilever following its construction, an active earth pressure coefficient can be assumed for the upper cantilever portion of the wall. Because the upper cantilever is not able to redistribute load by soil arching to adjacent spans, as can the remainder of the wall facing below the top nail row, the strength limit state of the cantilever must be checked for moment and shear at its base, as described on figure 4.13.

SLD



N – establish from vertical equilibrium (e.g. $N = W$)

X_0 – establish from moment equilibrium
[e.g. $X_0 = \Sigma M/N$
$= (WB/2 - Py)/N$]

Bearing Stability

$$q_{max} = \frac{N}{B'} \leq \frac{q_{ult}}{F}$$

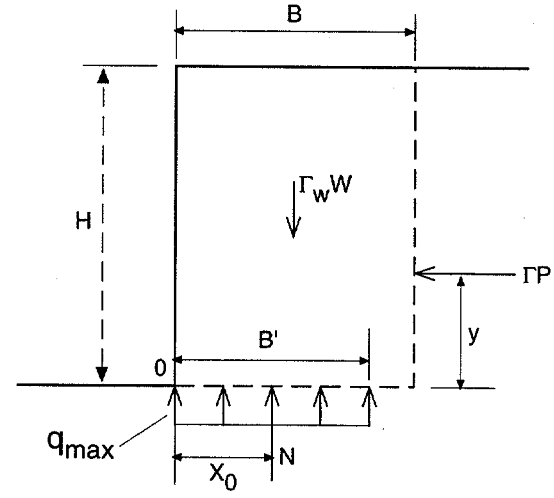
where $B' = 2X_0$
F = Factor of Safety

q_{ult} = ultimate bearing capacity

Eccentricity Check for Overturning

$$e = B/2 - X_0 \leq B/6$$

LRFD



N – establish from vertical equilibrium using factored loads (e.g. $N = \Gamma_w W$)

X_0 – establish from moment equilibrium using factored loads
[e.g. $X_0 = \Sigma M/N$
$= (\Gamma_w WB/2 - \Gamma Py)/N$]

Bearing Stability

$$q_{max} = \frac{N}{B'} \leq \Phi\, q_{ult}$$

where $B' = 2X_0$
Φ = Bearing Resistance Factor

q_{ult} = ultimate bearing capacity

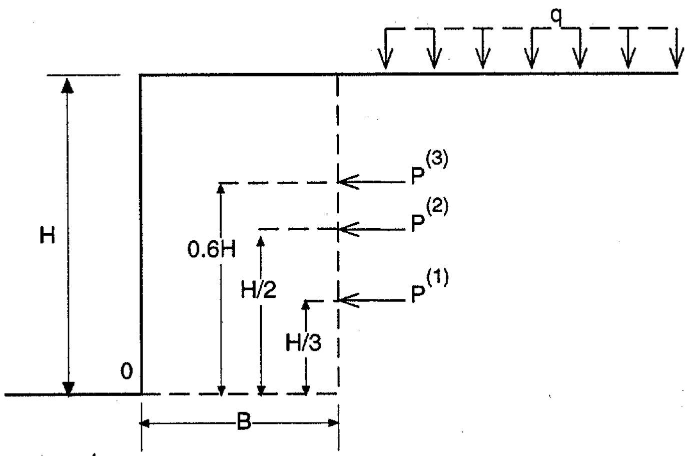
Eccentricity Check for Overturning

$$e = B/2 - X_0 \leq B/4$$

Legend
W = Weight of Reinforced Zone
P = Earth Pressure Load
y = Point of Application of P
B = Width of Base of Soil Nailed Block

Figure 4.12 Bearing Capacity Methodology for Soil Subgrade (continued on next page)

Magnitude of Earth Pressure Load P and Point of Application 'y'



$P^{(1)}$ = Static earth pressure load due to self weight

$P^{(2)}$ = Static earth pressure load due to surcharge

$P^{(3)}$ = Dynamic earth pressure load due to self weight

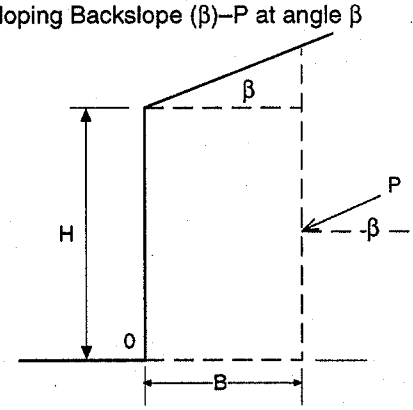
Notes:

(1) Earth Load Components determined using methods of Coulomb and Mononobe-Okabe

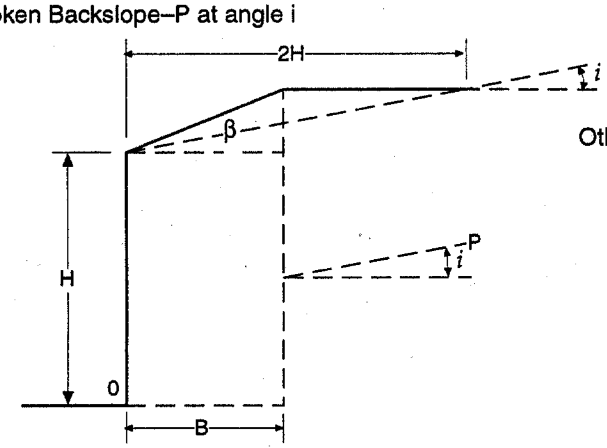
Inclination of Earth Pressure Load P

a. Horizontal Backslope–P horizontal

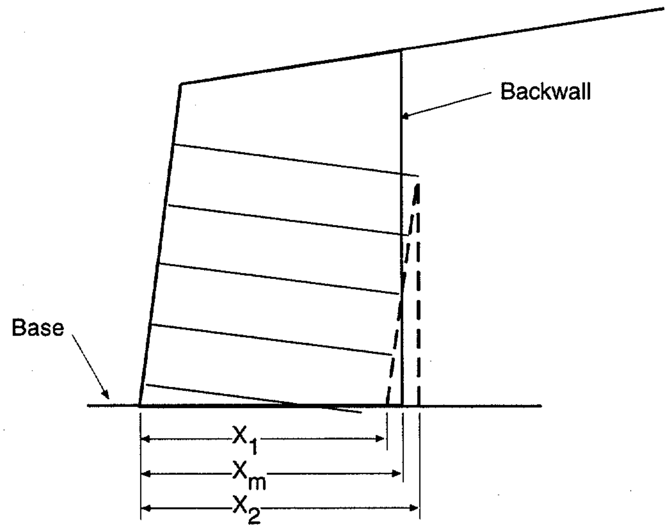
b. Sloping Backslope (β)–P at angle β



c. Broken Backslope–P at angle i



LRFD



$X_m = (X_1 + X_2)/2$

The reinforced 'gravity' wall for external stability analysis can be defined by a horizontal base and a vertical backwall, as shown.

Ultimate Bearing Capacity 'q_{ult}'

Per AASHTO (15th Edition, 1992) - Section 4.4.7

$$q_{ult} = c_u N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

N_c, $N\gamma$ – bearing capacity factors

s_c, s_γ – shape factors

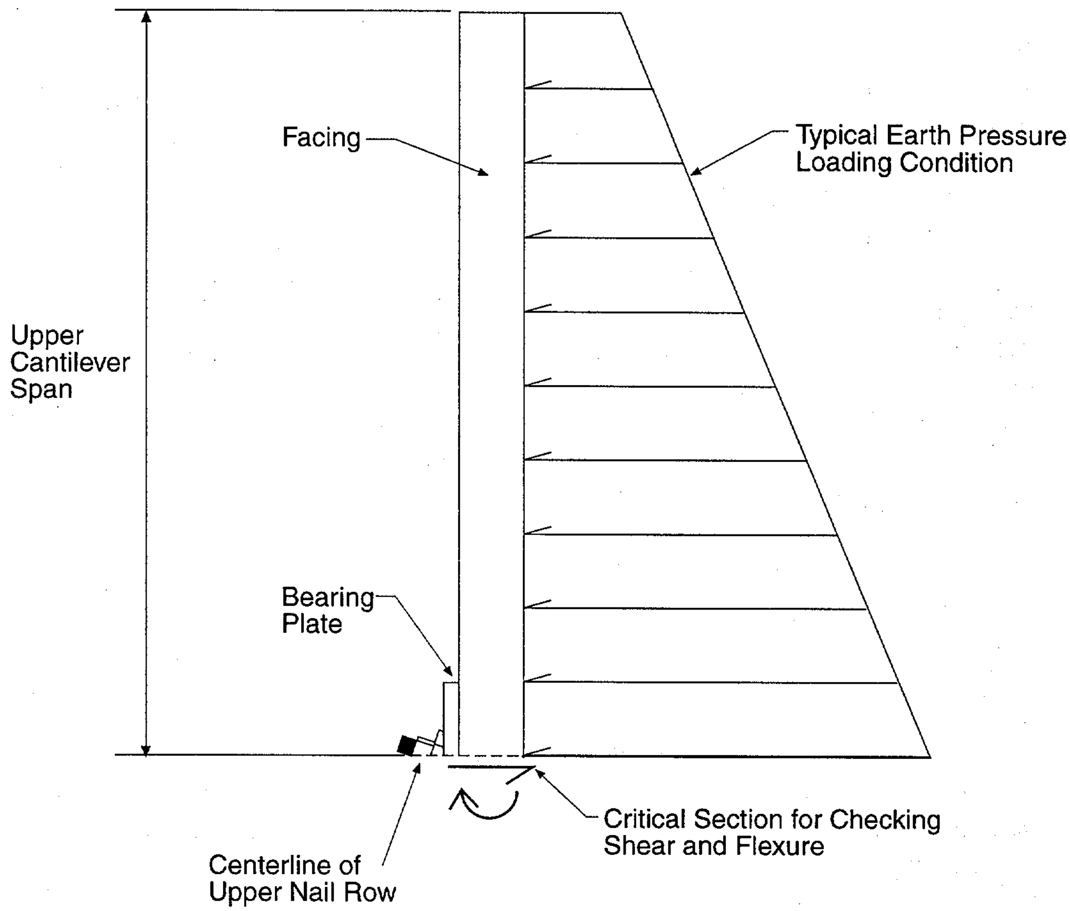
i_c, i_γ – load inclination factors

γ – unit weight of foundation soils

Where the groundwater table is located within depth B to 1.5B of the wall base, use appropriate reduction factor to account for buoyancy effects per AASHTO (15th Ed.) 4.4.7.1.1.6 or AASHTO (LRFD, 1st Ed.) 10.6.3.1.

Other accepted approaches for defining q_{ult} may be used.

Figure 4.12(cont.) Bearing capacity Methodology for Soil Subgrade



Design Checks (SLD)

1) At the critical section indicated above, the service shear (computed from force equilibrium of the cantilever) should not exceed the allowable shear (computed as the nominal shear strength of the cantilever based on the currently adopted AASHTO bridge specification and interims, multiplied by the nail head strength factor for facing punching shear from Table 4.4).

2) At the critical section indicated above, the service moment (computed from moment equilibrium of the cantilever) should not exceed the allowable moment (computed as the nominal flexural strength of the cantilever based on the currently adopted AASHTO bridge specification and interims, multiplied by the facing flexure strength factor from Table 4.4).

Design Checks (LRFD):

1) At the critical section indicated above, the factored shear (computed from force equilibrium of the cantilever) should not exceed the design shear strength (computed as the nominal shear strength of the cantilever based on the currently adopted AASHTO bridge specification and interims, multiplied by the nail head resistance factor for facing punching shear from Table 4.7).

2) At the critical section indicated above, the factored moment (computed from moment equilibrium of the cantilever) should not exceed the design moment strength (computed as the nominal flexural strength of the cantilever based on the currently adopted AASHTO bridge specification and interims, multiplied by the facing flexure resistance from Table 4.7).

Figure 4.13 Upper Cantilever Design Checks

If heavy compaction occurs in close proximity to the upper cantilever section of the wall, additional lateral earth pressures will be induced on the wall. Therefore, if it is necessary to place compacted fill in this area, light compaction equipment should be used.

In the horizontal span direction, cantilevered spans will exist at the ends of the soil nail wall (end spans) and at locations of vertical expansion joints for the permanent facing. For expansion joints, it is common practice to keep the same nail pattern and uniform horizontal nail spacing as in the remainder of the wall segment, and locate the expansion joint directly between two columns of nails. For cantilevered end spans, normal design and construction practice is such that the cantilever span is generally in the range of one-third to two-thirds of the average nail spacing. If these criteria are adhered to, no formal additional design of the facing is required at these locations. These construction practices have consistently resulted in good performance of horizontal cantilever spans. The main reason for this is that the geometry limitations described above allow the soil behind the cantilever span to redistribute pressures to adjacent interior spans with only minor increases in deformations in that region of the wall.

For the cantilever at the bottom of the wall, the method of construction tends to result in minimal to zero loads on this cantilever section during construction. There is also the potential for any long-term loading at this location to arch across this portion of the facing to the base of the excavation. It is therefore recommended that no formal design of the facing be required for the bottom cantilever. It is also recommended, however, that the distance between the base of the wall and the bottom row of nails not exceed two-thirds of the average vertical nail spacing.

Step 10 - Check the Facing Reinforcement Details

Check waler reinforcement requirements, minimum reinforcement ratios, minimum cover requirements, and reinforcement development and splices, as described below.

Waler Reinforcement

For temporary shotcrete construction facings, it is common practice and recommended that a minimum of two No. 13 (Soft Metric designation) deformed horizontal waler bars be placed continuously along each nail row and located behind the face bearing plate at each nail head (i.e., between the face bearing plate and the back of the shotcrete facing). The main purpose of the waler reinforcement is to provide additional ductility in the event of a punching shear failure, through dowel action of the waler bars contained within the punching cone.

In permanent facings placed over the shotcrete construction facing, no waler bars are required due to the reduced pressure factors discussed in section 4.5.

Minimum Reinforcement Ratios

Minimum reinforcement ratios are specified in sections 8.17 and 8.20 of the Standard Specifications for Highway Bridges, 15th Edition [30]. These provisions are intended to provide that flexural failure mechanisms remain ductile (section 8.17) and to provide a minimum amount of resistance to shrinkage and temperature distress (section 8.20). However, soil nail wall facings are inherently ductile even at extremely low reinforcement ratios. Therefore, the provisions in section 8.17 should not be applied to soil nail walls.

In addition, for temporary shotcrete facings, shrinkage and temperature cracking are not significant concerns and the minimum steel ratio requirements of section 8.20 may be waived for the temporary facing.

In summary, only the shrinkage and temperature reinforcement requirements of section 8.20 must be checked, and only for permanent facing systems.

Minimum Cover Requirements

Concrete or shotcrete cover is necessary to provide bond resistance for the reinforcing steel and corrosion protection to the reinforcing steel, the bearing plate and headed studs, as well as any non-encapsulated or non-epoxy coated nail steel. In permanent applications, corrosion protection is a vital component of the design. These provisions are contained in section 8.22 of the Standard Specifications for Highway Bridges, 15th Edition [30]. On the front side of permanent facings exposed to the weather, the minimum cover required is 50 mm. For permanent shotcrete facings, on the side of the facing exposed to the soil, the minimum cover required is 75 mm. For permanent CIP facings, on the side of the facing cast against the temporary shotcrete, the minimum concrete cover required is 38 mm for the reinforcing steel (i.e., minimum distance from the CIP concrete reinforcing steel to the concrete-shotcrete interface), and the minimum shotcrete/concrete cover required from the side of the shotcrete facings exposed to the soil is 75 mm for all steel. For the temporary shotcrete construction facing, corrosion protection is not a concern, and it is adequate to place the reinforcing steel near the center of the facing.

Development and Splices of Reinforcement

Check that the splices and cutoff locations of all mesh reinforcement, deformed reinforcing bars, horizontal waler bars, and vertical bearing bars (if used) are sufficient to develop the yield stress of the reinforcement at all locations at which it is needed. All splices and development lengths shall be proportioned in accordance with the Standard Specifications for Highway Bridges, 15th Edition (sections 8.24, 8.30 and 8.32) [30].

Step 11 - Serviceability Checks

Check the wall function as related to excess deformation and cracking (i.e., check the service limit states). The following issues should be considered:

(a) Service Deflections and Crack Widths of the Facing

In accordance with the provisions for two-way slabs of the ACI Building Code Requirements for Reinforced Concrete [38] (note that two-way slabs are not addressed in AASHTO), crack widths in the soil nail wall facing are not checked. However, ACI 318-95 [38] does provide slab deflection criteria through minimum span-to-depth ratios or deflection-to-span ratios that must be met. Because the span to depth ratios for both temporary and permanent soil nail wall facings never exceed about 20, the structural deflections that occur at service load levels are insignificant and therefore not an issue.

The upper cantilevers of permanent soil nail walls are essentially one-way cantilevered slabs and may have larger effective span-to-depth ratios than interior two-way spans. Therefore, the service crack widths (steel stresses) must be checked in the same manner as for the stem of a conventional cantilever retaining wall. The provisions of section 8.16.8.4 of the Standard Specifications for Highway Bridges, 15th Edition [30] are used to check the crack widths. For most temporary facings (whether as part of a temporary shoring system or as the construction facing of a permanent wall system), serviceability requirements are not imposed because the deflections do not pose any significant aesthetic or durability concerns.

(b) Overall Displacements Associated With Wall Construction

Section 2.4.6 discusses the displacement magnitudes and patterns typically associated with the construction of soil nail walls in various ground types. Since the serviceability of the wall must be assessed in terms of potential impacts on adjacent structures and facilities, the data presented in section 2.4.6 can be used to make these determinations. Of course, an assessment of displacements associated with wall construction should be made at the start of the project to determine the suitability of nailing.

(c) Facing Vertical Expansion and Contraction Joints

Vertical joints are not required in the temporary shotcrete construction facing.

Per AASHTO section 5.5.6.5 [30], contraction joints spaced at intervals not exceeding approximately 10 m and expansion joints spaced at intervals not exceeding approximately 30 m, as required for conventional concrete retaining walls, can be used in permanent CIP or permanent shotcrete final finish facings. Typical joint details are shown on the appendix A Example Plans.

Seismic Design

For seismic loading conditions, the structural strength factors and the soil factors of safety should be modified by the percentages given in table 4.3, for Load Combination Group VII, in accordance with the recommendations discussed previously. The construction facing need not be considered for seismic loading conditions, as it will have a limited life. The seismic loading is accounted for by application of a seismic coefficient as a pseudo-static inertia force. The following guidance is recommended in defining the appropriate design seismic coefficient:

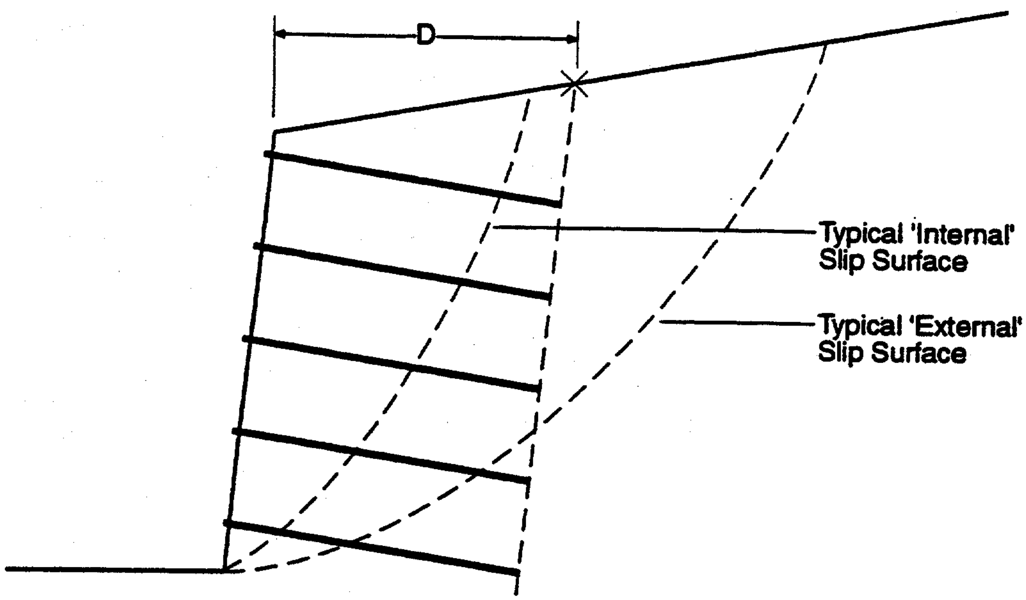
1. Select the appropriate design earthquake peak ground acceleration A_{PK}. In the absence of site specific data or local seismic map, A_{PK} can be taken off the AASHTO Division 1A [30] map of Horizontal Acceleration A.

2. For slip surfaces that are primarily "internal" in nature (i.e., intersect the nail reinforcements), define a design seismic coefficient $A = (1.45\text{-}A_{PK})A_{PK}$, in accordance with section 5.8.10 of the Standard Specifications for Highway Bridges, 15th Edition [30] recommendations for MSE walls. This design seismic coefficient shall be applied to "internal" slip modes as a pseudostatic earthquake acceleration, where the definition of "internal" is given on figure 4.14.

3. For slip surfaces that are primarily "external" in nature (i.e., either do not intersect the nail reinforcements or intersect them to a more limited extent), the design pseudo-static seismic coefficient A will vary depending on the permanent displacements that the retaining wall can tolerate during the design event. For example, if the wall can tolerate permanent displacements of up to $250A_{PK}$ mm (where A_{PK} is the design earthquake acceleration as a fraction of gravitational acceleration), then a design seismic coefficient equal to $0.5A_{PK}$ can be assumed (Section 6, Division 1A-Seismic Design, AASHTO, 15th Edition, [30]). For other tolerable permanent displacements, the appropriate seismic acceleration coefficient can be determined in accordance with AASHTO (figure 37, Seismic Design Commentary section, 15th Edition, [30]).

4. For assessment of seismic bearing stability of the reinforced soil block, a design seismic coefficient equal to $0.5\ A_{PK}$ is recommended.

The above design methodology is demonstrated by example in chapter 5 and appendix G.



'Internal' Slip Surfaces - intersect ground surface at < D from top of wall

'External' Slip Surfaces - intersect ground surface at > D from top of wall

D = Distance of top wall to envelope of nail tip (intersects ground surface)

Figure 4.14 Definition of 'Internal' and 'External' Slip Surfaces for Seismic Loading Conditions

4.7.2 Load and Resistance Factor Design (LRFD)

Step 1 - Set Up Critical Design Cross-Section(s) and Select a Trial Design

Select a trial design for the design geometry and loading conditions. The design cross-section should show the ultimate soil strengths for the various subsurface layers and design water table location (should be below wall base), and all applied service loadings such as self weight of soil and surface surcharges. These loads will be factored (see table 4.6) in accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29]. The load combinations to be considered are also summarized in table 4.6. A proposed trial design nail pattern, including nail lengths, tendon sizes, and trial vertical and horizontal nail spacings should be shown superimposed on the design section.

TABLE 4.6
LOAD COMBINATIONS AND LOAD FACTORS (AASHTO, LRFD, 1st Edition, 1994)

Limit State	DC DW EH EV ES	LL LS	WA	TU SH	EQ
STRENGTH I	γ_p	1.75	1.00	0.50	
STRENGTH IV EH,EV,ES,DW DC only	 γ_p 1.5		1.00	0.50	
EXTREME EVENT I	γ_p		1.00		1.00

Notes:
DC = dead load of structural components and non-structural attachments.
DW = dead load of wearing surfaces and utilities.
EH = horizontal earth pressure load.
EV = vertical pressure from dead load of earthfill.
ES = earth surcharge load.
LL = vehicular live load.
LS = live load surcharge.
WA = water load and stream pressure.
TU = uniform temperature.
SH = shrinkage.
EQ = earthquake.

LOAD FACTORS FOR PERMANENT LOADS, γ_p

Type of Load	Maximum Load Factor	Minimum Load Factor
DC	1.25	0.90
DW	1.50	0.65
EH		
Active	1.50	0.90
At-Rest	1.35	0.90
EV		
Overall Stability	1.35	N/A
Retaining Structures	1.35	1.00
ES	1.50	0.75

The loads and load combinations shown in this table are intended to illustrate AASHTO requirements considered most relevant to soil nail wall design. Specific designs may require consideration of additional loads and load combinations.

Step 2 - Compute the Design Nail Head Strength

Evaluate the design nail head strength for the trial construction facing and connector design, using the following procedure:

(a) Determine the nominal nail head strength for each potential failure mode of the facing and connection system, using the methods presented in section 4.5 and in appendix F. **As a design aid, the nominal nail head strengths for several soil nail wall typical facing thickness, facing reinforcement and bearing place connection combinations are tabulated in table 4.2B.**

(b) For each possible failure mode in table 4.7, determine the design nail head strength by multiplying the nominal nail head strength by the corresponding resistance factor. Table 4.7 presents resistance factors for both the Strength Limit States and the Extreme Limit State I (seismic loading) of table 4.6. In accordance with AASHTO [29], all resistance factors are taken as 1.0 for the Extreme Limit States of table 4.6. The design nail head strength is the *lowest* calculated value for the various failure modes.

TABLE 4.7
NAIL HEAD RESISTANCE FACTORS - LRFD

Failure Mode	Nail Head Resistance Factor (Strength Limit States) Φ_F	Nail Head Resistance Factor (Extreme Limit States) (Seismic)
Facing Flexure	0.90[a]	1.0[a]
Facing Punching Shear	0.90[a]	1.0[a]
Headed-Stud Tensile Fracture		
ASTM A307 Bolt Material	0.67[a]	1.0[a]
ASTM A325 Bolt Material	0.80[a]	1.0[a]

[a] Based on resistance factors per AASHTO LRFD Bridge Specifications, 1st Edition [29].

Step 3 - Minimum Design Nail Head Strength Check

Perform a minimum design nail head strength check for the trial facing design. This empirical check is performed to ensure that the computed design nail head strength exceeds the estimated (factored) nail head service load that may actually be developed as a result of soil-structure interaction. The nail head service load is multiplied by a load factor of 1.5 (see table 4.6) in accordance with AASHTO [29], for active horizontal earth pressure loads. With reference to figure 2.7, the nail head service load actually developed can be estimated by the following empirical equation:

$$t_F = F_F K_A \gamma H S_H S_V \qquad (4.8)$$

Unless the designer has site specific monitoring information from walls constructed in similar soils, it is recommended that a value of nail head service load factor, F_F, equal to 0.50 be adopted for design purposes. (See section 2.4.5).

For simple configurations (i.e., uniform soil conditions, no surcharge, etc.) the active earth pressure coefficient K_A can be determined directly from published equations and charts, neglecting the cohesive component of soil strength in accordance with the discussion in sections 2.4.4 and 2.4.5. For more complex configurations that are not tabulated in the published literature (e.g., variable soil layers, complex wall geometries and surcharge load distributions), the nail head service load can be estimated from

$$t_F = 2 F_F P_A S_H S_V / H \qquad (4.9)$$

The active load P_A can typically be determined using a Coulomb-type slip surface (i.e., slope stability) calculation. If the computed design nail head strength is less than the factored estimated nail head service load, the trial facing/connector design should be modified and step 2 repeated.

Note: This check is not applied for seismic loading conditions. Since 1)There is no service load data for seismic conditions 2)There is no evidence of poor permanent facing performance under such loading.

Step 4 - Define the Design Nail Strength Support Diagrams

Define the design nail strength for each of the nails as a function of location along the nail length. As shown on figures 4.3, the design nail strength will vary along the length of the nail and will depend on the design nail head strength, the design nail tendon strength, and the design nail-ground pullout resistance.

The design nail head strength is determined as discussed in step 2 using LRFD resistance factors presented in table 4.7.

The design nail tendon strength is taken as the tendon yield strength multiplied by a resistance factor presented in table 4.8, in accordance with AASHTO [29]. It is recommended that the minimum bar size used for soil nails be No. 19 (Soft Metric designation) - corresponding to a No. 6 standard bar size. However, nail sizes smaller than No. 25 (metric) may cause installation problems for moderate to long nail lengths due to their low stiffness. Bar size designations are provided in table 4.8A and appendix F.

The design nail pullout resistance will determine the rate at which the design nail strength can change along the length of the nail and is taken as the ultimate ground-grout pullout resistance multiplied by a resistance factor (see table 4.8). The ultimate pullout resistance may be determined from local experience, published data, or field testing and is often expressed in terms of a force per unit length of nail. See section 3.2 for guideline published pullout resistance data.

Table 4.8 summarizes the recommended LRFD resistance factors for both the Strength Limit States I and IV, and the Extreme Limit State I (seismic loading) of table 4.6.

TABLE 4.8
RESISTANCE FACTORS - LRFD

Element	Resistance Factor (Strength Limit States) Φ	Resistance Factor (Extreme Limit States) (Seismic)
Nail Head Strength	Φ_F = Table 4.7	see Table 4.7
Nail Tendon Tensile Strength	$\Phi_N = 0.90$	1.0
Ground-Grout Pullout Resistance	$\Phi_Q = 0.70$	0.8
Soil Cohesion	$\Phi_C = 0.90$ (0.90*)	1.0 (1.0*)

Element	Resistance Factor (Strength Limit States) Φ	Resistance Factor (Extreme Limit States) (Seismic)
Soil Friction	$\Phi_\phi = 0.75$ (0.65*)	1.0 (0.9*)

Element	Resistance Factor (Strength Limit States) Φ	Resistance Factor (Extreme Limit States) (Seismic)
Soil Cohesion - Temporary Construction Condition†	$\Phi_C = 1.00$ (1.00*)	NA
Soil Friction - Temporary Construction Condition†	$\Phi_\phi = 0.85$ (0.75*)	NA

*Soil strength resistance factors for "critical" structures.

Notes:

Design Nail Head Strength (T_F) = Φ_F (Nominal Nail Head Strength) = $\Phi_F T_{FN}$
Design Nail Tendon Strength (T_N) = Φ_N (Tendon Yield Strength) = $\Phi_N T_{NN}$
Design Pullout Resistance (Q) = Φ_Q (Ultimate Pullout Resistance) = $\Phi_Q Q_U$
Design Soil Cohesion (c) = Φ_C (Ultimate Soil Cohesion) = $\Phi_C c_U$
Design Soil Friction Angle (ϕ) = $\tan^{-1}(\Phi_\phi[\tan \phi_U])$

LRFD requires that the factored resistances equal or exceed the factored loads i.e. resistance/load ratio ≥ 1.

† Refers to temporary condition existing following cut excavation but before nail installation. Does not refer to "temporary" versus "permanent" wall.

Step 5 - Select Trial Nail Spacings and Lengths

As discussed in section 4.3.2 and as shown on figure 4.7, satisfaction of the strength limit state requirements will not in itself ensure an appropriate design. Additional constraints are required to provide for an appropriate nail layout. The following empirical constraints on the design analysis nail length pattern are therefore recommended to be applied when performing the limiting equilibrium design calculations:

(a) Nails with heads located in the upper half of the wall height should be of uniform length.

(b) Nails with heads located in the lower half of the wall height should be considered to have a reduced length in accordance with the recommendations given on figure 4.11.

The purpose of these recommendations is to ensure that adequate nail reinforcement (length and strength) is installed in the upper part of the wall. Performance monitoring of several instrumented soil nail walls, in which both nail loads and wall movements were measured, have demonstrated that the top-down method of construction of soil nail walls generally results in the nails in the upper part of the wall being more significant than the nails in the lower part of the wall in developing resisting loads and controlling displacements. If the strength limit state calculation overstates the contribution from the lower nails, then this can have the effect of indicating shorter nails and/or smaller tendon sizes in the upper part of the wall, which is considered undesirable as it could result in poorer in-service performance.

It should be noted that the above recommendation for the nail length pattern to be assumed for design calculation purposes does not imply that the installed nail pattern must correspond exactly to the design nail pattern. It is most common to install nails of uniform length, often to simplify construction. Provided the appropriate external stability checks are performed (see step 8), it may be possible to install shorter nails in the lower part of the wall.

Step 6 - Define the Design Soil Strengths

Define the design soil strengths for analysis. Methods for determining appropriate soil strengths for design are discussed in section 3.1. Reasonably accurate soil strength characterization is an important part of the design process and should be performed only by qualified experienced geotechnical personnel. The design soil strength is defined by multiplying the ultimate soil strength by the resistance factors given in table 4.8.

Step 7 - Calculate the Resistance/Load Ratio

Calculate the limiting equilibrium resistance to load ratio for each potential slip surface, taking into account the additional stabilizing forces provided by the trial pattern of nails. For LRFD, all loads are factored up and all resistances, including resistance provided by the soil shear strength, are factored down. A minimum calculated global resistance/load ratio of 1.0 is required, consistent with the design nail strengths and the design soil strengths (see table 4.8), to ensure that the factored resistances equal or exceed the factored loads. Table 4.8 shows that the recommended soil strength resistance factors are decreased when the soil nail wall supports critical structures.

It will also be necessary to check the stability of the soil nail wall during its construction. In particular, the temporary construction condition in which the lift excavation has occurred but the nail has not yet been installed, must be considered. In these circumstances, because of the temporary nature of such conditions, it is recommended that the nail resistance factors applied be the same as those shown in table 4.8, but that the soil strength resistance factors be increased to a value of 0.85 (0.75 for critical structures) for soil friction and 1.0 for soil cohesion, as shown in table 4.8. In general, for most applications and typical construction conditions, construction stability requirements will not control the design. However, in certain circumstances such as significant existing surcharge loadings adjacent to the wall during construction, construction conditions may be more critical. This issue is further discussed in section 4.10.2, 4.10.3 and figure 4.18.

Step 8 - External Stability Check

Perform stability analyses for potential "external" failure modes. As noted in section 4.2.4, the potential external failure modes that require consideration with the slip surface method include overall slope failure external to the nailed mass, and foundation bearing capacity failure beneath the laterally loaded soil nail "gravity" wall. The methods of analysis for these failure modes are equivalent to those used for any gravity retaining structure.

For overall slope stability, the appropriate resistance factor to be applied to the ultimate soil strengths is 0.85 for strength limit state loading (Section 10.5.4 - AASHTO, [29]). When performing overall slope stability checks, consideration should be given to potential slip surfaces that pass beneath the base of the wall and exit downslope or in front of the wall toe. This condition will be more critical if the ground water table is located close to the base of the wall. For this reason, it is recommended that nail lengths should not be shortened in the lower part of the wall (reference step 5, above) unless stability analyses confirm that more deep-seated failure modes will not control.

Although bearing capacity seldom controls the design, a rough bearing capacity check should be made to insure global stability. In general, bearing capacity should be checked in situations where cohesive soils exist within a depth equal to the width of the soil nailed mass. The equations shown in section 4.2.4 should be used to determine if the potential exists for bearing capacity failure or basal heave. If the safety factor from these analyses is less than 2.5, a more rigorous bearing capacity analysis as shown in figure 4.12 should be performed. The appropriate resistance factor to be applied to the ultimate bearing capacity from the rigorous analysis is described in section 10.5.4 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], for the Strength Limit States (I and IV). For Extreme Limit States, the appropriate resistance factor is 1.0.

Step 9 - Check the Upper Cantilever

The upper cantilever section of a soil nail wall facing (above the top row of nails) will be subjected to earth pressures that arise from the self weight of the adjacent soil and any surface surcharge loadings or inertial forces acting upon the adjacent soil. The magnitude of these earth pressures will depend not only on the strength of the soil but also on factors such as the method of fill placement (if any) behind the cantilever. For no fill placement and associated compaction-induced stresses behind the upper cantilever following its construction, an active earth pressure coefficient can be assumed for the upper cantilever portion of the wall. Because the upper cantilever is unable to redistribute load by soil arching to adjacent spans as can the remainder of the wall facing below the top nail row, the strength limit state of the cantilever must be checked for moment and shear at its base, as described in figure 4.13.

If heavy compaction occurs in close proximity to the upper cantilever section of the wall, additional lateral earth pressures will be induced on the wall. Therefore, if it is necessary to place compacted fill in this area, light compaction equipment should be used.

In the horizontal span direction, cantilevered spans will exist at the ends of the soil nail wall (end spans), and at locations of vertical expansion joints for the permanent facing. For expansion joints, it is common practice to keep the same nail pattern and uniform horizontal nail spacing as in the remainder of the wall segment, and locate the expansion joint directly between two columns of nails. For cantilevered end spans, normal design and construction practice is such that the cantilever span is generally in the range of one-third to two-thirds of the average nail spacing. If these criteria are adhered to, no formal additional design of the facing is required at these locations. These construction practices have consistently resulted in good performance of horizontal cantilever spans. The main reason for this is that the geometry limitations described above allow the soil behind the cantilever span to redistribute pressures to adjacent interior spans with only minor increases in deformations in that region of the wall.

For the cantilever at the bottom of the wall, the method of construction tends to result in minimal to zero loads on this cantilever section during construction. There is also the potential for any long-term loading at this location to arch across this portion of the facing to the base of the excavation. It is therefore recommended that no formal design of the facing be required for the bottom cantilever. It is also recommended, however, that the distance between the base of the wall and the bottom row of nails not exceed two-thirds of the average vertical nail spacing.

Step 10 - Check the Facing Reinforcement Details

Check waler reinforcement requirements, minimum and maximum reinforcement ratios, minimum cover requirements, and reinforcement development and splices, as described below.

Waler Reinforcement

For temporary shotcrete construction facings, it is common practice and therefore recommended that a minimum of two (2) No. 13 (Soft Metric designation - see appendix F) deformed horizontal waler bars be placed continuously along each nail row and located behind the face bearing plate at each nail head (i.e., between the face bearing plate and the back of the shotcrete facing). The main purpose of the waler reinforcement is to provide additional ductility in the event of a punching shear failure, through dowel action of the waler bars contained within the punching cone.

In permanent facings placed over the shotcrete construction facing, no waler bars are required because of the reduced pressure factors discussed in section 4.5,

Minimum and Maximum Reinforcement Ratios

Minimum and maximum reinforcement ratios are specified in sections 5.7.3.3 and 5.10.8 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29]. These provisions are intended to provide that flexural failure mechanisms remain ductile (section 5.7.3.3) and to provide a minimum amount of resistance to shrinkage and temperature distress (section 5.10.8). However, soil nail wall facings are inherently ductile even at extremely low reinforcement ratios. Therefore, the provisions in section 5.7.3.3 should not be applied to soil nail walls.

In addition, for temporary shotcrete facings, shrinkage and temperature cracking is not a significant concern and the minimum steel ratio requirements of section 5.10.8 may be waived for the temporary facing.

The AASHTO LRFD Bridge Design Specifications, 1st Edition [29], does specify maximum reinforcement ratio requirements in section 5.7.3.3. However, because typical soil nail wall construction practice involves very lightly reinforced facings, these provisions need not be checked.

In summary, therefore, , only the shrinkage and temperature reinforcement requirements of section 5.10.8 must be checked and only for permanent facing systems.

Minimum Cover Requirements

Concrete or shotcrete cover is necessary to provide bond resistance for the reinforcing steel and corrosion protection to the reinforcing steel, the bearing plate and headed studs, as well as any non-encapsulated or non-epoxy coated nail steel. In permanent applications, corrosion protection is a vital component of the design. These provisions are contained in section 5.12.3 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29]. On the front side of permanent facings exposed to the weather, the minimum cover required is 50 mm. For permanent shotcrete facings, on the side of the facing exposed to the soil, the minimum cover required is 75 mm. For permanent CIP facings, on the side of the facing cast against the temporary shotcrete, the minimum concrete cover required is 38 mm for the reinforcing steel (i.e., minimum distance from the CIP concrete reinforcing steel to the concrete-shotcrete interface), and the minimum shotcrete/concrete cover required from the side of the shotcrete facing exposed to the soil is 75 mm for all steel. For the temporary shotcrete construction facing, corrosion protection is not a concern, and it is adequate to place the reinforcing steel near the center of the facing.

Development and Splices of Reinforcement

Check that the splices and cutoff locations of all mesh reinforcement, deformed reinforcing bars, horizontal waler bars, and vertical bearing bars (if used) are sufficient to develop the yield stress of the reinforcement at all locations at which it is needed. All splices and development lengths shall be proportioned in accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition (section 5.11) [29].

Step 11 - Serviceability Checks

Check wall function as related to excess deformation and cracking (i.e., check the service limit states). The following issues should be considered:

(a) Service Deflections and Crack Widths of the Facing

In accordance with the provisions for two-way slabs of the ACI Building Code Requirements for Reinforced Concrete [38] (note that two-way slabs are not addressed in AASHTO), crack widths in the soil nail wall facing are not checked. However, ACI 318-95 [38] does provide slab deflection criteria through minimum span-to-depth ratios or deflection-to-span ratios that must be met. Because the span-to-depth ratios for both temporary and permanent soil nail wall facings never exceed about 20, the structural deflections that occur at service load levels are insignificant and therefore not an issue.

The upper cantilevers of permanent soil nail walls are essentially one-way cantilevered slabs and may have larger effective span-to-depth ratios than interior two-way spans. Therefore, the service crack widths (steel stresses) must be checked in the same manner as for the stem of a conventional cantilever retaining wall. The provisions of section 5.7.3.4 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], are used to check the crack widths. For most temporary facings (whether as part of a temporary shoring system or as the construction facing of a permanent wall system), serviceability requirements are not imposed because the deflections do not pose any significant aesthetic or durability concerns.

b) Overall Displacements Associated With Wall Construction

Section 2.4.6 discusses the displacement magnitudes and patterns typically associated with the construction of soil nail walls in various ground types. Since the serviceability of the wall must be assessed in terms of potential impacts on adjacent structures and facilities, the data presented in section 2.4.6 can be used to make these determinations.

(c) Facing Vertical Expansion and Contraction Joints

Vertical joints are not required in the temporary shotcrete construction facing.

Per AASHTO section 11.6.1.5, LRFD, 1st Ed. [29] and AASHTO section 5.5.6.5, 15th Ed. [30], contraction joints spaced at intervals not exceeding approximately 10 m and expansion joints spaced at intervals not exceeding approximately 30 m, as required for conventional concrete retaining walls, can be used in permanent CIP or permanent shotcrete final finish facings. Typical joint details are shown on the appendix A Example Plans.

Seismic Design

For seismic loading conditions, the resistance factors should be set to the values indicated in table 4.8 and the load combination and load factors of Extreme Event I (table 4.6) should be evaluated. The construction facing need not be considered for seismic loading conditions, as it will have a limited life. The seismic loading is accounted for by application of a seismic coefficient as a pseudo-static inertia force. The following guidance is recommended in defining the appropriate design seismic coefficient:

1. Select the appropriate design earthquake peak ground acceleration A_{PK}. In the absence of site specific date or local seismic map, A_{PK} can be taken off the AASHTO Division 1A [29] map of Horizontal Acceleration A.

2. For slip surfaces that are primarily "internal" in nature (i.e., intersect the nail reinforcements), define a design seismic coefficient $A = (1.45-A_{PK})A_{PK}$, in accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition , section 11.9.6 [29] recommendations for MSE walls. This design seismic coefficient shall be applied to "internal" slip modes as a pseudostatic earthquake acceleration using the definition of "internal" given in figure 4.14.

3. For slip surfaces that are primarily "external" in nature (i.e., either do not intersect the nail reinforcements or intersect them to a more limited extent), the design pseudo-static seismic coefficient A will vary depending on the permanent displacements that the retaining wall can tolerate during the design event. For example, if the wall can tolerate permanent displacements of up to $250A_{PK}$ mm (where A_{PK} is the design earthquake acceleration as a fraction of gravitational acceleration), then a design seismic coefficient equal to $0.5A_{PK}$ can be assumed (section 11, appendix A, AASHTO, LRFD 1st Edition, [29]). For other tolerable permanent displacements, the appropriate acceleration coefficient can be determined in accordance with AASHTO [29].

4. For assessment of seismic bearing stability of the reinforced soil block, a design seismic co-efficient equal to $0.5\ A_{PK}$ is recommended.

The above design methodology is demonstrated by example in chapter 5 and appendix G.

4.8 Corrosion Protection

The long-term performance of permanent soil nails requires that they be able to withstand corrosive attack from their local environment. Characteristics defining the corrosive potential of the soil environment (i.e., ground aggressivity) are summarized in section 3.1.

4.8.1 Nail Tendon Corrosion Protection

The following constitutes FHWA recommended guidelines for nail corrosion protection on U.S. Federal-aid highway projects. For **permanent** applications, soil nail corrosion protection should consist of the following:

- In non-aggressive ground, the nail section should be resin-bonded epoxied using an electrostatic process to provide a minimum epoxy coating thickness of 0.3 mm in accordance with AASHTO M-284 [39]. The intact epoxy coating will prevent tendon corrosion by isolating the tendon from the surrounding environment. In addition, the recommended minimum thickness of coating will generally prevent normal handling and construction-induced damage. A minimum grout cover of 25 mm is recommended throughout the length of the nail. Centralizers should be placed at distances not exceeding 2.5 meters center to center, and the lowest centralizer should be placed a maximum of 0.3 meters from the bottom of the grouted drill hole. The centralizers should be made from a plastic material, be attached to the nail in a way that will not impede the free flow of grout, and be sized to position the nail tendon within approximately 25 mm of the center of the drill hole.

- In aggressive ground or for critical structures[1] (e.g., walls adjacent to lifeline high volume roadways or walls in front of bridge abutments) or where field observations have indicated corrosion of existing structures, encapsulated nails should be used. Encapsulation is generally accomplished by grouting the nail tendon inside a corrugated plastic sheath. A neat cement grout containing admixtures to control water bleed from the grout is usually employed to fill the annular space (typically 5 mm minimum) between the plastic sheath and the tendon. For this type of protection, the minimum grout cover between the sheath and the borehole wall should not be less than 12 mm.

For **temporary** applications (commonly stated to be 18-36 months, but may be shorter or longer based on actual project conditions) in non-aggressive ground, the soil nail grout is considered adequate protection.

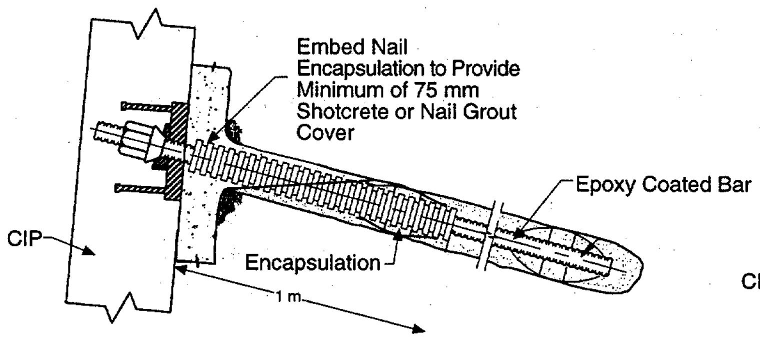
4.8.2 Nail Head Corrosion Protection

If the nail is encapsulated or is an epoxy-coated deformed bar with machine threads at the upper end, the corrosion protection is terminated to expose the bare tendon at the head of the nail in order to allow attachment of the bearing plate and nut. This area may be more susceptible to corrosion than the remainder of the nail since oxygen is more readily available.

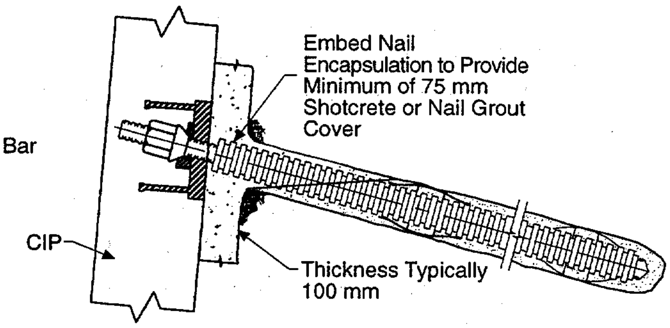
[1] *Determination of structures considered "critical" is the prerogative of individual State agencies.*

For the above type of nail tendons and corrosion protection, the following approach has been most commonly used for providing corrosion protection to the nail head. First, the bearing plate assembly is embedded in the permanent facing with the normal depths of cementitious cover to control steel corrosion. Second, the nail tendon protection (epoxy coating or encapsulation) is extended into the shotcrete construction facing to ensure a minimum depth of shotcrete/nail grout cover of 75 mm. Figure 4.15 shows examples of the types of acceptable corrosion protection systems for permanent soil nails.

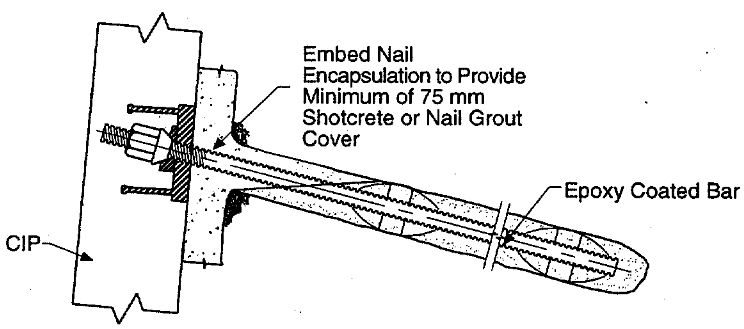
When epoxy coated continuous threadbars are used, the threadbars are commonly coated full length. CALTRANS experience indicates that the use of a 0.3 mm coating thickness still allows the bearing plate nut to be threaded onto the bar over the epoxy coating. This may not be the case with other types of continuous threadbars.



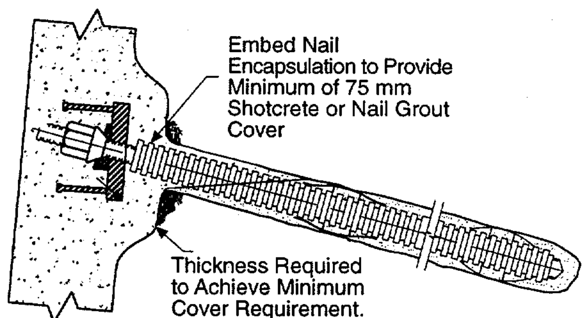
a) Epoxy Coated Soil Nail Detail (Upper 1 Meter Encapsulated) With Temporary Shotcrete Construction Facing and Permanent CIP Facing (Caltrans)



c) Encapsulated Soil Nail With Bare Steel at the Top Detail, With Temporary Shotcrete Construction Facing and Permanent CIP Facing



b) Epoxy Coated Nails With Machine Threads Detail, With Temporary Shotcrete Construction Facing and Permanent CIP Facing



d) Permanent Shotcrete Facing Detail With Encapsulated Nail

Figure 4.15 Alternative Soil Nail Corrosion Protection Details

4.9 Wall Drainage

Typical soil nail wall drainage systems, which are detailed in section 3.1 of appendix B1, include geotextile face drains, shallow PVC drain pipes and weep holes, surface interceptor/collector ditches, and surface waterproofing. Other approaches include deep horizontal drains for control of flowing water and for ground water depressurization when an unanticipated water table is encountered and use of vegetation with stepped or benched walls to inhibit infiltration and to lower soil water contents by evapotranspiration.

For most applications control of water during construction will be concerned with controlling surface runoff and subsurface flow associated with either perched water or localized seepage areas.

A surface interceptor ditch, excavated along the crest of the excavation and lined with concrete applied during the shotcreting of the first excavation lift, is a recommended element for controlling surface flows. The ditch should be contoured to drain away from the working area, with tightline collector drain pipes installed at appropriate locations, if necessary. Where larger graded slope areas exist above the wall, installation of plastic film slope protection sheeting above the interceptor ditch provides another quick and inexpensive means of controlling surface water during construction. Similar permanent surface drainage measures are generally required to prevent surface waters from infiltrating behind the facing or flowing over the top of the wall, during the operational life of the structure. For stepped or benched walls, vegetation can also be used to inhibit infiltration and lower soil water.

With respect to subsurface groundwater control, long-term drainage measures may include the following:

- Face Drains: These are typically 400-mm-wide prefabricated geotextile drain strips that are placed in vertical strips down the excavation face, on a horizontal spacing corresponding to the nail horizontal spacing, *and* discharging either into a base drain or through weep holes at the bottom of the wall. The top of these geocomposite drains should not be extended to the ground surface as these drains are not designed to handle surface drainage.
- Shallow Drains (Weep Holes): These are typically 300-to 400-mm-long, 50-to 100-mm diameter PVC pipes discharging through the face and located where heavier seepage is encountered.
- Horizontal Drains: If it is determined that the soil retained by a soil nail retaining wall will be subjected to ground water pressures (perhaps related to a seasonal change in the ground water table) and there is no serious impediment to construction of the wall, then the wall must be designed to support the anticipated driving and uplift ground water forces. Deep horizontal drains, typically consisting of 50 mm diameter slotted or perforated tubes and inclined upward at 5 to 10 degrees to the horizontal, may be installed to control the ground water pressures imposed on the retained soil mass. The design spacing and depth of these drains are site specific, but they will typically be longer than the length of the nails and with a density of approximately one drain per 10 square meters of face. Deep horizontal drains may also be used to control unanticipated

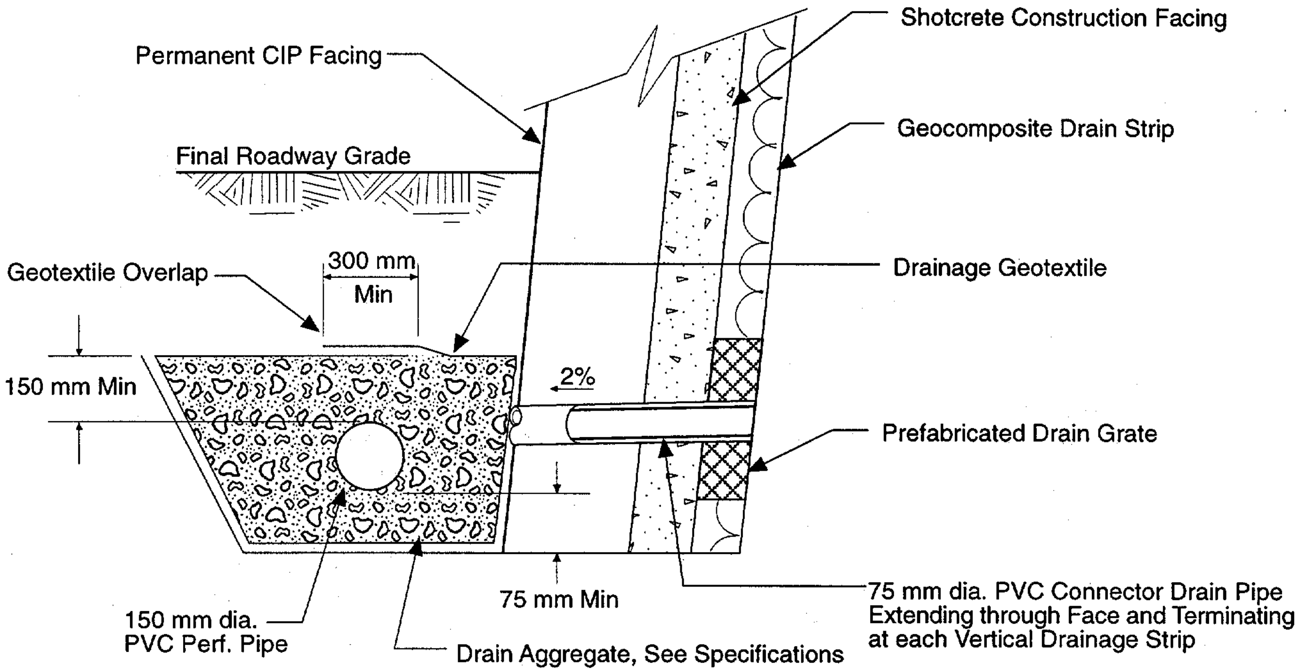
water flow during construction. Design issues that require consideration for horizontal drains include:

- Horizontal drains are installed after nail installation to prevent potential problems with nail grout entering the slotted pipes.
- Construction phase water lowering by horizontal drains is generally only done in special situations as requirements are needed for special face sloping, berming, or trenching and special collection procedures for drain effluent as the excavation deepens.
- Nail rows should not be layed out in an offset or staggered pattern (section 4.4.2). A sufficiently wide, unobstructed path is needed for installation of the upward sloping drains. Alignment of the horizontal drain drilling equipment should be carefully checked during installation to avoid impacting any previously installed nails.
- For aesthetic reasons, the drain outlets may have to be plumbed and carried down the wall face between the shotcrete construction facing and permanent CIP facing and then outlet at the wall base. Horizontal drain flow exiting directly out through and flowing down the exterior permanent face is unsightly.

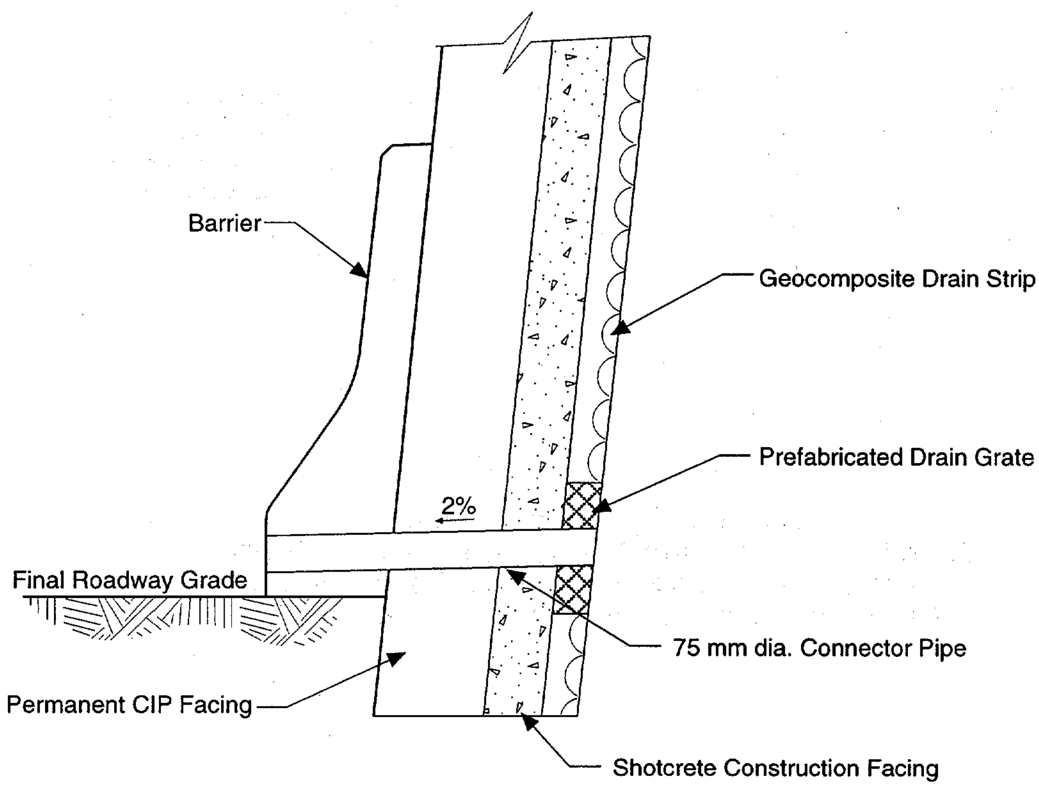
During construction, if drainage systems are collecting significant water, it is often necessary for the contractor to collect and conduct these inflows away from the construction area so that the water does not "follow" the contractor down the face.

In blocky ground that produces a very rough and irregular excavation face, the placement of prefabricated drain strips against the excavation face is difficult and often impractical. In some cases, the prefabricated drain strips may be sandwiched between the shotcrete construction facing and the permanent CIP facing, with the drain placed over 50 to 75 mm diameter weep holes passing through the construction facing (see figure 4.16).

Typical permanent face drain configurations for geotextile drain strips discharging either into toe drains or through weep holes in the facing are shown on figure 4.16.

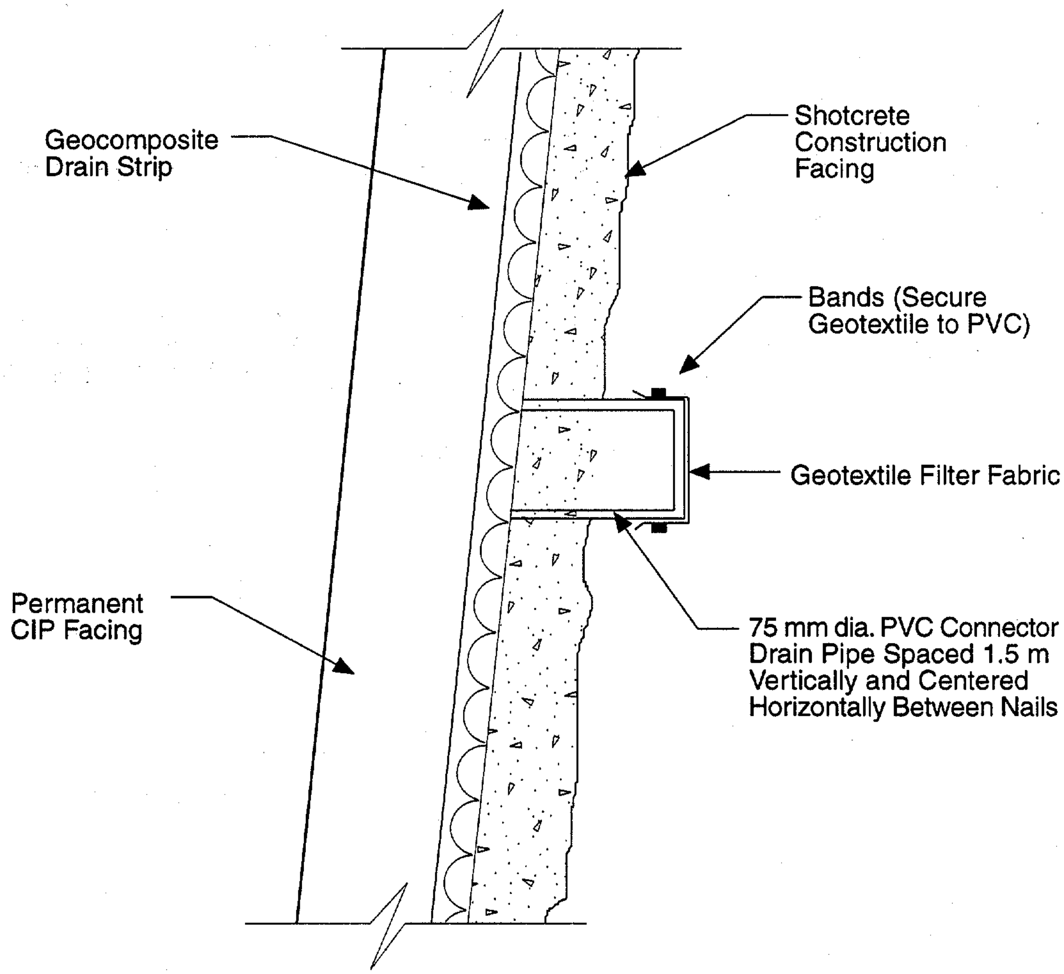


a) Typical Wall Toe Drain



(b) Typical Weep Hole Drain

Figure 4.16 Example Wall Drainage Details
(continued on next page)



c) Rough Excavation Face
Drain Strip Detail With Geocomposite Drain Strip Sandwiched Between Shotcrete and Permanent CIP Facing (courtesy Schnabel Foundation Co.)

Figure 4.16(cont.) Example Wall Drainage Details

4.10 Special Design Considerations

The simplest soil nail retaining wall consists of a vertical or battered planar wall, and a homogeneous soil reinforced with nails of constant length and orientation. More complex configurations are not uncommon, however, including heterogeneous ground conditions, nails of variable length and orientation, non-planar facings, and wall loadings other than those associated with the self weight of the reinforced and retained ground. These variations, discussed briefly below, can often be relatively easily incorporated into the recommended slip surface limiting equilibrium design methodology presented in section 4.7. In some applications, however, there is relatively little experience on which to base design recommendations and expert assistance should be obtained.

4.10.1 Heterogeneous Soil Profiles

Because the soil nailing technique is concerned with the reinforcement of in situ ground rather than of controlled structural fills, it is relatively common to encounter heterogeneous conditions with respect to self weight, soil strength and nail pullout resistance. In principle, such conditions pose no particular computational difficulties for slip surface limiting equilibrium techniques. However, the following points should be considered:

- Some of the available soil nailing computer design codes are restricted to relatively simple soil profile heterogeneities, such as single uniform soil type or only horizontally layered systems.

- Sensitivity studies should be conducted to examine the impacts of soil with severe heterogeneities, such as soil overlying bedrock. Good engineering judgment might require that the full rock shear strength or nail pullout resistance in the rock not be incorporated into specific zones of the design model, if it appears that this would result in an unrealistic computed factor of safety. Also, it is considered generally inappropriate to develop designs in which a small fraction of the nails are responsible for a large portion of the total nail support.

- Whenever heterogeneities of any type are introduced (e.g., variable soil properties, highly non-uniform surface surcharges), the critical slip surface might not exit in the vicinity of the toe of the wall and a more complete search is generally required. An example of this condition would be a weaker soil overlying a substantially stronger soil, in which the critical slip surface might exit the wall facing in the vicinity of the weak soil-strong soil contact, rather than exiting through the wall toe (figure 4.17).

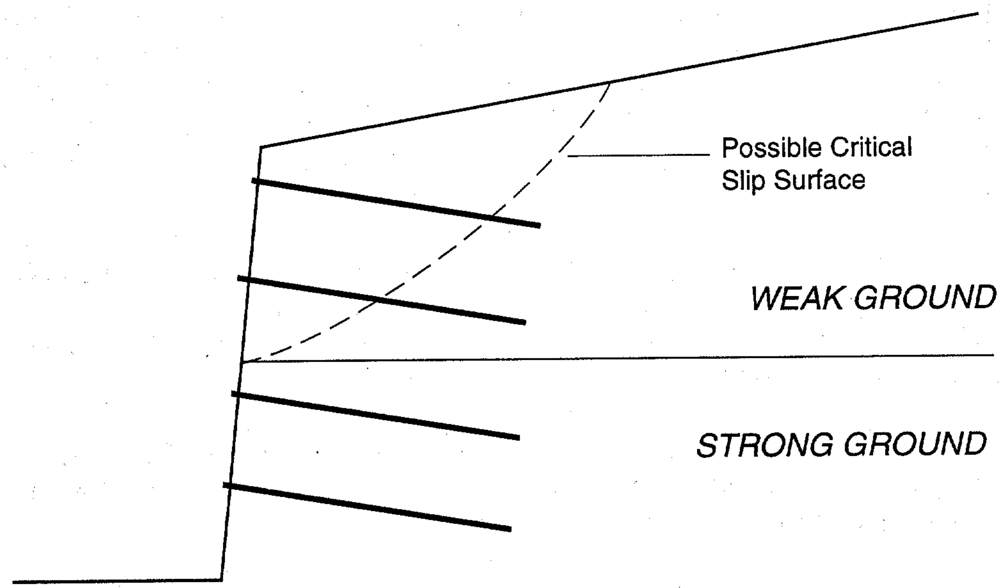


Figure 4.17 Influence of Heterogeneous Ground Conditions on Critical Slip Surface

4.10.2 Surcharge Loading

Relatively uniform vertical and horizontal surcharge loadings applied to the surface of a soil nail retaining wall structure can be addressed in a similar manner to the self weight gravity load of the reinforced and retained ground. The modeling of soil surcharges should generally account for both the self weight of the soil (vertical loading) and the lateral earth pressure (horizontal loading) exerted by the fill. All potential slip surfaces must be evaluated and minimum factor of safety requirements met in accordance with the criteria presented in section 4.7. The surcharge loadings may range from relatively light (e.g., nominal live load allowances for equipment and traffic operating above the retaining wall) to very heavy loads in relation to the weight of the retained ground (e.g., surcharge corresponding to an MSE or conventional retaining structure or bridge abutment spread footing located on top of the soil nail retaining wall). The minimum facing/connection system strength requirements should also be established in accordance with the recommendations of section 4.7, taking into account the loads applied by both the ground self weight and the surface surcharges. It should be noted that there is currently little published information on measured service nail and facing loads for soil nail walls loaded with heavy surface surcharges, although a few such instrumented walls have been built on recent U.S. highway projects (e.g., I-405 Renton, WA; Portland LRT, Portland, OR).

For relatively uniform surface surcharges and homogeneous soil profiles, the critical slip surfaces giving the lowest calculated factors of safety will tend to pass near the toe of the wall and not through an intermediate point higher up in the wall.

4.10.3 Bridge Abutments

As discussed in chapter 1, one of the most useful applications of the soil nailing technique in highway construction and improvement projects is for bridge underpass widening (figure 2.3). This activity requires the removal of lateral restraint in the vicinity of the bridge foundation, by excavation of the bridge abutment retaining slope, and the almost simultaneous replacement of the removed lateral support with a soil nail retaining wall. A detailed example of a bridge overpass widening is shown in appendix G.

For soil nail walls constructed in front of existing shallow or deep foundation supported bridge abutments, the estimated top of wall and abutment movement that will be induced by the wall excavation must be able to be tolerated by the abutment and superstructure. This will be an especially important design consideration for abutments on shallow foundations, particularly where the wall face excavation would have to be made close to the foundation footing. It is emphasized that this is a critical application and should be considered only for sites where the shallow bridge footing is supported by very competent ground.

Where the bridge is supported on a deep foundation such as vertical or battered piles or piers, and the deep foundation extends well below the base of the wall such that its bearing capacity will not be significantly influenced by the slope removal and wall construction, it is recommended that the deep foundation and the soil nail retaining wall be considered as essentially independent systems. For example, the bridge vertical loads might be carried exclusively by the deep foundation and the soil nail retaining wall will be proportioned to carry the surcharge loads associated with the bridge approach fill, together with the other typical dead and live surface loads. In this respect, the design problem is no different from a typical cut slope application.

However, there are some additional issues that must be considered for such an application:

- Some fill soils may not be well suited for soil nailing. Examples include clean, loose granular soils (poor stand-up time) or fills containing numerous cobbles, boulders, rubble or other obstructions (difficult excavation and drilling).

- The presence of the piles or piers behind the future retaining wall will place restrictions on the nail layout, in particular on the nail horizontal spacings and nail-head locations.

- In addition to the vertical loads that are supported directly by the deep foundation, the bridge abutment will also be subjected to lateral earth pressure loads associated with the approach fill, as well as horizontal loads associated with longitudinal bridge temperature shrinkage and expansion. Unless the deep foundation can be demonstrated to have sufficient lateral stiffness to support these horizontal loadings with minimal horizontal displacement (e.g., battered piles), it is recommended that these horizontal loads be applied as lateral surcharge loadings in the nail wall design, together with the vertical surcharge loads associated with the approach fill. This approach is demonstrated by the bridge abutment example problem in chapter 5. In addition, any longitudinal bridge movement due to temperature shrinkage and expansion that can be transmitted into the nail wall must be assessed and judgement made as to whether it is tolerable and can be accommodated by the nail wall.

- Water flows in existing bridge drains must be controlled.

Where ground conditions are suitable and very competent (e.g., very dense soil or weathered rock with favorable geologic structure), a soil nail retaining wall can also be used to achieve bridge underpass widening where the bridge is supported by shallow foundations. Once again, the surface surcharge loading imposed by the shallow footing does not pose any particular problems from a design analysis perspective, although the following consideration must be addressed:

- The surface loading will tend to be non-uniform, with a higher bearing pressure applied over the relatively narrow width of the footing. As with other non-homogeneities, a broader range of potential slip surfaces must be considered as the critical slip surface location will tend to depend on the location of the concentrated footing loading behind the facing (figure 4.18).

Particular attention must be paid to the details of the construction process, including the temporary conditions that will exist during construction in the periods between lift excavation and nail and facing installation. It must be ensured that stability will not be compromised at any stage during the construction excavation process, and not just for the final configuration. Stability analyses should therefore be conducted for all potentially critical intermediate conditions to ensure that the foundation is not temporarily compromised by removal of lateral restraint (figure 4.18). Assessment of stability during construction is a requirement for all soil nail walls, but is of particular importance in applications that include significant surcharge loads. As noted in section 4.7, temporary construction conditions should be assessed with reduced soil strength (or global) factors of safety (SLD) or increased soil strength resistance factors (LRFD).

Some photo examples of permanent soil nail walls used in bridge end slope removal applications on U.S. highway projects in front of both deep and shallow foundation supported bridge abutments are shown in the FHWA "Soil Nailing Field Inspector's Manual" [26].

4.10.4 Stepped Structures

Aesthetic requirements may call for the use of a stepped or benched facing for a soil nail retaining wall, with horizontal setbacks between the individual wall sections (figure 4.19). These setback areas are often used for planting vegetation i.e., "greening" the wall. Where horizontal setbacks are "small" in relation to the height of the individual benches, the structure will tend to act as a single equivalent wall with a battered face. Conversely, where the horizontal setbacks are "large" (typically 1.5 times the height) in relation to the height of the individual benches, the individual benches will tend to act as totally independent walls. Since the definition of "small" and "large" in this context is dependent on the material properties, it will generally be necessary to evaluate the stability both of the overall wall and of sections of the wall comprised of single (and possibly multiple) benches and to design for the most critical case.

4.10.5 Composite Structures

Composite structures involving the use of soil nailing may take a variety of forms, together with very different construction sequences.

(a) Nails and Tiebacks

One type of composite structure is that in which nails are combined with a second method of support (typically active tieback anchors), and both supporting elements are installed as excavation proceeds from the top down. The design methodology will depend on the configuration of the support system, and particularly on the relative contribution and intended function of the nails and the tiebacks. Figure 4.20 shows two basic composite nail-tieback

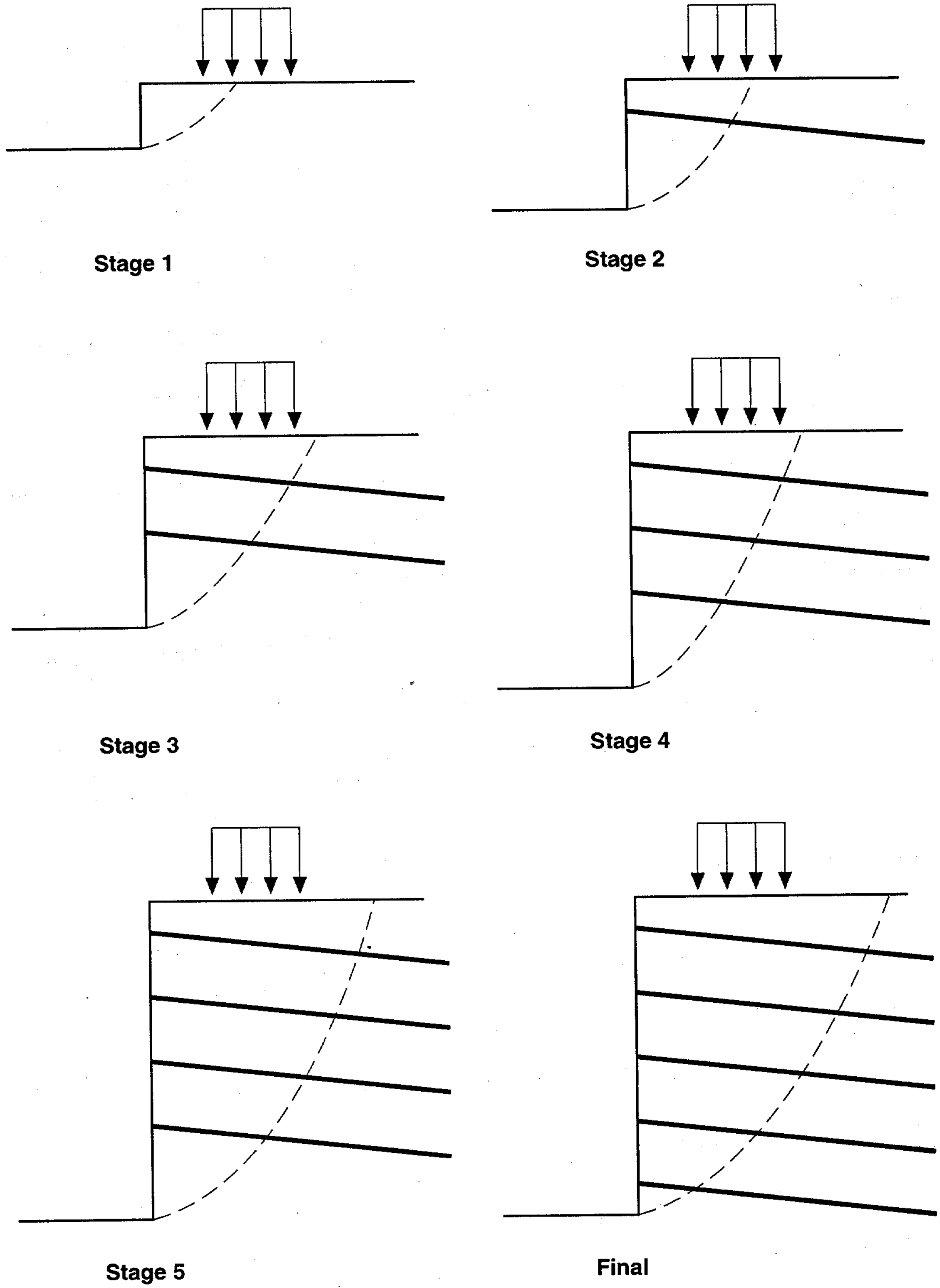


Figure 4.18 Stability Assessments for Construction Conditions

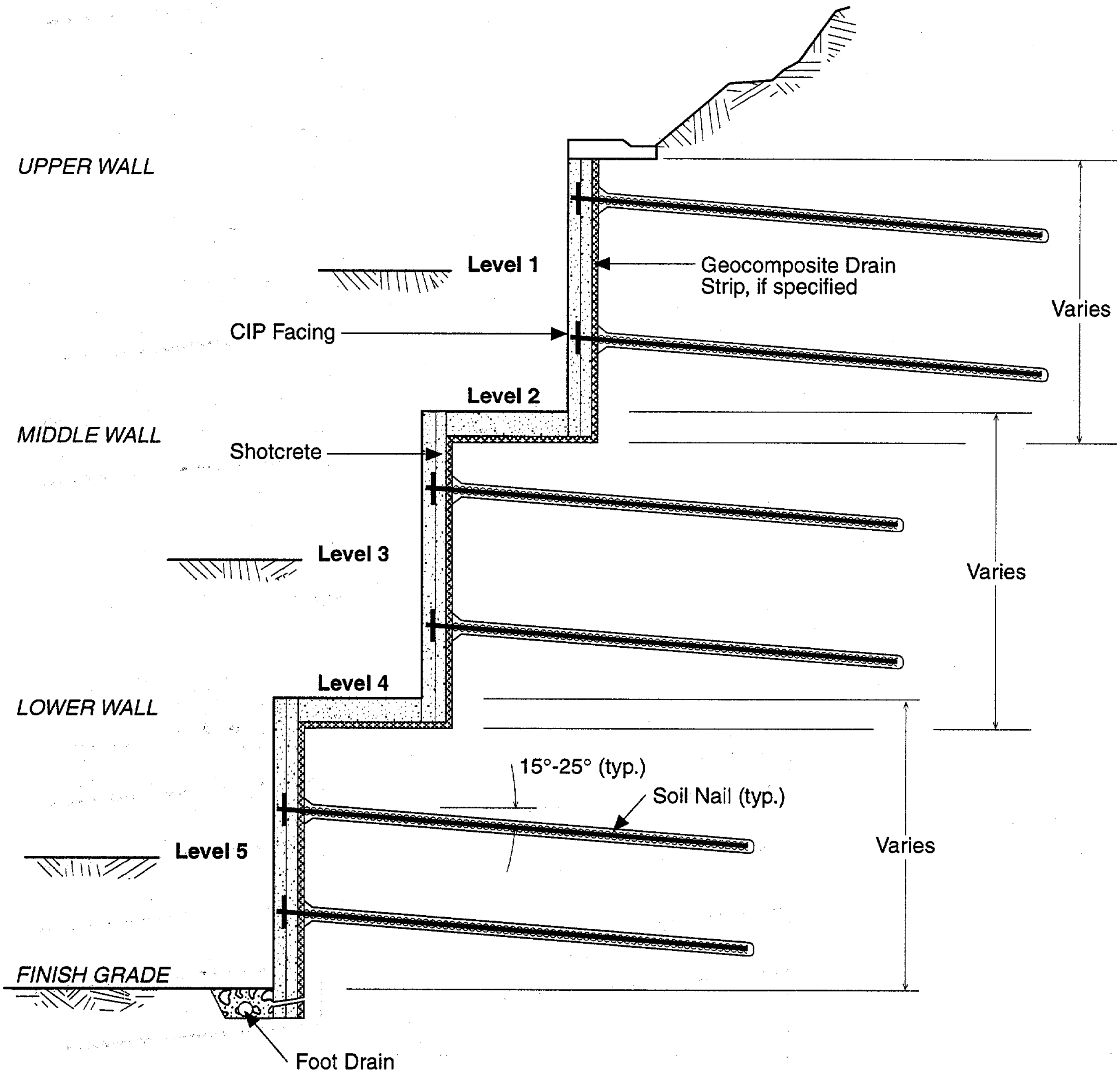
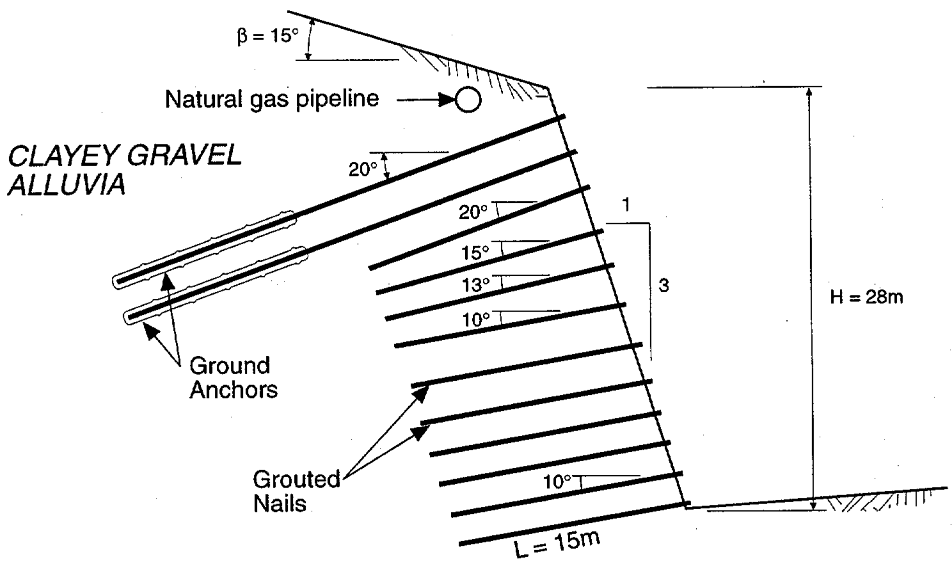
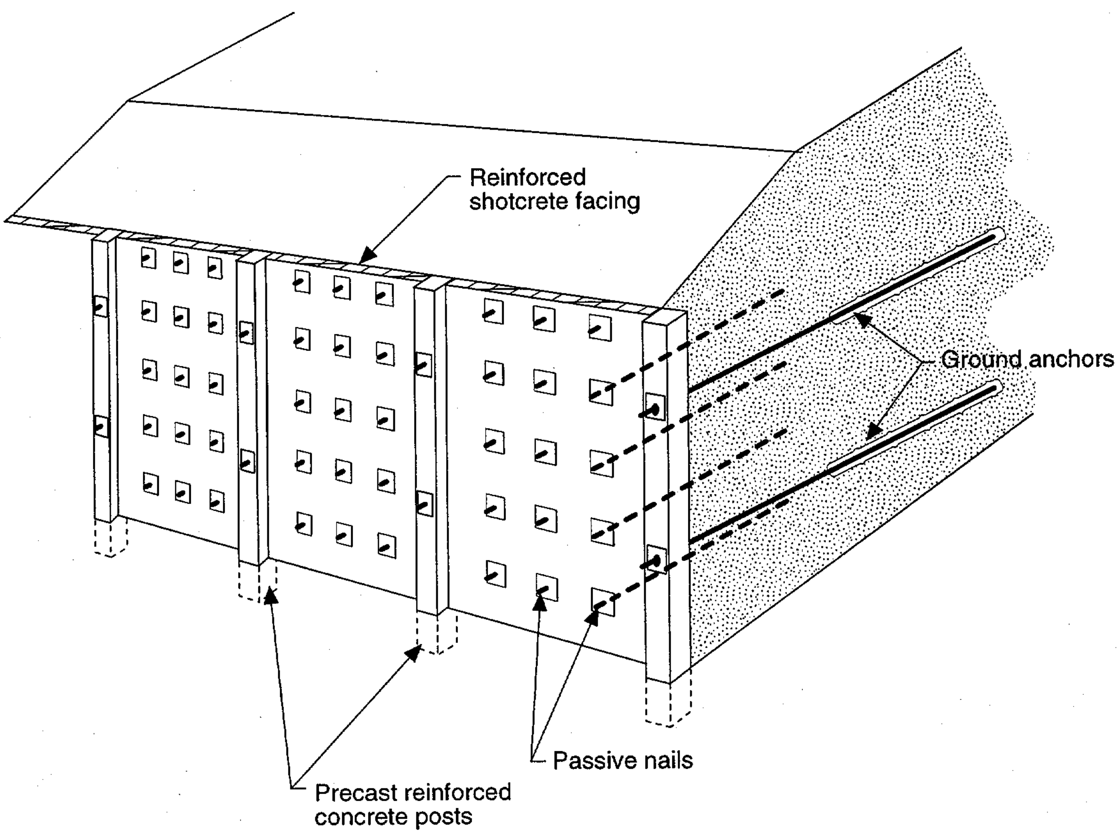


Figure 4.19 Typical Section Through Stepped Wall



(a) The Wall at the North Entrance of the Cotière Tunnel (TGV Rhône-Alpes, 1990)



(b) Nailed Berlin Wall

Figure 4.20 Composite Wall Structures

support systems that have been used and demonstrates fundamentally different support concepts requiring different design approaches.

Figure 4.20(a) shows a composite nail-tieback support system applied at the north entrance to the Cotiere Tunnel [10]. In this application, a conventional soil nail support system is complemented with tiebacks in the upper part of the wall to provide additional stability against deep-seated failures and to limit displacements in the upper wall area in order to protect a nearby critical structure. Problems with deep-seated stability can occur where significant backslopes exist and material strengths are modest. The recommended design approach for this type of application is to use the method outlined herein for the soil nail portion of the support system, but to limit the slip surfaces considered so that the deep-seated slips are not addressed (e.g., limit slip surfaces to those that intersect the ground surface at a distance of no greater than 1.5 times the proposed nail length behind the top of the wall). Nail lengths will typically not be less than are required for good soil conditions and a horizontal backslope. The soil nail reinforced zone can then be considered to act as a gravity retaining structure and the tiebacks then designed to provide the additional support required to stabilize more deep-seated slips and prevent overturning of the wall. The grouted tieback anchorages should be placed behind both the nailed zone and the most critical deep-seated slip surface affecting the whole structure.

Figure 4.20(b) shows an alternative nail-tieback support system (nailed Berlin wall), where the primary support comes from the tiebacks and short nails are used to provide more localized face stability during construction. In the application shown on figure 4.20(b), the nails also make it possible to increase the distance between the soldier piles by reducing the bending moments in the facing.

(b) Supporting Other Wall Types

A fundamentally different type of composite structure from those discussed above is an earth retaining structure such as an MSE wall or conventional retaining wall constructed on top of the soil nail wall following completion of the nailing (figure 4.21). In these applications, the nail-reinforced ground comprises the foundation for the upper structure, and it is recommended that the bearing pressures beneath both the upper retaining wall (taking account of vertical loads, horizontal loads, and overturning moments) and its retained earth be considered as surface surcharge loads for the design of the soil nail portion of the structure.

4.10.6 Structures with Variable Nail lengths

There are no design computation restrictions to considering nails of variable length, although service load monitoring indicates that wall displacements will be minimized if nail lengths are kept relatively uniform, particularly in the upper two-thirds to three-quarters of the wall height. Such a nail length distribution should therefore be the objective of the design. In the lower part of the wall it is often permissible to shorten the length of the nails since the soil-nail interaction in this region is such as to induce maximum nail loads closer to the head of the nail.

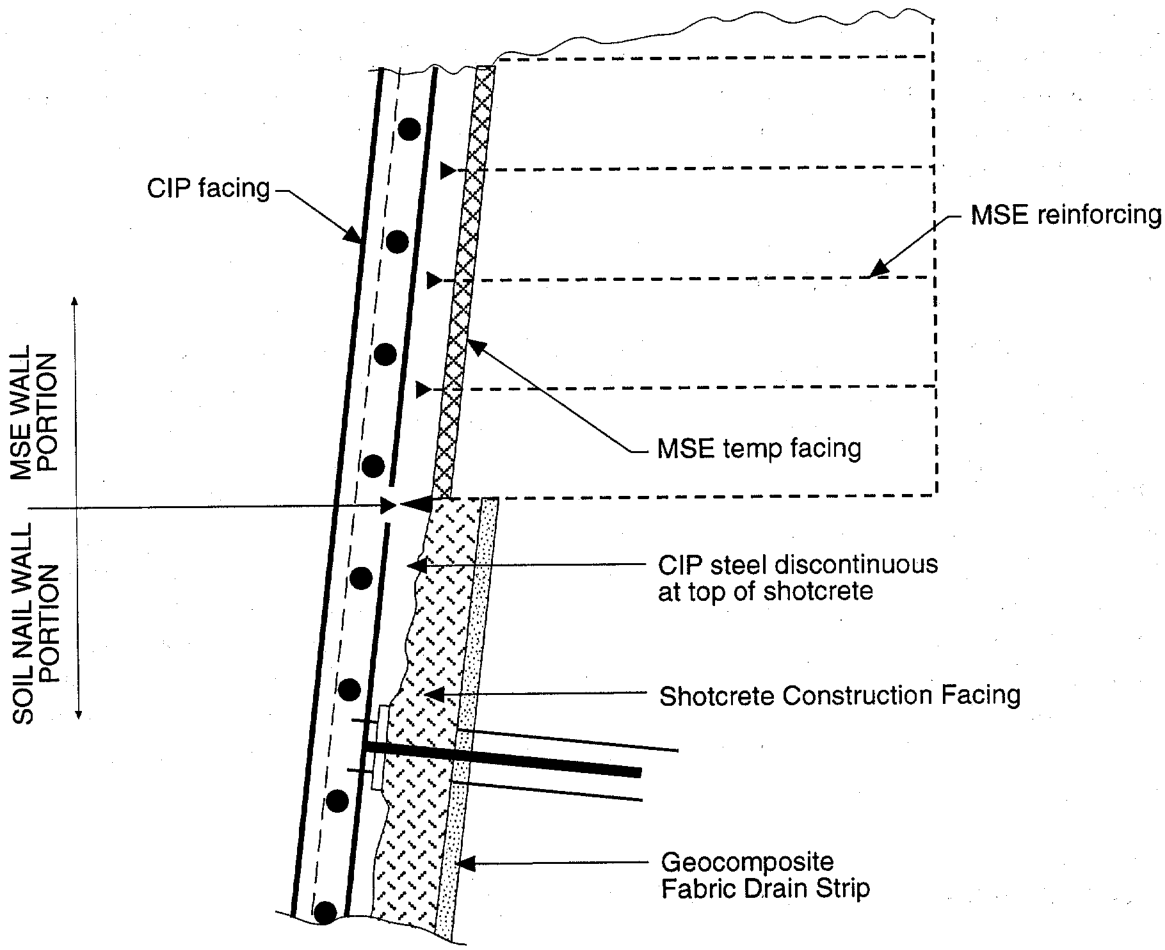


Figure 4.21 Example of Soil Nail Wall Supporting MSE Wall (Oregon DOT Portland, OR Light Rail Project)

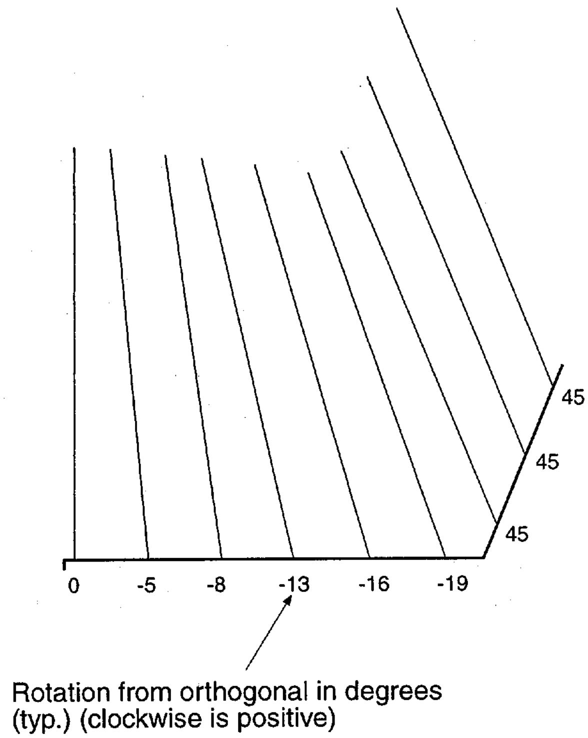
It is often necessary to place restrictions on the length of the upper row of nails at specific locations to avoid interference with utilities, for example. The nails must be sufficiently long, however, to provide for local stability of upper wedges of the ground. Restricting the length of upper nails will limit, or virtually eliminate, their contribution to the stability of larger zones of ground associated with the overall wall height and will also reduce their ability to control surface displacements. Since the top-of-wall deformations will be reduced by having full length reinforcement within the upper part of the wall, it is desirable to minimize the use of "short" nails in the upper rows to the extent possible. However, use of short nails in the top row of soil nail walls is relatively common and has been successfully applied, particularly in urban environments, to avoid utilities.

4.10.7 Structures with Variable Nail Inclinations

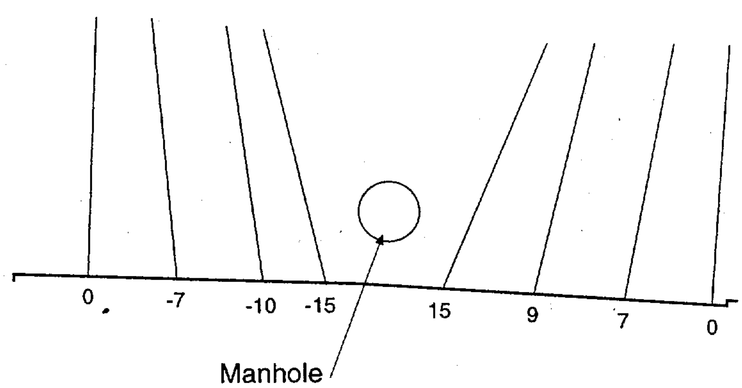
There are no design computation restrictions in the consideration of nail patterns with variable nail inclinations. From the perspectives of improving the reinforcing efficiency of the nails and also of limiting wall displacements, however, it is desirable to install the nails as close to horizontal as possible. There should, therefore, be no general incentive for installing nails more steeply than about 15 degrees below horizontal, which is the typical minimum declination required to enable grouting of the nail holes under gravity or low pressure. As discussed above for variable nail lengths, it is often necessary to steepen the inclination of the upper row of nails somewhat (e.g., to 20-25 degrees) to provide for utility clearance. Steeply inclined nails may need to be offset from vertical row alignment to prevent interference with lower rows. When nailing under bridge decks, it is sometimes necessary to install the top row of nails at an inclination of less than 15 degrees because of limited clearance for the drill rig mast beneath the deck. Special grouting procedures will generally be required with shallow nail inclinations to ensure proper installation of the nail.

4.10.8 Structures with Variable Nail Orientations

Nails will generally be installed in a vertical plane that is normal to the wall facing. For planar walls or for walls with inside curvature, this approach should be followed. For walls containing outside curvature, however, it will often be necessary to install nails in a splayed pattern that is not normal to the wall facing because of the problem of adjacent columns of nails interfering with each other. An example of this situation is shown on figure 4.22. In such instances, it is recommended that the soil nail wall design analyses be performed as though the nails were installed in vertical planes normal to the wall facing. The splayed nails actually installed should then be of sufficient length to extend to the depth normal to the facing as required by the analysis. Similarly, the component of the installed nail strength in a direction normal to the facing should equal or exceed that required from the design analysis.



Splay Nail Layout
Exterior Corner



Splay Nail Layout
Manhole Clearance

Figure 4.22 Splay Nails

4.10.9 Ground Water Seepage Forces or Water Table Close to Wall Base

As with slope stability problems, ground water and the associated seepage forces can significantly affect the stability of a soil nail structure. As discussed in section 2.2, the use of soil nailing in situations below the ground water table is not generally recommended. There may be special circumstances where consideration of seepage forces is required, such as when an unanticipated ground water table is encountered above the wall base. Most of the problems associated with the presence of ground water and the associated seepage forces are related to constructability, and these issues must be considered by personnel experienced with such conditions. From a design perspective, however, the inclusion of seepage forces in the stability analysis poses no special difficulty and most of the computer design models currently available permit the inclusion of water seepage pressures in the analysis.

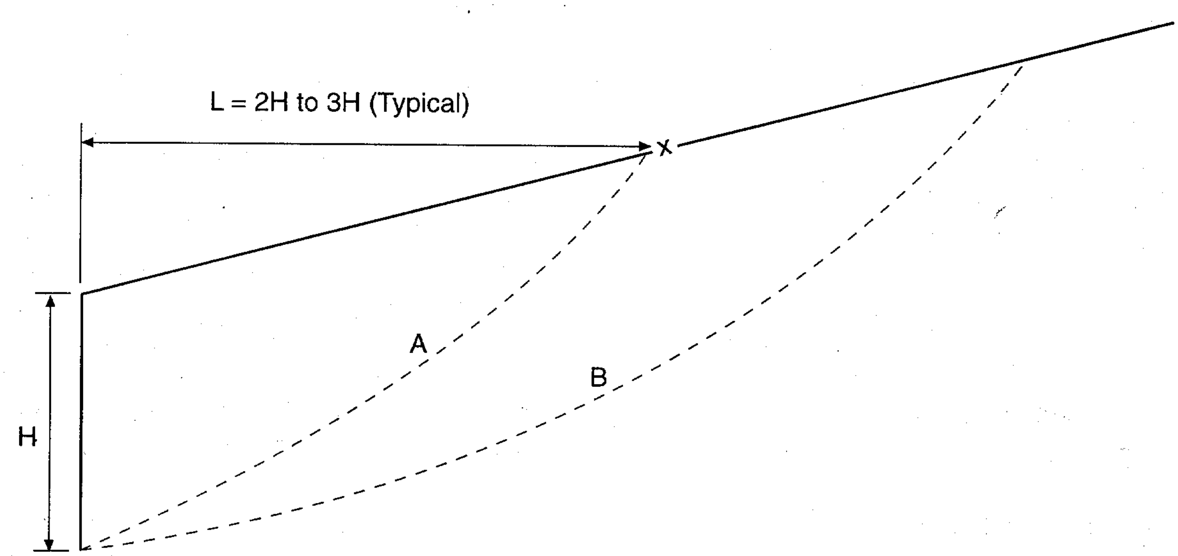
As noted in Step 8 (sections 4.7.1 and 4.7.2), a ground water table that is close to the base of the wall will tend to promote more deep-seated instabilities that pass beneath the base of the wall and exit downslope or in front of the toe of the wall. This condition must be checked for in design.

4.10.10 Infinite Slope Condition

In instances where a soil nail retaining wall is installed at the toe of a long ("infinite") slope, and that slope has a calculated factor of safety (e.g., 1.25) less than that recommended for the retaining wall itself (e.g., 1.35), then the design requirements may have to be modified. The reason: if potential slip surfaces considered in the design of the reinforced soil nail wall are unrestricted, then very large failure zones that encompass essentially the entire slope will be considered. The analysis may therefore indicate that very long, high capacity nails are required with the nails being installed in the toe region of an essentially "infinite" slope. This is generally considered to be an inappropriate use for such a retaining system. Under these circumstances, it is recommended that 1) the overall stability of the slope be independently determined and modified by other methods, if necessary, and 2) the soil nail wall be designed by limiting the scope of potential slip surfaces to the immediate area of the wall itself e.g., to within typically two to three times the height of the cut from the top of the proposed wall. This recommendation is shown graphically on figure 4.23.

4.10.11 Performance Under Seismic Loading

Recent experience has demonstrated that soil nail walls perform well under seismic loading. A number of observations of the performance of soil nail walls was made for the October 17, 1989 Magnitude 7.1 Loma Prieta earthquake in California. A post-earthquake report [4], presents field observations on eight soil nail walls in existence in the San Francisco Bay area during the earthquake. The walls, varying in height between 2.7 meters and 9.8 meters, were the subject of detailed post-earthquake visual inspections. In some cases, nails were retested after the earthquake.



Slip Surface Type A – Limited to slip surfaces that exit the surface at L≤ 2H to 3H behind wall. Conventional safety factors required.

Slip Surface Type B - More deep-seated slip surfaces. Required safety factors consistent with overall slope stability requirements.

Figure 4.23 "Infinite" Slope Conditions - Design Approach

None of the walls showed signs of distress even though one of them was located in Santa Cruz, an area that experienced significant seismic related damage. A 4.6-meter-high wall located on the University of California Santa Cruz campus approximately 18 km from the earthquake epicenter experienced a horizontal ground acceleration estimated at 0.47 g. Soil conditions at the site consist of a hard clayey sandy silt. Construction of this wall was completed less than three weeks before the earthquake. Prior to the earthquake, some wall footings had also been poured at the bottom of the excavation immediately in front of the wall. The post-earthquake inspection revealed significant cracking of the concrete footings. This cracking was not attributed to shrinkage, since foundations constructed after the earthquake showed fewer cracks. Inspection of the soil nail wall revealed no cracking or other distress. Subsequent pullout testing of nine nails to 150 percent of their design load also indicated no loss of pullout capacity due to the seismic activity.

A UCLA research project co-sponsored by NSF and contractor Kulchin-Condin & Associates has also recently been performed to study in more detail the behavior of soil nail walls during earthquake loading [40]. The study included model centrifuge testing of test walls with nail length to wall height ratios ranging from 0.33 to 1.0. Tests simulating static loading indicated that for nail lengths commonly used in practice (i.e., length to height ratios greater than about 0.67), the static deformations at the top of the wall were on the order of a few tenths of one percent of the wall height. Under simulated earthquake loading, the soil nail walls performed exceedingly well. For nails of appropriate length, many cycles of shaking with peak accelerations to 0.45 g were required to induce excessive deformations of the wall. It was concluded that typical soil nail structures with grouted nails should have the capacity to resist large earthquakes, confirming the field observations from the Loma Prieta earthquake.

Both field and laboratory investigations have demonstrated the generally robust performance of soil nail walls under relatively severe earthquake shaking. For design purposes, therefore, seismic loading effects should be addressed as recommended in section 4.7 and as shown in sections 5.1.1.2 and 5.2.1.2 (in appendix G) of the example problems. These recommendations will generally require that additional design capacity (e.g., nail length or strength) beyond that required from static design considerations may be needed only for relatively severe peak ground accelerations (i.e., greater than about 0.25 g to 0.30 g).

4.10.12 Frost Protection

The formation of ice lenses in the vicinity of the soil nail wall facing in frost-susceptible soils has been reported in a few cases. This has led to the development of high loads on both the facing and the head of the nail, because of the fully bonded nature of the nail and its inability to tolerate large strains in the adjacent soil without developing correspondingly high loads in the nail. This phenomenon has resulted in damage to the facing. In situations where the facing is very resistant, damage can occur to either the nail or to the connection between the nail and the facing.

The magnitude of the facing/nail loads developed will depend on the depth of frost penetration, the intensity and duration of the freeze period, and the availability of water. Increases in nail and facing loads should be anticipated in areas where frost durations are generally greater than one week and where there are frost susceptible soils near the face and in close proximity to a source of water. Frost loading effects may be eliminated or mitigated by the use of porous backfill (used on a few projects to date) or insulating material placed either between the shotcrete construction facing and the CIP or precast panel final facing, or outside the permanent concrete facing. Figure 4.24 shows proposed frost protection details using styrofoam insulation. A 1-inch thickness of styrofoam insulation board is generally considered to be equivalent to 1-foot thickness of gravel. Styrofoam insulation board has been used to insulate some permanent tieback wall facings (personal communication from D. Weatherby, Schnabel Foundation Co.). At the time of final preparation of this manual, the Maine and New Hampshire DOT Details shown on figure 4.24 are the first known soil nail walls in U.S. using insulation board for facing frost protection. Use should be considered experimental at this time.

4.10.13 Expansive Soils

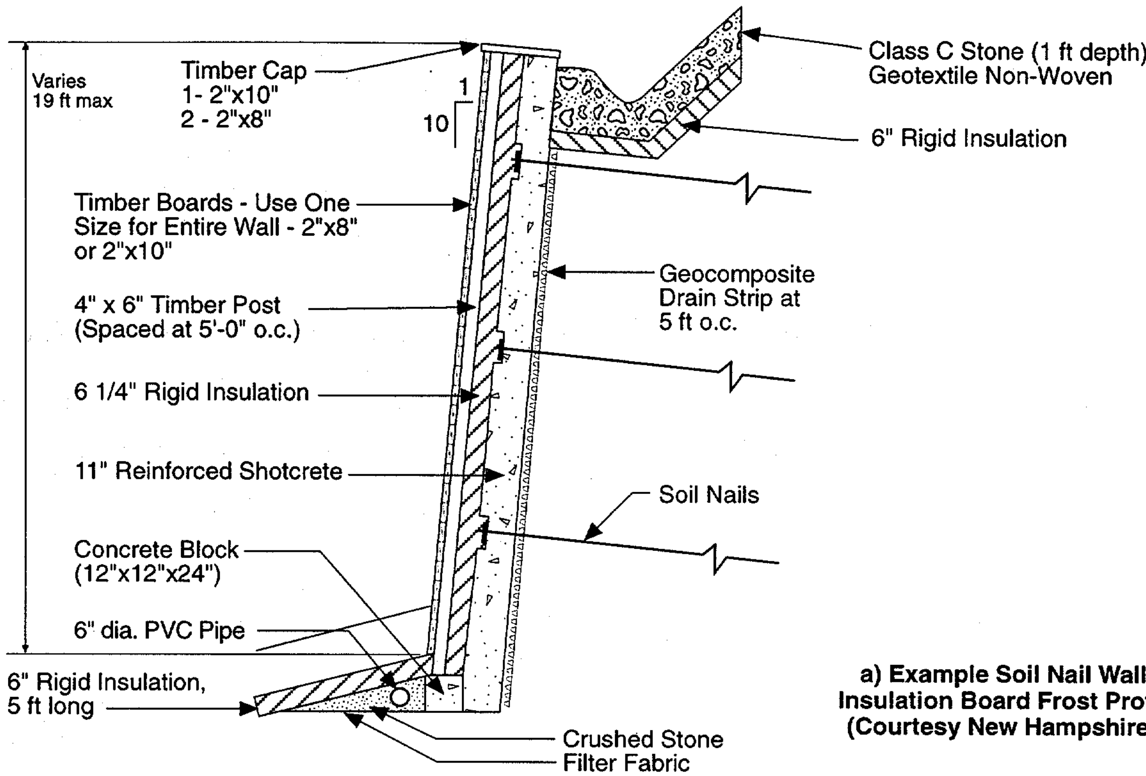
At present, there are no established design procedures for soil nail walls in highly expansive soils, and this is generally considered a non-application for permanent walls. However, soil nail walls may have application in soils that are expansive in nature, particularly for temporary shoring applications, provided measures are taken to inhibit significant changes in moisture content of the soils during the life of the structure. Under such conditions, a conventional design approach such as recommended herein can be adopted.

4.10.14 Residual Soils

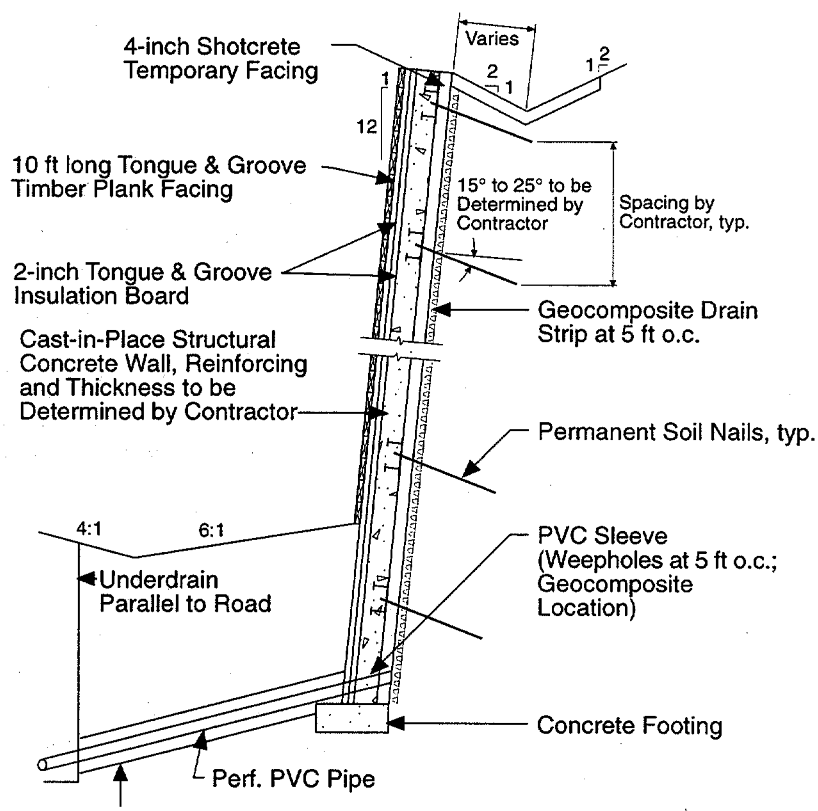
Residual soils will often exhibit specific slip surfaces, defined by relict structure, with shear strength characteristics that are significantly lower than those that apply to the ground mass in general. For example, two joint sets may combine to form potentially unstable wedges dipping out of the face. Provided it can be demonstrated that the relict structure will exhibit sufficient stand-up time to enable safe and economic construction of a soil nail wall, the design procedure is essentially identical to that previously discussed. However, since all potential failure modes must be considered, the analyses must address both general or non structurally controlled slip surfaces in association with the strength of the ground mass, together with specific structurally controlled slip surfaces in association with the strength characteristics of the relict joint surfaces themselves. The soil nail reinforcement must then be configured to support the most critical of these two conditions.

4.10.15 Structures with Externally Loaded Wall Facings

The nails and permanent facing of a soil nail retaining wall may be required to support external loads, and associated shear forces and bending moments, applied directly to the facing, in addition to those developed by the nail-ground-facing interactions during construction of the wall. Most commonly, these external loads will be applied at the top of the wall facing and may vary from relatively light highway appurtenance loads (e.g., roadway lighting supports) to much more significant loads associated, for example, with the integration of a relatively large cantilever retaining structure on top of the wall. An example of the latter type of external load



a) Example Soil Nail Wall with Insulation Board Frost Protection (Courtesy New Hampshire DOT)



b) Example Design/Build Soil Nail Wall With Insulation Board Frost Protection (Courtesy Maine DOT) Experimental Features Project

Figure 4.24 Frost Protection

application is shown in figure 4.25. For relatively light loading conditions, the external loads may be used to define statically-determinate additional shear forces and bending moments in the cantilevered section of the wall above the first row of nails, together with additional loads in the top row of nails themselves. For more significant loads, it may be necessary to perform a full soil-structure interaction analysis to define how the additional facing and nail loads are distributed throughout the entire soil nail structure.

A common case where external loads should be considered is the design of soil nail walls in front of bridge abutments. The loads transferred to the wall for both structures supported on piles and structures supported on spread footings will depend on the distance between the wall and the foundation elements. These loads can be significant for structures that are subject to seismic forces. The example problems in chapter 5 contains detailed analyses for both the static (5.1.1.2, step 9) and seismic loading (5.2.1.2 in appendix G, step 9) of a soil nail wall subjected to loads from a pile supported abutment.

4.10.16 Design of Support for Facing Dead Load

As noted in section 2.4.7, the weight of the construction facing must be supported by the installed nails or other supplementary means until compressive stresses develop at the facing-soil contact and the facing can be supported by bond and shear (frictional) loads developed at the contact. This is particularly important for the facing of the initial excavation lifts, where it is not possible to "hang" the next facing layer from the overlying facing previously installed. For typical construction facings consisting of 100-mm-thick shotcrete, experience has shown that the soil nails will support the weight of the facing in direct shear and bearing. For thicker applied shotcrete facings, the support for the facing self weight by direct shear of the nails and bearing beneath the nails should be formally evaluated. The maximum thickness of shotcrete facing that can be supported in the manner is dependent on the nature of the soils and, in competent ground, shotcrete facings up to 200 to 250 mm thick have been successfully supported.

Where necessary, support of the dead weight load of soil nail wall facings may be achieved by the installation of additional short, steeply inclined reinforcing elements acting as compression struts. Strut nails act in compression while the soil nails act in tension to develop a truss that supports the facing dead load. The recommended design approach is presented below, with reference to figure 4.26.

Figure 4.26 shows a soil nail/strut nail system supporting a facing panel of self weight W. The soil nail has a conventional headed stud connection to the facing. The strut nail has a bearing plate and washer connection system, as shown. Figure 4.26 also shows the method for calculating the service loads (associated with self weight support of the facing) within both the soil nail and the strut nail. The design criteria are given below for both SLD and LRFD.

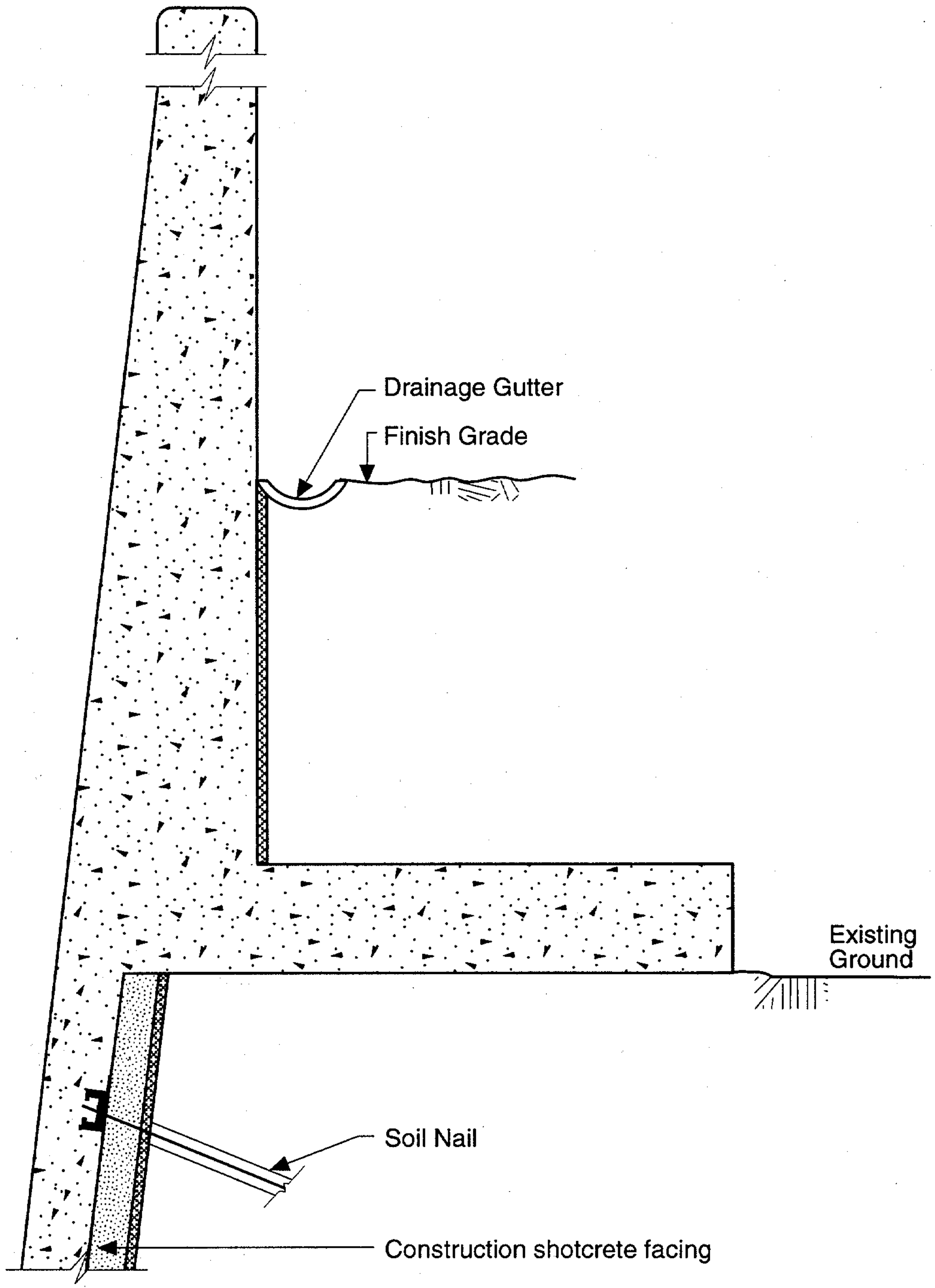
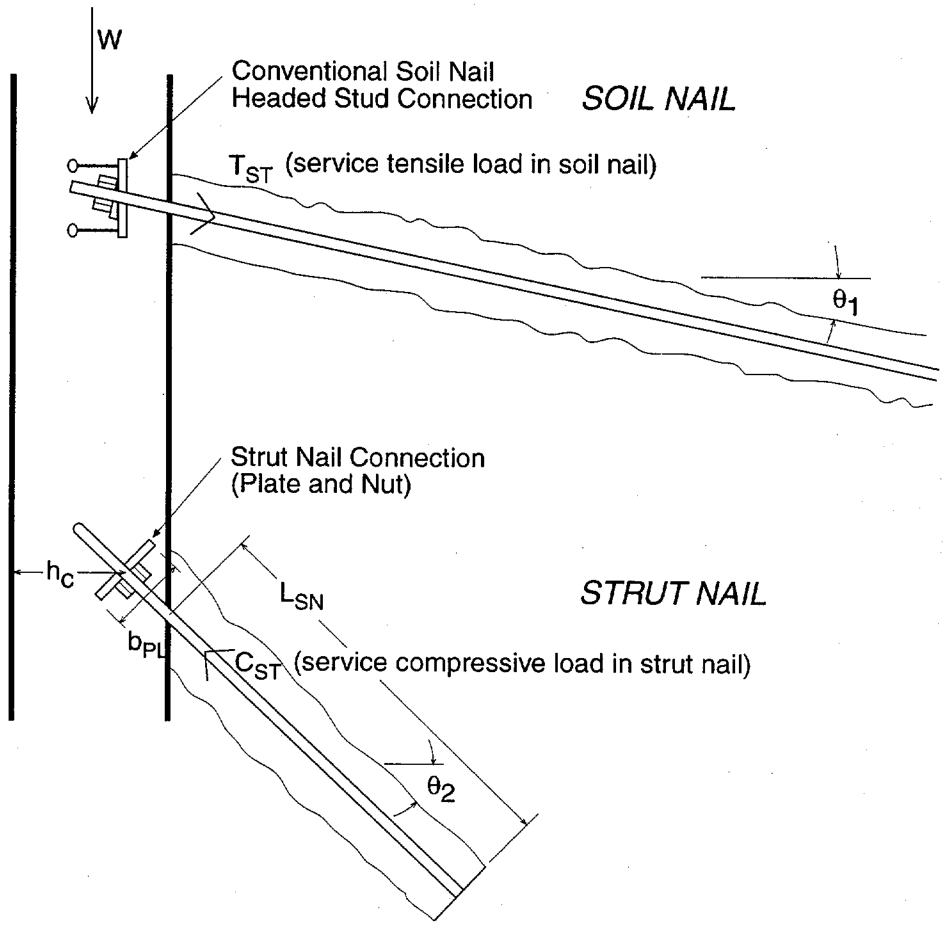
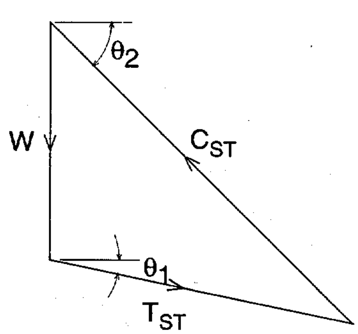


Figure 4.25 Example Cantilever Wall Section Detail





NOTE:

C_{ST} and T_{ST} determined from consideration of horizontal and vertical equilibrium

For soil nails inclined at 15° and strut nails at 45°;

T_{ST} = 1.4 W

C_{ST} = 1.9 W

where W = weight of facing supported by soil nail and strut nail

Figure 4.26 Strut Nail Concept

SLD

Soil Nail

$$T_{ST} <= \min \left[(0.67)\, C_F\, (m_{V,NEG} + m_{V,POS}) \left(\frac{8\, S_H}{S_V} \right)^{(a)}; (0.67)\, V_N \left(\frac{1}{1 - C_S(A_C - A_{GC})/(S_V S_H - A_{GC})} \right)^{(b)}; (0.50^{(1)})(4A_{HS}F_U)^{(c)}; (0.55)T_{NN}{}^{(d)} \right]$$

Strut Nail

$$C_{ST} <= \min[(0.35)(0.80)(0.85f'_C(A_{SN} - A_{ST}) + F_Y A_{ST})^{(e)}; (0.67)V_N{}^{(f)}; (0.50)Q_U L_{SN}{}^{(g)}]$$

LRFD

Soil Nail

$$\Gamma^{(h)} T_{ST} <= \min \left[(0.90)\, C_F\, (m_{V,NEG} + m_{V,POS}) \left(\frac{8\, S_H}{S_V} \right)^{(a)}; (0.90)\, V_N \left(\frac{1}{1 - C_S(A_C - A_{GC})/(S_V S_H - A_{GC})} \right)^{(b)}; (0.67^{(1)})(4A_{HS}F_U)^{(c)}; (0.90)T_{NN}{}^{(d)} \right]$$

Strut Nail

$$\Gamma^{(h)} C_{ST} <= \min[(0.75^{(i)})(0.80)(0.85f'_C(A_{SN} - A_{ST}) + F_Y A_{ST})^{(e)}; (0.90)V_N{}^{(f)}; (0.70)Q_U L_{SN}{}^{(g)}]$$

Notes:

(1) Assumes ASTM A307 bolt material for headed studs
(a) Reference equation 4.1, for facing flexure failure mode, and table 4.4 (strength factor for SLD) or table 4.7 (resistance factor for LRFD)
(b) Reference equations 4.2 and 4.3, for facing punching shear failure mode, and table 4.4 (strength factor for SLD) or table 4.7 (resistance factor for LRFD)
(c) Reference equation 4.4, for headed stud fracture failure mode, and table 4.4 (strength factor for SLD) or table 4.7 (resistance factor for LRFD)
(d) Reference table 4.5 (SLD) or table 4.8 (LRFD), for nail tendon tensile strength.
(e) Reference section 8.15.4 of AASHTO (15th Edition, 1992) [30], for compression member capacity (SLD) or equation 5.7.4.4-3 of AASHTO (1st Edition, 1994) [29], for factored axial resistance of reinforced concrete compressive components (LRFD).
(f) Reference equation 4.2, for the nominal internal punching shear strength of the facing, with $D'_C = b_{PL} + h_C$, where b_{PL} and h_C are defined for the strut nail on figure 4.26, and table 4.4 (strength factor for SLD) or table 4.7 (resistance factor for LRFD).
(g) Reference table 4.5 (SLD) or table 4.8 (LRFD) for ground-grout pullout resistance.
(h) Load factor Γ equals 1.25 in accordance with table 4.6 for load type DC.
(i) Reference section 5.5.4.2.1 of AASHTO (1st Edition, 1994) [29], for resistance factor for axial compression.

In general, the soil nail and its connection to the facing will be more than adequate to provide the required tensile support. For the strut nail, the nail compressive strength and punching shear at the nail head will generally be adequate to provide the required compressive load support, and the strut nail must be made sufficiently long to prevent bond failure at the nail grout-ground interface.

4.10.17 End of Wall Transitions

Where the soil nail wall will transition to another wall type (e.g., soil nail wall transitioning to a temporary excavation slope and conventional concrete cantilever wall), the plans and specifications should include a requirement for temporary shoring, typically installed perpendicular to the wall face at the end of the wall transition.

4.10.18 CIP Concrete Form Connection to Shotcrete Facing

It is relatively common practice to use a one-sided form, attached to the temporary shotcrete construction facing, for placement of the permanent CIP wall facing. The form ties are typically anchored to the shotcrete facing using either mechanical or epoxied anchorages. It is necessary to demonstrate that the anchor capacity is adequate to support the design concrete fluid pressures developed within the forms. The design fluid pressures are determined using empirical procedures provided in relevant standard specifications and will depend, in part, on the rate at which the level of the concrete pour is raised.

The anchorage capacity must consider the potential for punching shear and flexural failure of the shotcrete facing, as discussed in section 4.5 for the connection of the nail head to the facing. There is no specific testing database to provide guidance for the design of such anchorage systems, as most available manufacturer's literature appears to be based on pullout mechanisms that do not reflect the adverse influence of flexural cracking behavior that would be expected to occur with a thin shotcrete facing. Therefore, the following approach for determining the form tie anchorage capacity is recommended when one-sided forms are used for construction of the CIP facing:

- For facings with surface areas of up to 500 m^2, a minimum of three anchor capacity tests should be performed in which the anchor is located at the approximate mid-point location between two nails and the nail locations are used as the reaction points. The anchor should be loaded to a minimum of 1.5 times the required design load (calculated as the design concrete fluid pressure times the anchor tributary area). This test will evaluate minimum anchor capacity under combined shear and flexural loading conditions. For wall areas in excess of 500 m^2, one additional test should be performed for each additional 250 m^2 of wall surface area.

- As an additional quality control measure, local punching shear or pullout type tests should also be performed on two (2) percent of the installed form tie anchorages. This test provides a less critical test of anchorage capacity than that described above, because the effect of wall flexure is not included. The anchors should be loaded to a minimum of

2.0 times the required design load, and the tensile load reaction support location should be no closer to the edge of the anchor than the depth of embedment of the anchor into the construction facing. This arrangement will permit punching shear type failures of the anchor to occur without interference from the test load reaction support.

4.11 Aesthetic Issues

For most long-term soil nail structures associated with highway construction, the structure appearance is an important consideration. Architectural considerations will often play a role in selecting the overall configuration of the retaining wall, including the face batter and the use of a stepped or terraced geometry. Smooth line top of wall profiles provide a more pleasing appearance than several short abrupt up and down straight line segments. In addition, a variety of architectural finishes may be applied to the permanent facing, and soil nailing is extremely adaptable in this regard. Architectural face treatments are commonly applied to CIP facings through the use of commercially available form liners, boards, or other materials placed inside the face forms. Coloring may also be applied by painting with a pigmented sealer. This architectural treatment versatility, combined with the proven long-term durability and performance of CIP concrete, make it the most common choice for permanent facings on U.S. highway projects. Face treatment can also be applied to permanent shotcrete facings by hand finishing and texturing (by experienced structural shotcreters/finishers), and coloring can be done by painting, staining, or adding coloring agent to the shotcrete mix. In some cases, architectural aesthetic requirements will call for a non-structural fascia to cover a permanent shotcrete or CIP facing. Such architectural fascias may consist of precast concrete panels, masonry stone, or masonry block. Example of architectural finishes that have been used with soil nail walls are shown on photos in the FHWA Soil Nailing Field Inspector's Manual [26].

4.12 Simplified Design Charts for Preliminary Design of Cut Slope Walls

The final design of the nail length will involve an iterative process as described in section 4.7 and demonstrated in the example problem in section 5.1. However simplified design charts have been developed for a 15° nail inclination, uniform ground conditions, and non-critical installations assuming a safety factor F of 1.35 for SLD and a resistance factor Φ of 0.9 for LRFD. These preliminary estimates of nail length and strength requirements can be used to roughly estimate right of way requirements, general nail costs, or as a starting value for more detailed interactive analyses. The preliminary design charts in figures 4.27 through 4.34 are presented in dimensionless format. The following variables are required to be determined before entering the charts:

<u>Geometric Variables of Backslope Angle, β, and Face or Batter Angle, δ</u>

Four sets of design charts are presented (three charts per set), with each set of charts corresponding to a single backslope angle of 0, 10, 20, or 34 degrees. For intermediate backslope angles, interpolate between the charts. For each backslope angle, design information is presented for two face batter angles of 0 and 10 degrees from the vertical. For intermediate face batter angles, interpolate between the charts.

<u>Strength Variables - Factored Friction Angle, ϕ_D, and Dimensionless Cohesion, c_D</u>

The factored friction angle of the soil, ϕ_D, is defined by the following relationship:

<u>SLD</u>

$$\phi_D = \tan^{-1}[\tan(\phi_U)/F] \tag{4.0a}$$

<u>LRFD</u>

$$\phi_D = \tan^{-1}[\Phi_\phi \tan(\phi_U)] \tag{4.0b}$$

Where: ϕ_U = Ultimate soil cohesion
F = Global soil factor of safety (use 1.35 for preliminary estimate)
Φ_ϕ = Soil friction resistance factor (use 0.9 for preliminary estimate).

The factored friction angle is shown on the horizontal axis of chart A of each chart set.

The dimensionless cohesion, c_D, is the soil cohesion normalized with respect to the soil unit weight and the vertical height of the cut:

<u>SLD</u>

$$c_D = c_U/(F\gamma H) \tag{4.0c}$$

LRFD

$$c_D = \Phi_C\, c_U/(\Gamma_W \gamma H) \tag{4.0d}$$

Where: C_u = ultimate soil cohesion
F = global soil factor of safety (use 1.35 for preliminary estimate)
Φ_C = soil cohesion resistance factor.

The dimensionless cohesion is shown as a parameter for each slope geometry for three values of 0.01, 0.03, and 0.05. Interpolate for intermediate values of the dimensionless cohesion.

Dimensionless Nail Tensile Capacity, T_D

Find the dimensionless nail tensile capacity, T_D, by entering the vertical axis of the first chart of the appropriate chart set.

The dimensionless nail tensile capacity is the factored nominal nail strength normalized with respect to the soil unit weight, γ; the vertical height of the slope, H_l; and the nail spacings, S_V, S_H:

SLD

$$T_D = \alpha_N T_{NN} /(\gamma H S_V S_H) \tag{4.0e}$$

LRFD

$$T_D = \Phi_N T_{NN} /(\Gamma_W \gamma H S_V S_H) \tag{4.0f}$$

Where: T_{NN} = required nominal nail strength
α_N = nail tendon strength factor
Φ_N = nail tendon resistance factor
Γ_W = load factor for unit weight

Preliminary Nail Size

Find A_N (nail cross sectional area) = T_{NN}/F_y and enter table 4.8 to find bar size for F_y 420 bars.

TABLE 4.8A
BAR SIZES (ENGLISH AND SOFT METRIC*)

Bar Designation No.	Nominal Diameter, in. [mm]	Nominal Area, in^2 [mm^2]
3 [10]	0.375 [9.6]	0.11 [71]
4 [13]	0.500 [12.7]	0.20 [129]
5 [16]	0.625 [15.9]	0.31 [199]
6 [19]	0.750 [19.1]	0.44 [284]
7 [22]	0.875 [22.2]	0.60 [387]
8 [25]	1.000 [25.4]	0.79 [510]
9 [29]	1.128 [28.7]	1.00 [645]
10 [32]	1.270 [32.3]	1.27 [819]
11 [36]	1.410 [35.8]	1.56 [1006]
14 [43]	1.693 [43.0]	2.25 [1452]
18 [57]	2.257 [57.3]	4.00 [2581]

* Soft metric bar designation numbers, nominal diameters and areas are the values enclosed within brackets. Bar designation numbers approximate the number of millimeters of the nominal diameter of the bar.

The dimensionless pullout resistance, Q_D, is the factored ultimate pullout resistance (expressed as a force per unit length of nail), normalized with respect to the soil unit weight and nail spacings:

SLD

$$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) \quad (5.4a)$$

LRFD

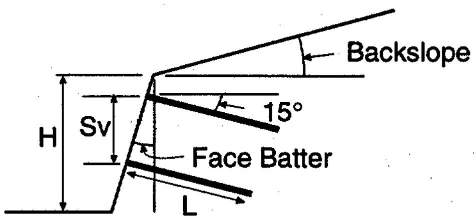
$$Q_D = \Phi_Q Q_U/(\Gamma_W \gamma S_V S_H) \quad (5.4b)$$

Where: Q_U = ultimate pullout resistance (found in section 3.2)
α_Q = strength factor (for preliminary estimate assume to be 0.5)
Φ_Q = resistance factor (for preliminary estimate assume to be 0.7)

Find dimensionless pullout resistance is shown as being incorporated into the ratio (T_D/Q_D) on the horizontal axis of the second and third charts or each chart set.

Preliminary Nail Length

Compute Q_D and the ratio T_D/Q_D. Enter either chart 2 or 3 of the appropriate chart set to find the ratio L/H. Since H is known, compute the preliminary nail length, L.



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$
$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$
$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{(LRFD)}$$

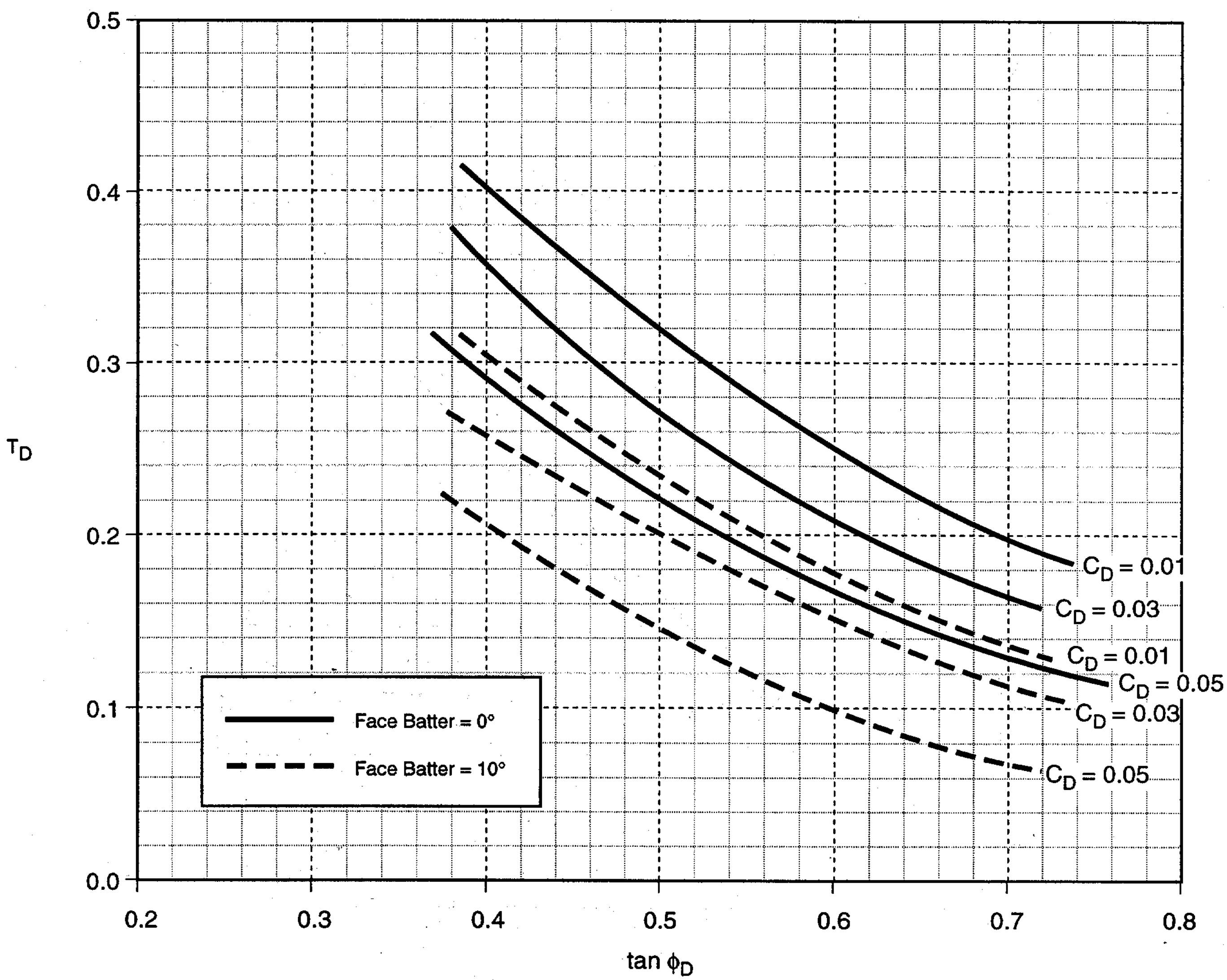
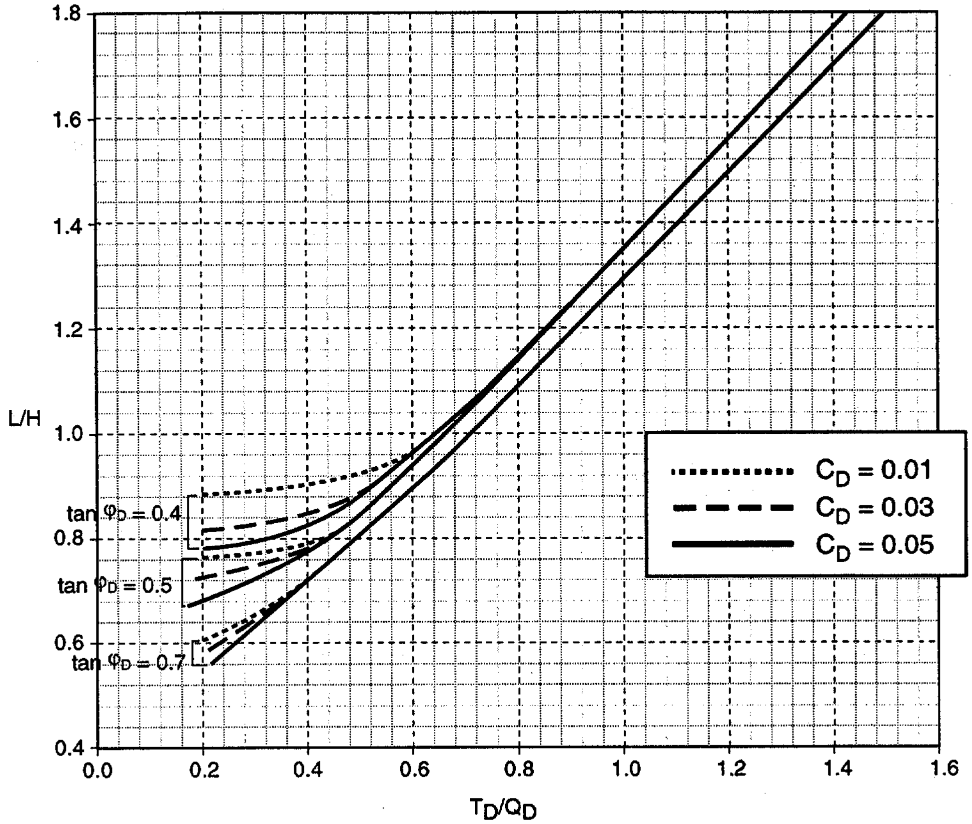
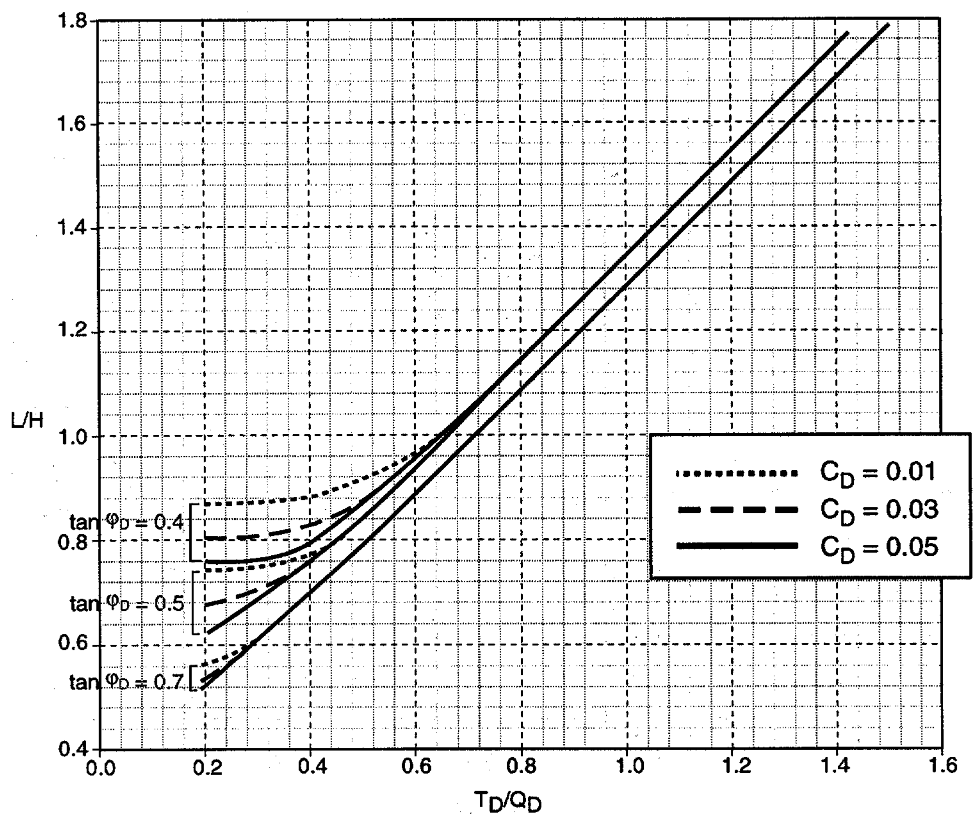


Figure 4.27 Preliminary Design Chart 1A
Backslope = 0°

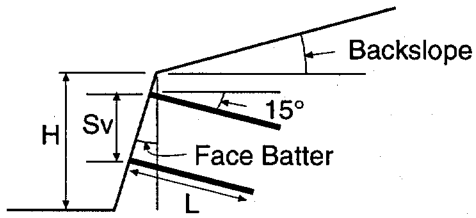


Backslope = 0° Face Batter = 0° (Chart 1B)



Backslope = 0° Face Batter = 10° (Chart 1C)

Figure 4.28 Preliminary Design Charts 1B and 1C



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$
$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$
$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{(LRFD)}$$

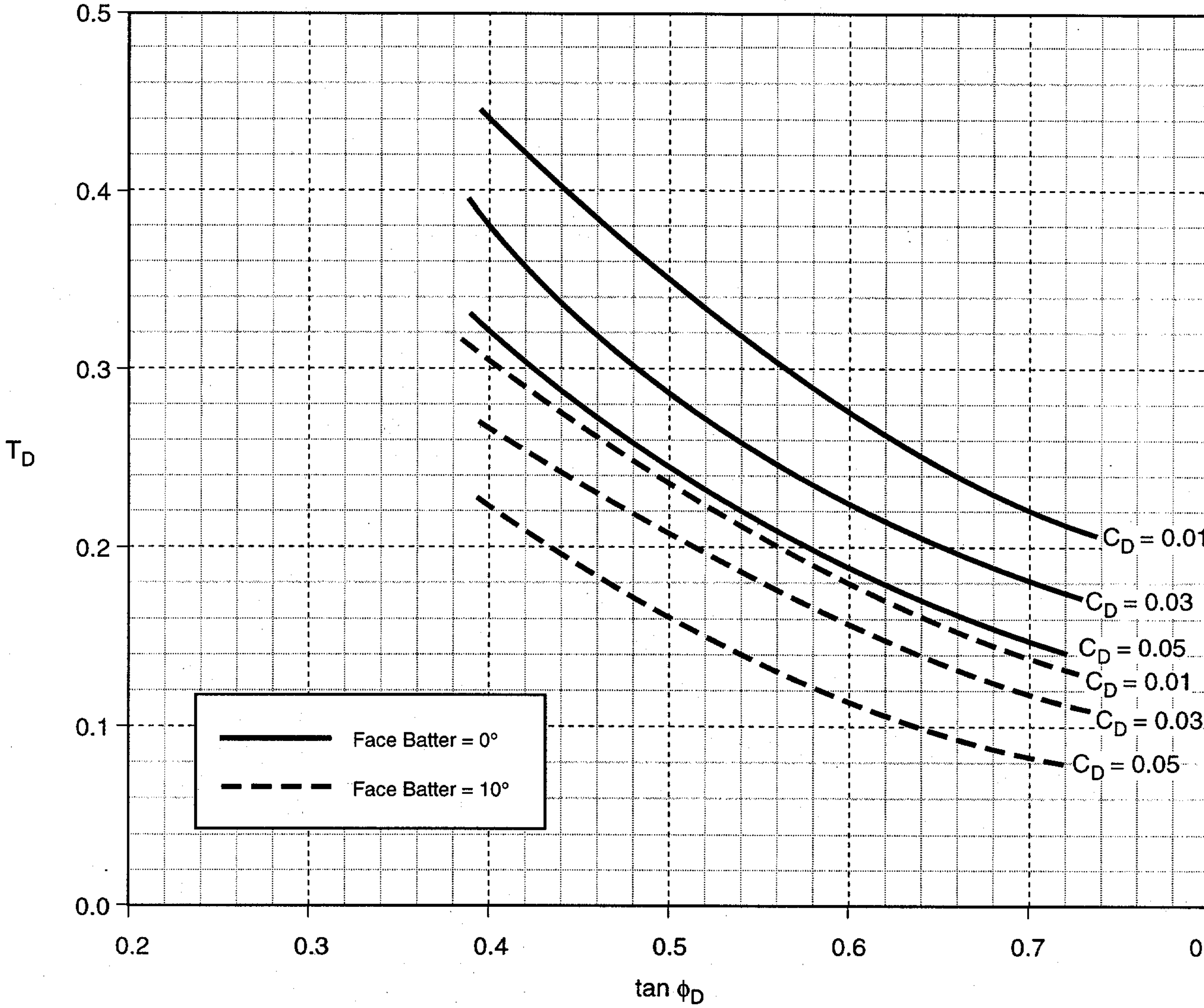
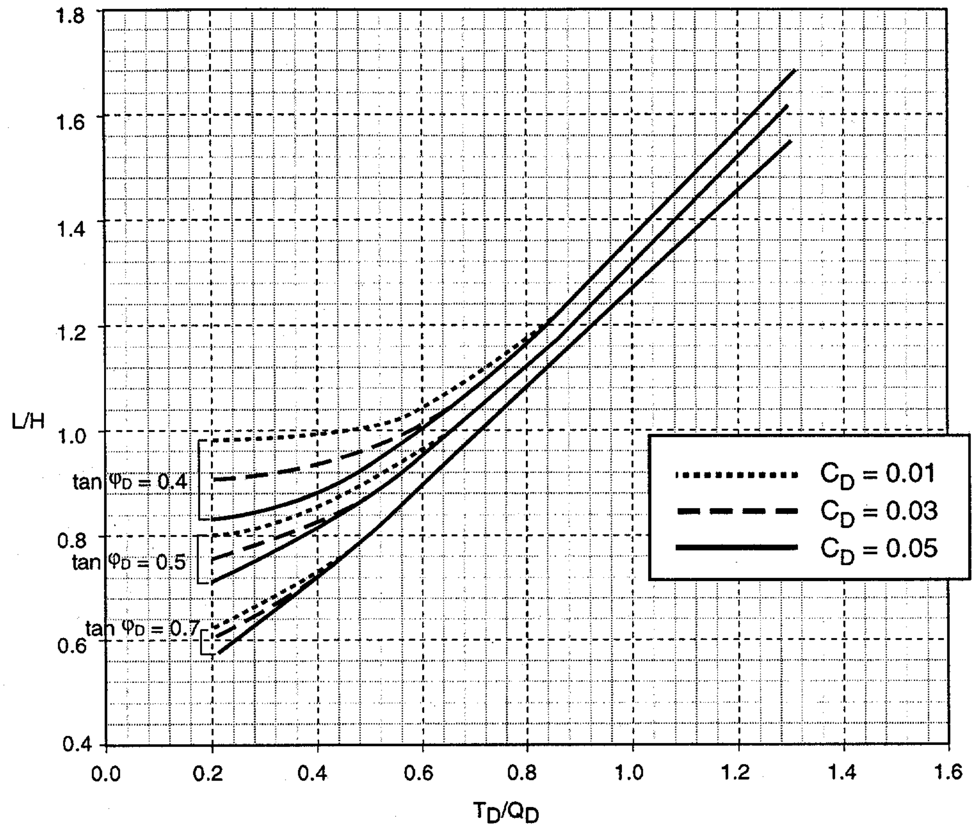
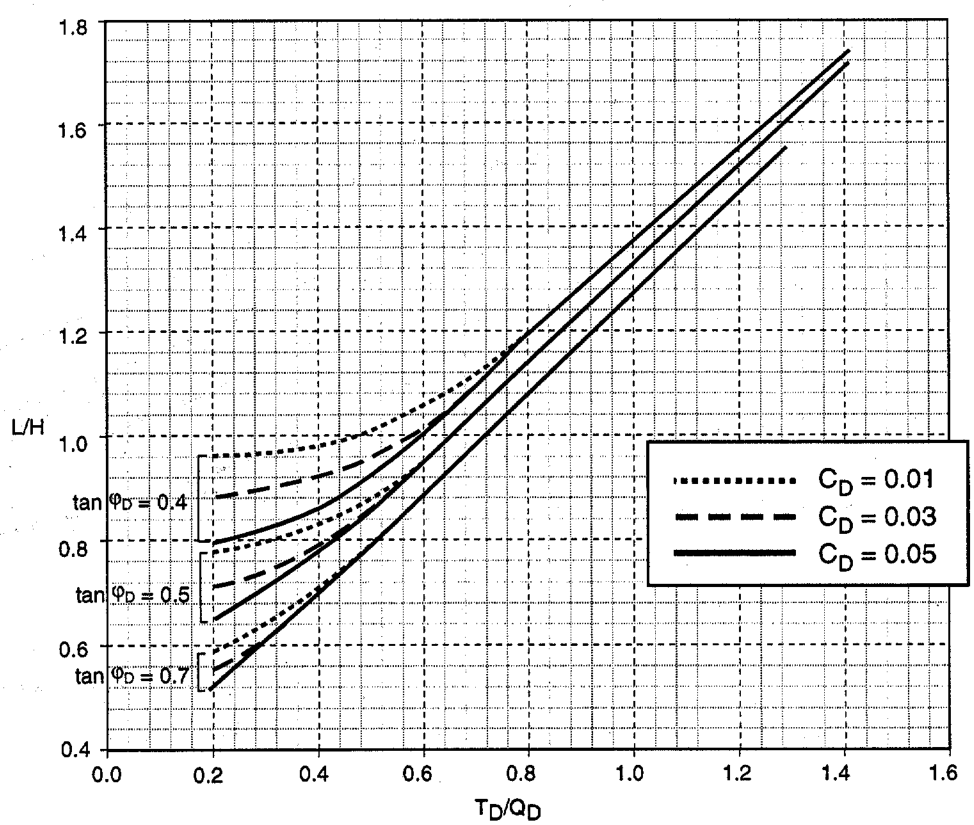


Figure 4.29 Preliminary Design Chart 2A
Backslope = 10°

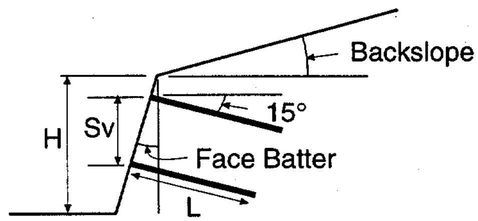


Backslope = 10° Face Batter = 0° (Chart 2B)



Backslope = 10° Face Batter = 10° (Chart 2C)

Figure 4.30 Preliminary Design Charts 2B and 2C



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$

$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$

$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{ (LRFD)}$$

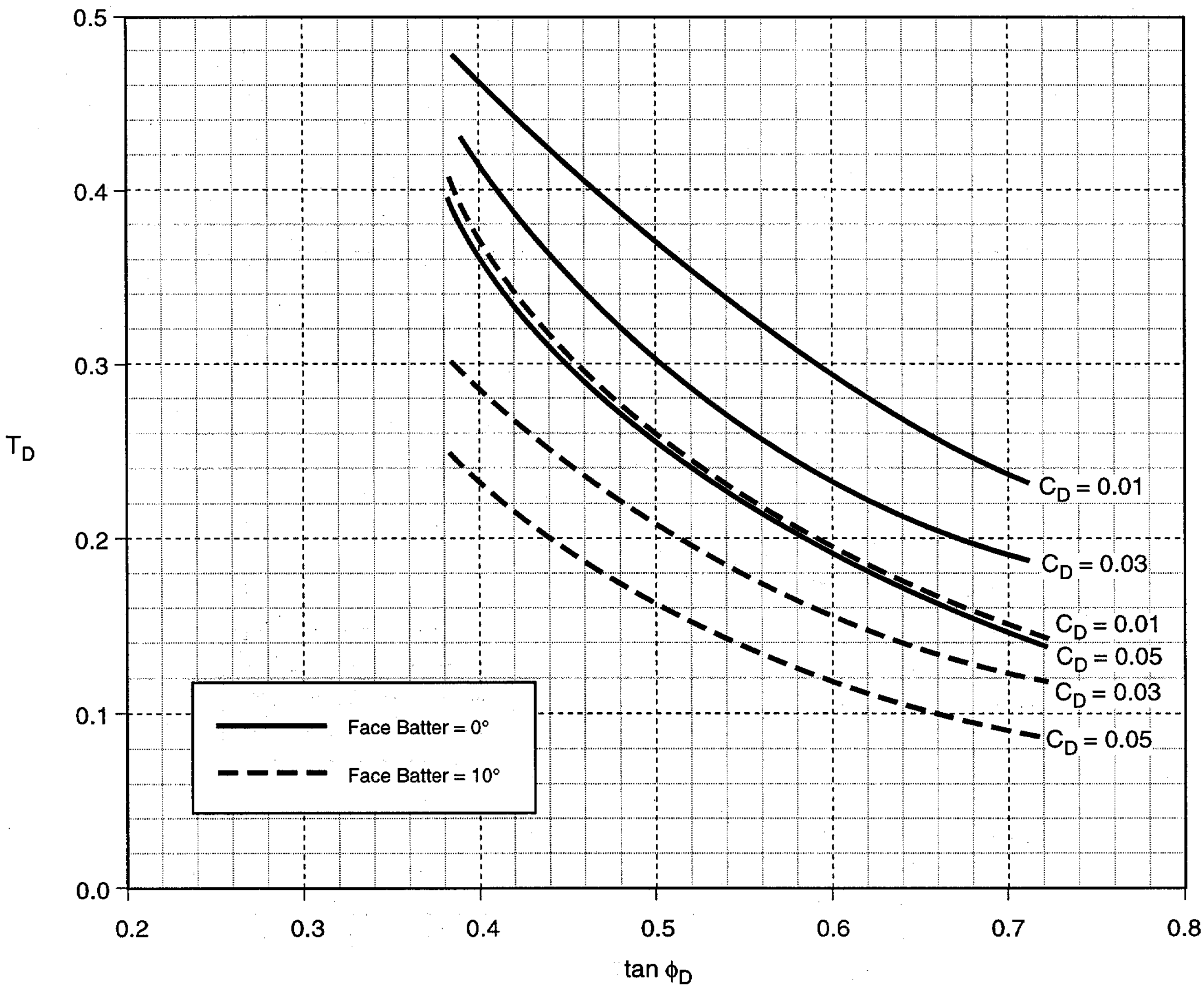
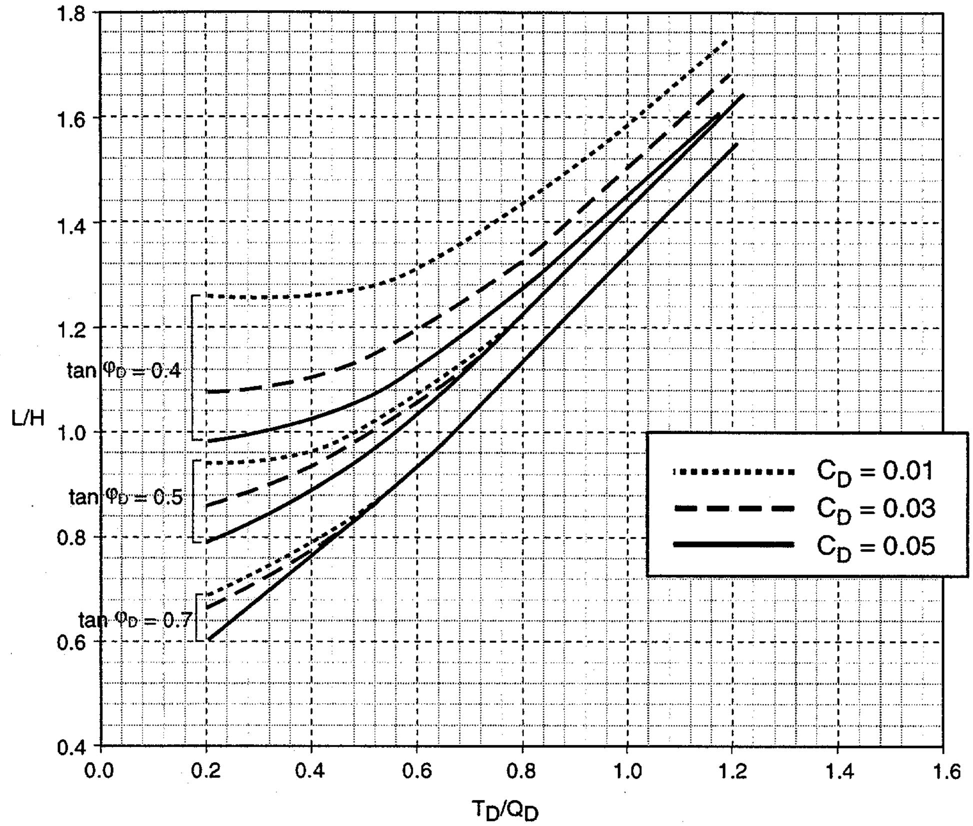
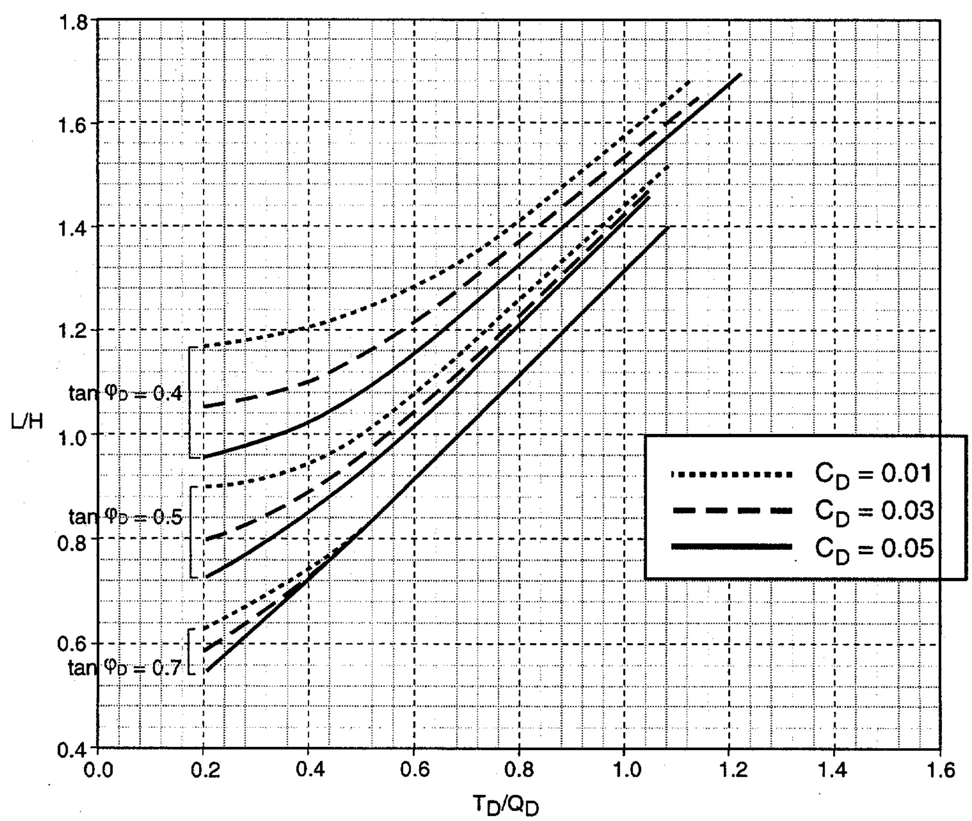


Figure 4.31 Preliminary Design Chart 3A
Backslope = 20°

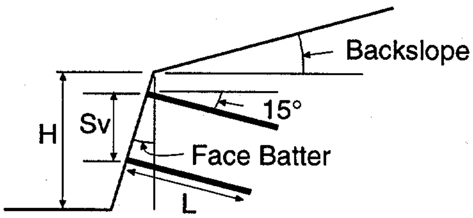


Backslope = 20° Face Batter = 0° (Chart 3B)



Backslope = 20° Face Batter = 10° (Chart 3C)

Figure 4.32 Preliminary Design Charts 3B and 3C



$$C_D = c_u/(F_c\gamma H) \quad \text{(SLD)}$$

$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$

$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{ (LRFD)}$$

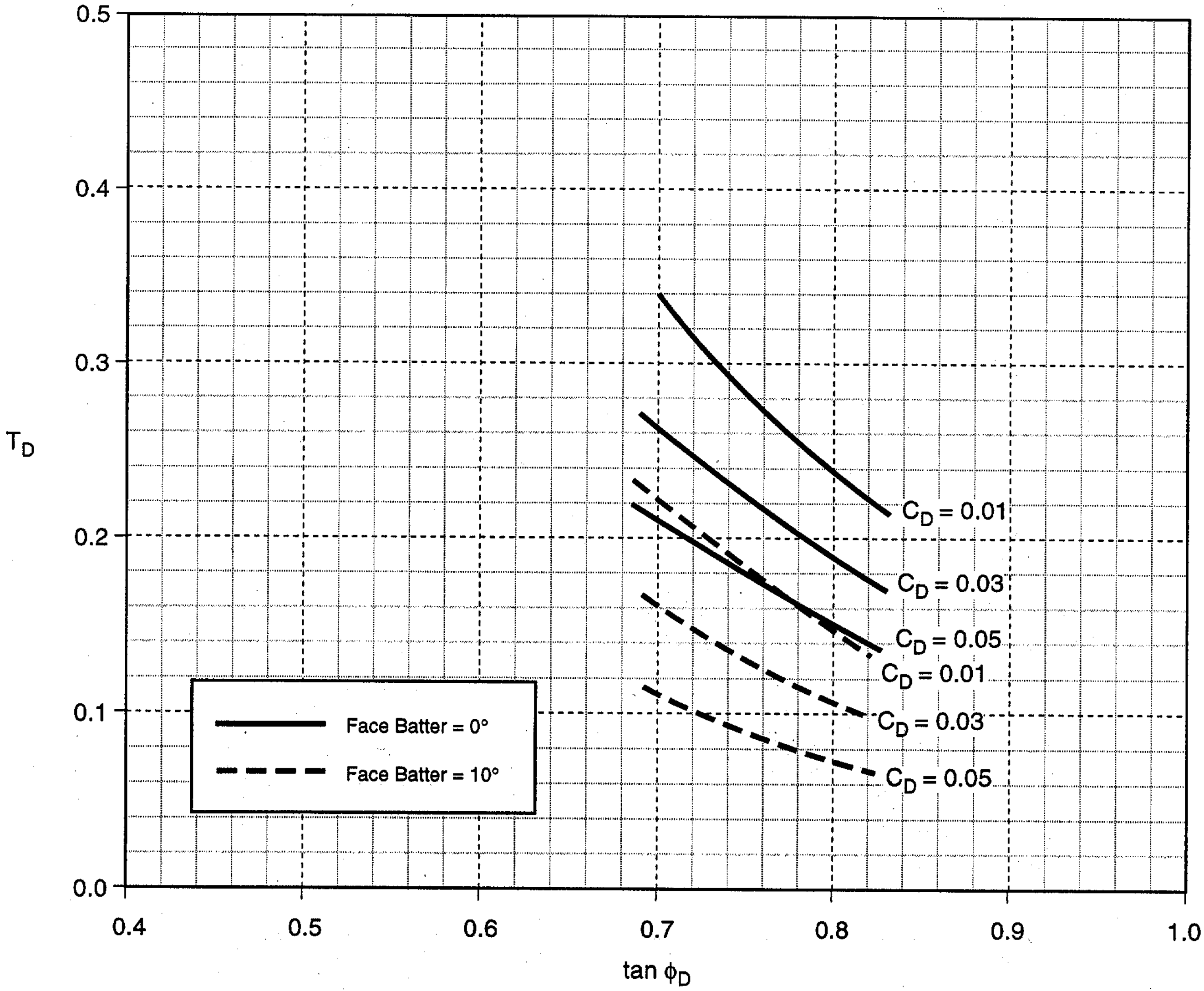
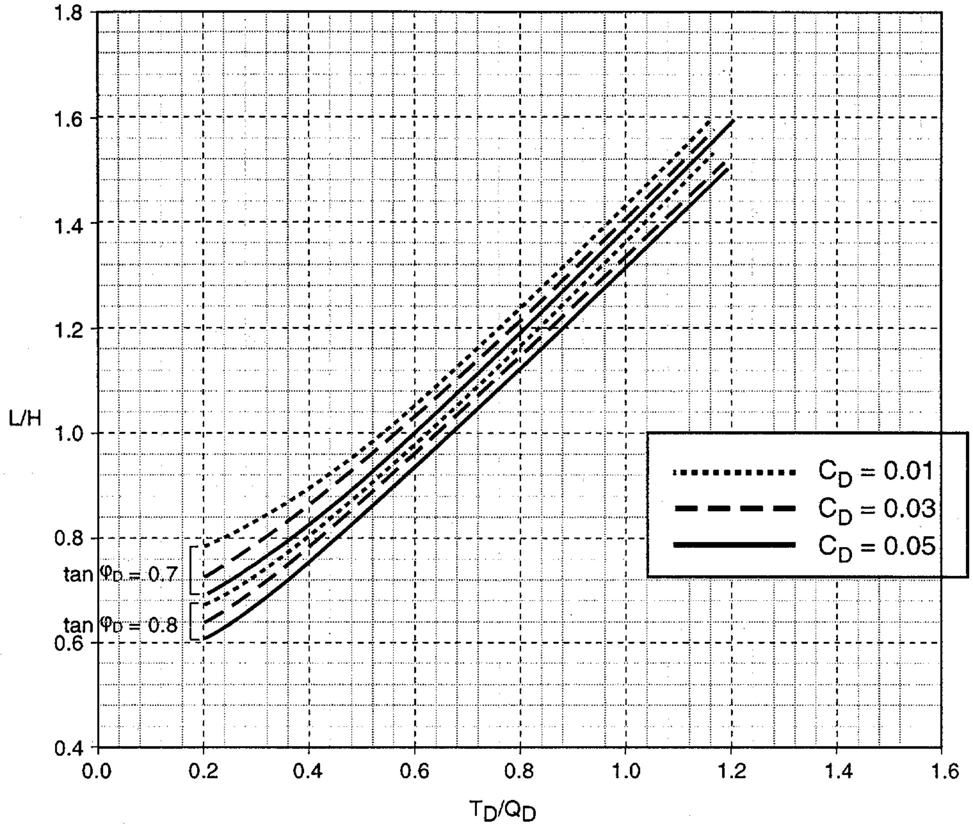
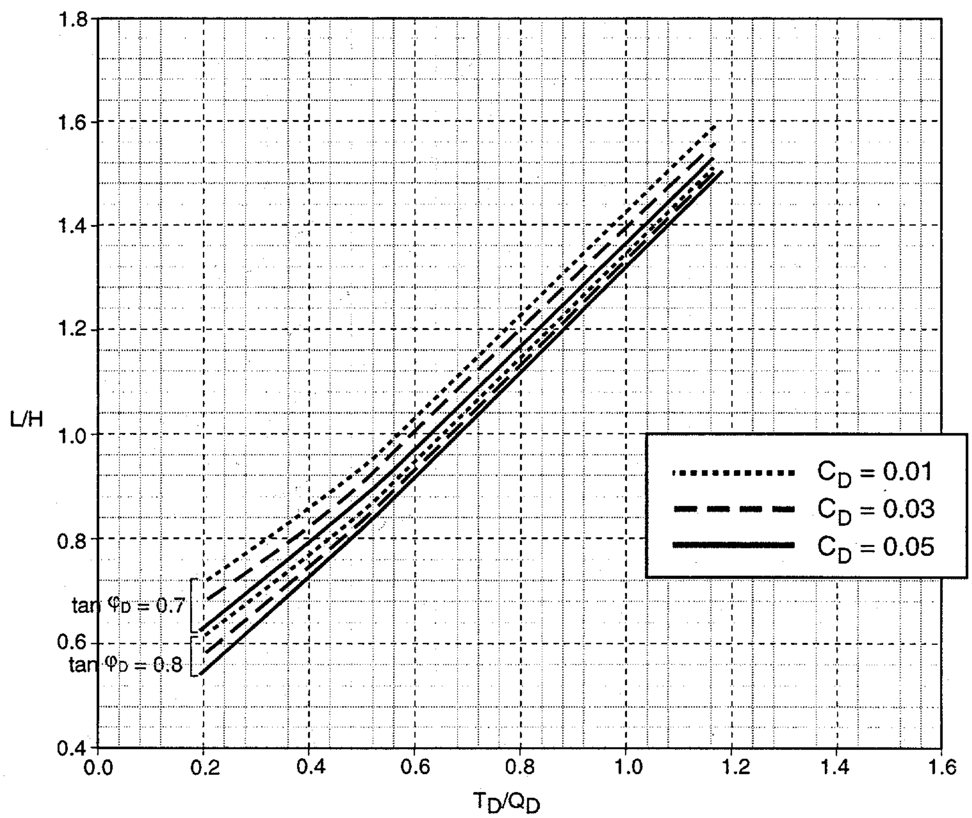


Figure 4.33 Preliminary Design Chart 4A
Backslope = 34°



Backslope = 34° Face Batter = 0° (Chart 4B)



Backslope = 34° Face Batter = 10° (Chart 4C)

Figure 4.34 Preliminary Design Charts 4B and 4C

CHAPTER 5. WORKED DESIGN EXAMPLES

In this chapter, the design method in section 4.7 for soil nail walls is summarized and demonstrated by the example of a common highway applications of this technology: a cutslope wall. A second example of a retaining wall associated with roadway widening under an existing bridge is shown in appendix G.

5.1 Design Example 1 - Cutslope Wall

5.1.1 Service Load Design (SLD)

5.1.1.1 Static Loading Condition

In accordance with the Standard Specifications for Highway Bridges, 15th Edition [30], Service Load Group I (table 4.2) defines the static loading condition for this problem.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

The technical and cost feasibility of a soil nail solution for a road cut through medium dense silty sands has been established. The location is remote from any buried utilities and there are no adjacent structures that will be impacted by the modest settlements that would be associated with construction of a soil nail wall within these materials. The ground water table is below the base of the planned cut, which has a maximum wall height of 9.50 meters, but minor perched ground water is expected and provision will be made for intercepting any seepage to the face and conducting the collected water to a footing drain through the use of geotextile strip face drains. The seismic design condition corresponds to a peak ground acceleration of 0.30 g.

The site investigation consisted of soil borings with standard penetration testing, laboratory classification testing, test pits, and in-situ density measurements. The investigation confirmed the subsurface soil and ground water conditions, and established that 2.5 meter high vertical cuts will stand unsupported for a minimum of several days. The soil profile consists of medium dense to dense silty sand. Average in-situ densities are 18.0 kN/m^3, and the soil strength parameters are estimated at a friction angle of 34.0 degrees and a cohesion of 5.0 kN/m^2. Although no nail pullout testing was performed during the site investigation, previous experience suggested that an ultimate pullout resistance on the order of 60.0 kN/m should be achievable with drillhole diameters on the order of 200 mm. This pullout resistance will have to be demonstrated for the contractor's proposed installation method during the initial stages of the wall construction.

Although the ground has not been determined to be aggressive in nature, the cut is adjacent to a major lifeline transportation corridor, and encapsulated nails will be used for corrosion protection. The site investigation has also confirmed that there will be no requirement for horizontal drains to address possible seasonal increases in the ground water table as the ground water table is located well below the base of the proposed wall.

The wall will have a vertical height of 9.50 meters, with a face batter of 10.0 degrees from vertical. The critical design cross section is shown in figure 5.1A and will have a 20.0 degree slope at the top of the wall, as shown. The trial nail spacing will be at 1.50 meters, vertically and horizontally, and the nails installed at 15.0 degrees below horizontal for constructability reasons.

Use the preliminary design charts in section 4.12 to determine the preliminary value for nail length and nail bar size. Select the design chart set corresponding to the appropriate backslope angle. If necessary, interpolate results for intermediate backslope angles to those given in the charts.

Figure 5.1A shows that the design section has a face batter of 10° and a backslope angle of 20°. Therefore use the design chart set presented on figures 4.31 and 4.32.

Compute the factored friction angle ϕ_D and the dimensionless factored soil cohesion c_D as defined above (equations 4.0a and 4.0c). From the appropriate chart A, determine the dimensionless nail tensile capacity T_D.

$$\phi_D = \tan^{-1}[\tan(\phi_U)/F_\phi] = \tan^{-1}[\tan(34°)/1.35] = 26.5°$$

$$\tan(\phi_D) = \tan(26°) = 0.5$$

$$c_D = c_U/(F_C\gamma H) = (5.0\ \text{kN/m}^2)/[1.35(18.0\ \text{kN/m}^3)(9.50\ \text{m})] = 0.022$$

From Chart A (figure 4.31), $T_D = 0.23$

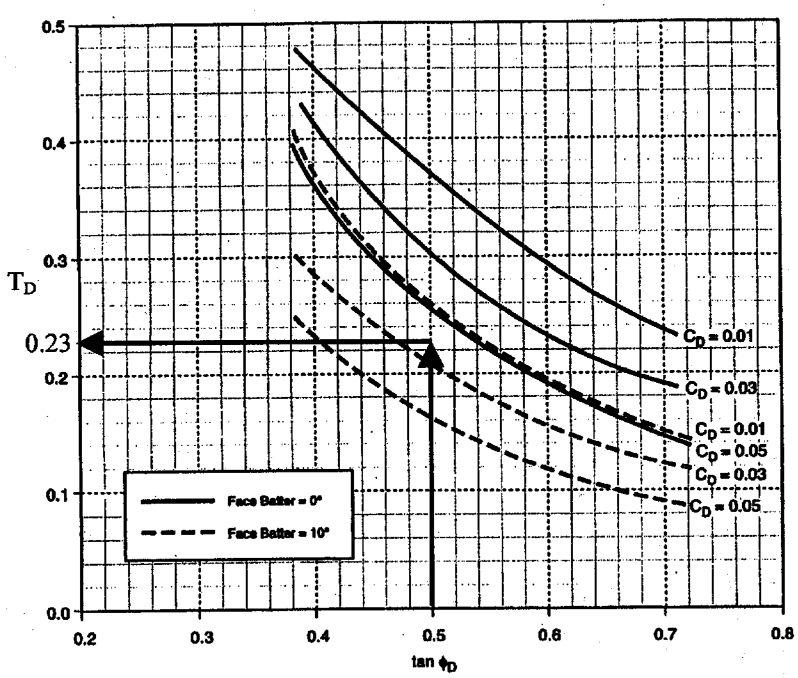


Figure 4.31 **Preliminary Design Chart 3A Backslope = 20° (Chart A)**

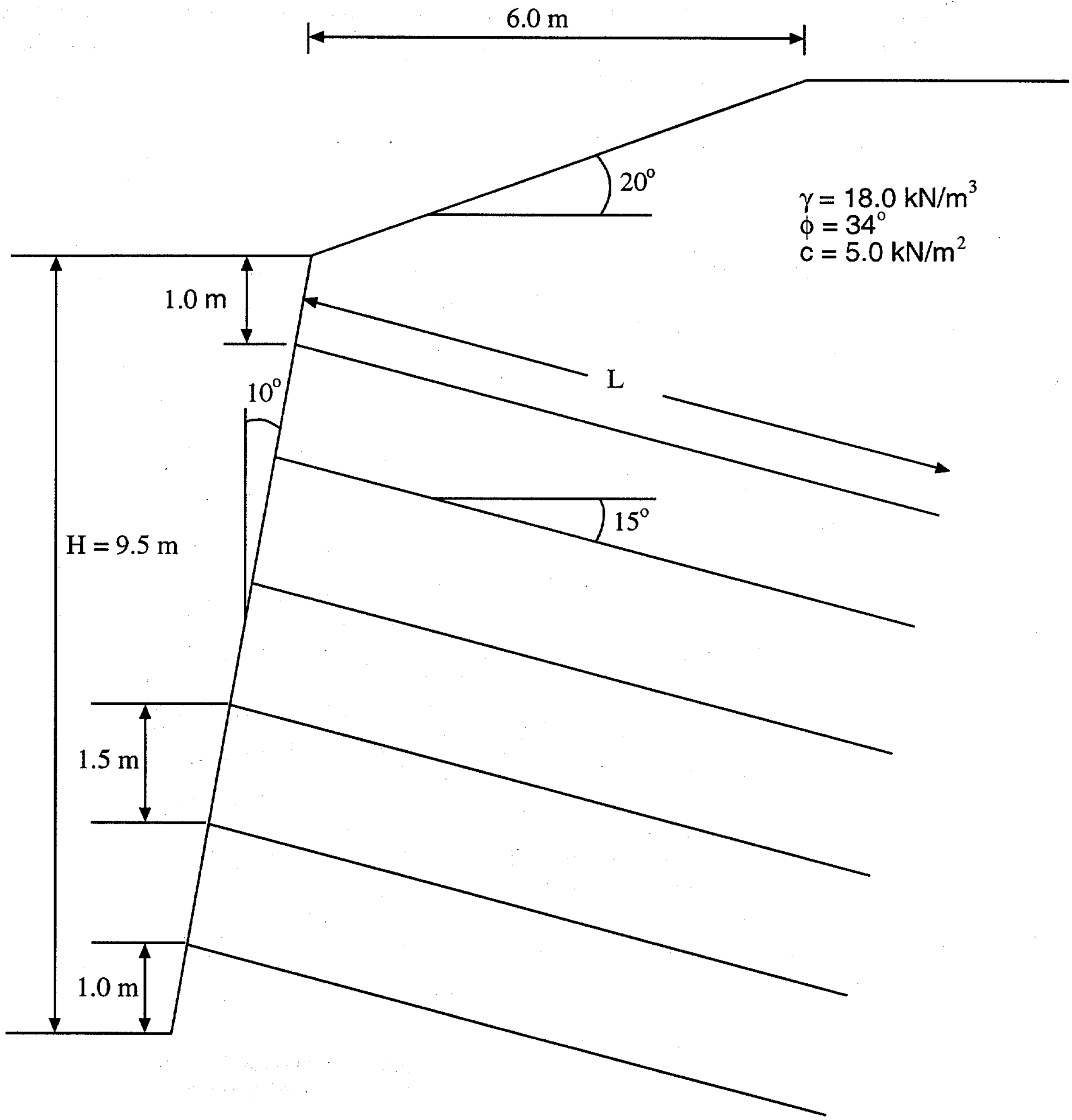


Figure 5.1A Cutslope Design Example

The required nominal nail tensile strength T_{NN} can be determined from the relations presented above, and from knowledge of the dimensionless nail tensile capacity T_D (from Chart A (figure 4.31)), the soil unit weight, the vertical height of the slope, the vertical and horizontal nail spacings, the nail tendon strength factor (SLD) or the nail tendon resistance factor and unit weight load factor (LRFD).

$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H)$
$T_{NN} = \gamma H S_V S_H T_D/\alpha_N = (18.0\ kN/m^3)(9.50\ m)(1.50\ m)(1.50\ m)(0.23)/(0.55)$
$T_{NN} = 161\ kN$ (Required nominal nail strength)
Area of bar (A_B) = $T_{NN}/F_y = 161/0.42 = 383\ mm^2$

From table 4.8A No. 22 ≈ 387 mm^2
No. 25 ≈ 510 mm^2

Select No. 25 bar for ease of handling and installation.

Compute the dimensionless nail pullout resistance Q_D. Divide the calculated dimensionless nail tensile capacity T_D by the computed dimensionless nail pullout resistance Q_D, and determine the required nail length from the appropriate Charts B/C, depending on the batter of the face of the wall.

$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) = (0.50)(60.0\ kN/m)/[(18\ kN/m^3)(1.50\ m)(1.50\ m)]$
$Q_D = 0.74$
$T_D/Q_D = 0.23/0.74 = 0.31$

From Chart C (figure 4.32), L/H = 0.87
L = 0.87(9.50 m) = 8.3 m

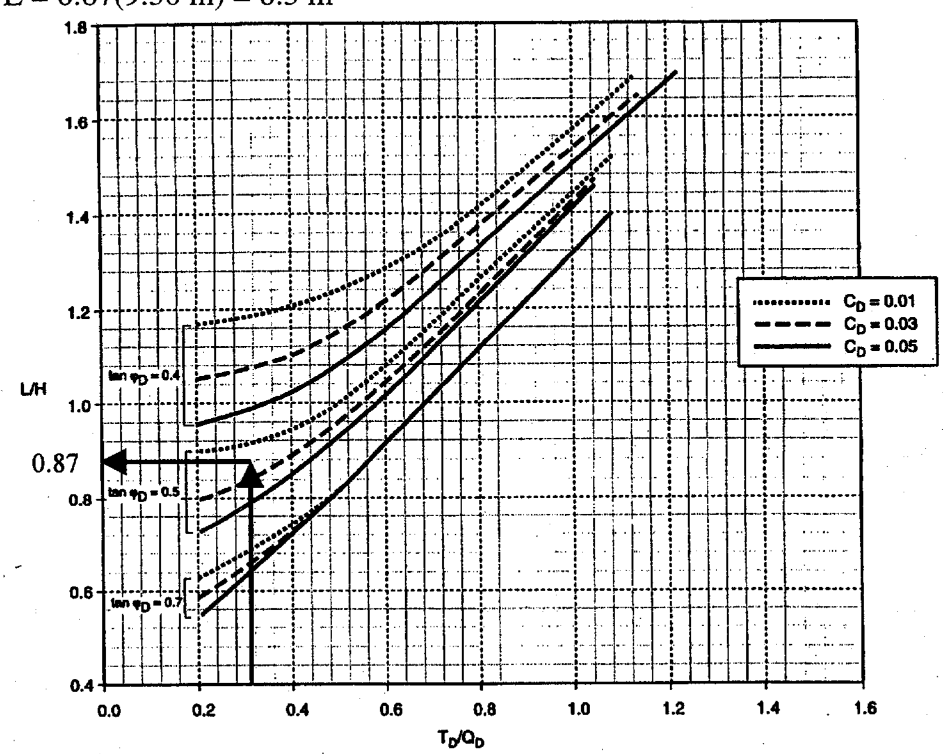


Figure 4.32 **Preliminary Design Chart 3C**
Backslope = 20° Face Batter = 10° (Chart C)

In summary the design charts in section 4.12 indicate a required nominal strength of about 161 kN (use No. 25, Grade 420 bars), and a nail length of about 8.3 meters. The nail length should be slightly conservative since the effective backslope angle of the design section shown in figure 5.1A may be something less than 20° (i.e., the design charts are prepared for constant backslope angles only, whereas the design example backslope angle is variable).

Therefore the trial analysis will assume that No. 25, Grade 420 bars approximately 8.3 meters long, will be required for support of the cut. Furthermore, based on local practice and material availability, the trial design will assume use of a "standard" temporary shotcrete construction facing (28-day compressive strength of 28 MPa) having a nominal thickness of 100 mm, reinforced with a single layer of 152x152 MW19xMW19 (6x6-W2.9xW2.9) welded wire mesh, two No. 13 Grade 420 continuous horizontal waler bars along each row of nails and two No. 13 vertical bearing bars at each nail head, as noted in section 4.5.

The nails will be connected to the shotcrete construction facing by a 225 mm square, 25 mm thick bearing plate, section 4.5.1. The nail pattern will be staggered such that the horizontal location of each nail is at the mid point of the nails in adjacent rows. The permanent facing will be a cast-in-place (CIP) concrete wall (28-day compressive strength of 28 MPa), 200 mm thick, reinforced with No. 13 Grade 420 deformed bars on 300 mm centers vertically and horizontally, and connected to the nail heads with a headed-stud connection system. Seismic loading will only be evaluated for the permanent facing.

Step 2 – Compute the Allowable Nail Head Loads

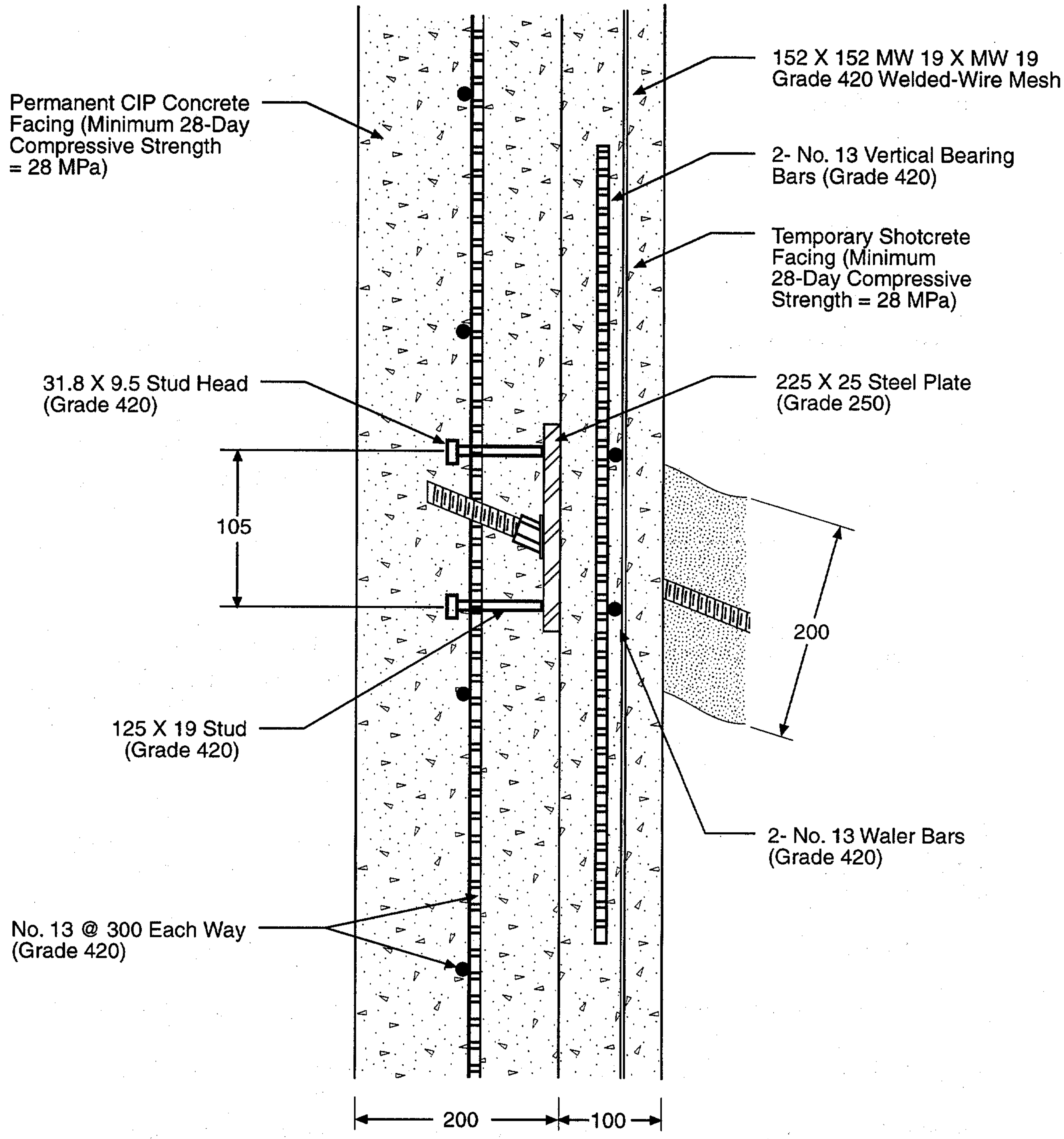
The problem specifies a 100-mm thick shotcrete construction facing and a 200-mm thick permanent cast-in-place (CIP) concrete facing (figure 5.1B). The nail head strength for the temporary facing are computed first, after which the permanent facing is examined.

Temporary Shotcrete Construction Facing

(a) Strength Criteria 1: Facing Flexure (section 4.5.2, equations 4.1 and 4.1A)

Based on soil and constructibility considerations as well as local practice, the vertical and horizontal nail spacings have both been selected to be 1.50 meters. The connection plate will be 225 mm wide and 25 mm thick. The facing reinforcement is selected by considering the various available mesh sizes, local practice, and strength requirements. Try a 152x152-MW19xMW19 mesh (steel area = 122.8 mm^2/m, table 4.1), with two No. 13 (Soft metric designation) waler bars and two No. 13 vertical bearing bars (steel area = 129 mm^2, table 4.8A). All facing steel is assumed to be nominally located at the center of the section. The yield stress of the reinforcement is specified as 420 MPa and the specified design concrete compressive strength at 28 days is 28 MPa.

The first step to evaluate equation 4.1 is to compute the negative and positive nominal unit moment resistances of the facing in the vertical direction. The equation 4.1A for the nominal unit moment resistance of a singly-reinforced, rectangular-shaped reinforced-concrete beam is as follows [1]:



All dimensions in millimeters

Figure 5.1B Facing Details

$$m = \frac{A_S F_Y}{b}\left(d - \frac{A_S F_Y}{1.70 f'_c\, b}\right)$$

The areas of vertical steel over the supports (2 No. 13 vertical bars and mesh vertical wires) and at midspan (mesh vertical wires) for a facing width b equal to 1.5 m are computed as follows:

$A_{S,NEG} = (122.8\ mm^2/m)(1.5\ m) + 2(129\ mm^2) = 443\ mm^2$

$A_{S,POS} = (122.8\ mm^2/m)(1.5\ m) = 185\ mm^2$

The corresponding average nominal unit moment resistances are computed as indicated below:

$$m_{V,NEG} = \frac{(443 mm^2)(420 MPa)}{1500 mm}\left(50.0 mm - \frac{(443 mm^2)(420 MPa)}{1.70(28.0 MPa)(1500 mm)}\right)$$

$= 5.88$ kN-m/m

$$m_{V,POS} = \frac{(185 mm^2)(420 MPa)}{1500 mm}\left(50.0 mm - \frac{(185 mm^2)(420 MPa)}{1.70(28.0 MPa)(1500 mm)}\right)$$

$= 2.53$ kN-m/m

From table 4.2, the facing flexure pressure factor C_F for a 100 mm thick temporary facing is 2.0. Substituting the corresponding values into, equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as follows:

$T_{FN} = C_F(m_{v,neg} + m_{v,pos})(8)(S_H)/(S_V)$

$T_{FN} = 2.0(5.88$ kN-m/m $+ 2.53$ kN-m/m$)(8)(1.50\ m)/(1.50\ m) = 135$ kN

(b) Strength Criteria 2: Facing Punching Shear (section 4.5.3, equation 4.3)

The nominal internal punching shear strength of the facing is computed from equation 4.2, where h_C and D'_C are as indicated below:

$h_C = 100$ mm

$D'_C = b_{PL} + h_C = 225\ mm + 100\ mm = 325\ mm$

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_N = 0.33\sqrt{28.0\text{MPa}}(\pi)(325\text{mm})(100\text{mm}) = 178\text{kN}$$

From table 4.2, the pressure factor for punching shear C_S for a 100 mm thick temporary facing is 2.5. The punching cone bottom diameter D_C is equal to $D'_C + h_C = 425$ mm. The diameter of the grout column is estimated to be about 200 mm. The corresponding areas are computed as follows:

$$A_C = 0.25(\pi)(D_C)^2 = 0.25(\pi)(425 \text{ mm})^2 = 1.42 \times 10^5 \text{ mm}^2$$

$$A_{GC} = 0.25(\pi)(D_{GC})^2 = 0.25(\pi)(200 \text{ mm})^2 = 3.14 \times 10^4 \text{ mm}^2$$

Substituting the above values into equation 4.3, the nominal nail head strength for the criteria of punching shear is computed as follows:

$$T_{FN} = (178\text{kN})\left(\frac{1}{1 - 2.5\dfrac{(1.42\text{x}10^5\text{mm}^2 - 3.14\text{x}10^4\text{mm}^2)}{[(1500\text{mm})(1500\text{mm}) - 3.14\text{x}10^4\text{mm}^2]}}\right) = 204\text{kN}$$

Permanent Cast-In-Place (CIP) Concrete Facing

(a) Strength Criteria 1: Facing Flexure (section 4.5.2, equation 4.1)

The same nail spacings and connection plate dimensions used for temporary facing apply to the permanent facing. Based on considerations including corrosion, durability, and quality of construction practice, the temporary facing is neglected in the permanent design. The thickness of the permanent facing is selected to be 200 mm.

By considering the available steel bar sizes and local practice, the facing reinforcement is selected to be No. 13 deformed bars at 300 mm centers in both the horizontal and vertical directions. All facing steel is assumed to be nominally located at the center of the section. The design yield stress of the reinforcement is again specified as 420 MPa and the specified 28-day design concrete compressive strength is 28.0 MPa.

The first step to evaluate equation 4.1 is to compute the negative and positive nominal unit moment resistances of the facing in the vertical direction. The areas of vertical steel over the supports and at the midspan for a width of 1500 mm are computed as follows:

$$A_{S,NEG} = A_{S,POS} = (129 \text{ mm}^2)(1500 \text{ mm})/(300 \text{ mm}) = 645 \text{ mm}^2$$

Using equation 4.1A, the average nominal unit moment resistances are computed as follows:

$$m_{V,NEG} = \frac{(645\text{mm}^2)(420\text{MPa})}{1500\text{mm}}\left(100\text{mm} - \frac{(645\text{mm}^2)(420\text{MPa})}{1.70(28.0\text{MPa})(1500\text{mm})}\right)$$

$= 17.4$ kN-m/m

$m_{V,POS} = m_{V,NEG} = 17.4$ kN-m/m

From table 4.2, the facing flexure pressure factor C_F for a 200 mm thick permanent facing is 1.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as follows:

$T_{FN} = 10(17.4 \text{ kN-m/m} + 17.4 \text{ kN-m/m})(8)(1.50 \text{ m})/(1.50 \text{ m}) = 278 \text{ kN}$

(b) Strength Criteria 2: Facing Punching Shear (section 4.5.3, equation 4.3)

The dimensions of the headed studs must be selected in order to calculate the geometry of the potential punching cone. Per Section 4.5.4, the headed-stud dimensions are first selected to satisfy the provisions of ACI Committee 349 [2]. Try a 22.0 mm body diameter, a 35.0 mm head diameter, a 9.5 mm head thickness, an overall length of about 125 mm (corresponding to a $^7/_8$ x $5^3/_{16}$ anchor size, table F.3), and a stud spacing of 105 mm. The provisions are checked as shown below:

<u>Provision 1:</u>	d_H	$\geq (2.5)^{0.5}(22.0 \text{ mm})$	
	35.0 mm	$\geq (2.5)^{0.5}(22.0 \text{ mm})$	
	35.0 mm	≥ 35.0 mm	(O.K.)
<u>Provision 2:</u>	t_H	$\geq 0.5\,(d_H - d_{HS})$	
	9.5 mm	$\geq 0.5(35.0 \text{ mm} - 22.0 \text{ mm})$	
	9.5 mm	≥ 6.5 mm	(O.K.)

The nominal internal punching shear strength of the facing is computed from equation 4.2, where h_C and D'_C are indicated below:

h_C = plate thickness + stud length

$h_C = 25 \text{ mm} + 125 \text{ mm} = 150 \text{ mm}$

$D'_C = S_{HS} + h_C = 105 \text{ mm} + 150 \text{ mm} = 255 \text{ mm}$

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_N = 0.33\sqrt{28.0\text{MPa}}\,(\pi)(255\text{mm})(150\text{mm}) = 210\text{kN}$$

From table 4.2, the pressure factor for punching shear for a 200 mm thick permanent facing is 1.0. The punching cone bottom diameter D_C is equal to $D'_C + h_C = 405$ mm. The diameter of the grout column is estimated to be about 200 mm. The corresponding areas are computed as follows:

$$A_{GC} = 0.25(\pi)(D_{GC})^2 = 0.25(\pi)(200 \text{ mm})^2 = 1.29 \text{ x } 10^5 \text{ mm}^2$$

Substituting the above values into equation 4.3, the nominal nail head strength for the criteria of punching shear is computed as follows:

$$T_{FN} = (210\text{kN})\left(\frac{1}{1-1.0\dfrac{(1.29\text{x}10^5\,\text{mm}^2 - 3.14\text{x}10^4\,\text{mm}^2)}{[(1500\text{mm})(1500\text{mm}) - 3.14\text{x}10^4\,\text{mm}^2\,]}}\right) = 219\text{kN}$$

(c) Strength Criteria 3: Headed-Stud Tension (section 4.5.6, equation 4.4)

The nominal nail head strength associated with the criteria of headed-stud tension is computed by equation 4.4. For 22.0 mm diameter studs and a specified headed-stud ultimate tensile stress of 420 MPa, the nominal nail head strength is computed to be:

$$T_{FN} = 4(0.25)(\pi)(22.0 \text{ mm})^2(420 \text{ MPa}) = 639 \text{ kN}$$

The nominal nail head strengths for all credible failure mechanisms are tabulated below for both the shotcrete and CIP facings. The allowable nail head loads for Service Load Group I are computed from the nominal strengths as indicated in the tables below for both the temporary shotcrete construction facing and the permanent CIP concrete facing.

SHOTCRETE CONSTRUCTION FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load (Group I) T_F (kN)
Facing Flexure	135	$0.67^a(135) = 90$
Facing Punching Shear	204	$0.67^a(204) = 137$

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load (Group I) T_F (kN)
Facing Flexure	278	$0.67^a(278) = 186$
Facing Punching Shear	219	$0.67^a(219) = 147$
Headed-Stud Tensile Fracture	639	$0.50^a(639) = 320$

[a] See Table 4.4.

Note: Because of the load types on the cutslope section, Group I loading is obviously more critical then Group IV.

Therefore, the allowable nail head load T_F is computed to be 90 kN. That is, facing flexure of the temporary facing is the controlling mode of failure.

Step 3 - Minimum Allowable Nail Head Service Load Check

Since the design problem has a relatively simple geometry and uniform soil conditions, the active earth pressure coefficient can be analytically determined from a Coulomb analysis (AASHTO, 15th Edition, 1992 - figure 5.5.2B [30]). For a friction angle of 34 degrees, a face slope angle of 10 degrees from vertical and a backslope angle of 20 degrees, the active earth pressure coefficient corresponding to a horizontal, triangular earth pressure distribution is given by K_A equal to 0.257 (ignoring the cohesive component of soil strength in accordance with the discussion of section 2.4.4 and 2.4.5). Values for other parameters are as follows:

$S_H = S_V = 1.50\text{m}$
$H = 9.50\text{ m}$
$F_F = 0.50$

Substituting the above values into equation 4.6, to determine the nail head service load:

$$t_F = F_F K_A \gamma H S_H S_V = (0.50)(0.257)(18\text{ kN/m}^3)(9.5\text{ m})(1.50\text{ m})^2$$
$$t_F = 49\text{ kN} < 90\text{ kN}$$

(OK - the estimated nail head service load does not exceed the allowable nail head load)

Step 4 - Define the Allowable Nail Load Support Diagrams

Develop the allowable nail load diagram for each nail by determining the allowable pullout resistance, the allowable nail head load, and the allowable nail tendon tensile load.

Allowable Pullout Resistance, Q, (Ground-Grout Bond)

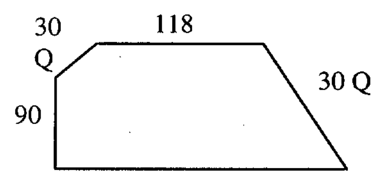
$Q = \alpha_Q Q_U$ (see table 4.5 notes)
$\alpha_Q =$ 0.50 (table 4.5)
$Q_U =$ 60.0 kN/m
$Q =$ (0.50)(60.0 kN/m) = 30.0 kN/m

Allowable Nail Tendon Tensile Load, T_N

$T_N = \alpha_N T_{NN}$ (see table 4.5 notes)
$\alpha_N =$ 0.55 (table 4.5)
$T_{NN} = A_B F_Y =$ (510 mm^2)(0.42 kN/mm^2) = 214 kN
$T_N =$ (0.55)(214 kN) = 118 kN

Allowable Nail Head Load

Per step 2, the allowable nail head load is 90 kN. The nail load support diagram is constructed by plotting the nail head load (90kN) vertically, extending the pullout resistance (Q) from the nail head load until the nail tendon (T_N) load is reached. The nail tendon load is extended horizontally until the pullout resistance line (Q) for the end of the box is intersected.



Step 5 - Select Trial Nail Spacings and Lengths

In step 1, a preliminary nail length of 8.3 m at a horizontal and vertical spacing of 1.5 m was selected. However as noted in section 4.7.1, this length only represents the nail length in the upper half of the wall. The nail lengths in the lower half of the wall need to be artificially shortened prior to performing a limiting equilibrium analysis in order that the upper nail lengths are adequate to resist the anticipated loads at small deflections. Figure 4.11 is used as follows to determine the distributions of nail lengths with depth.

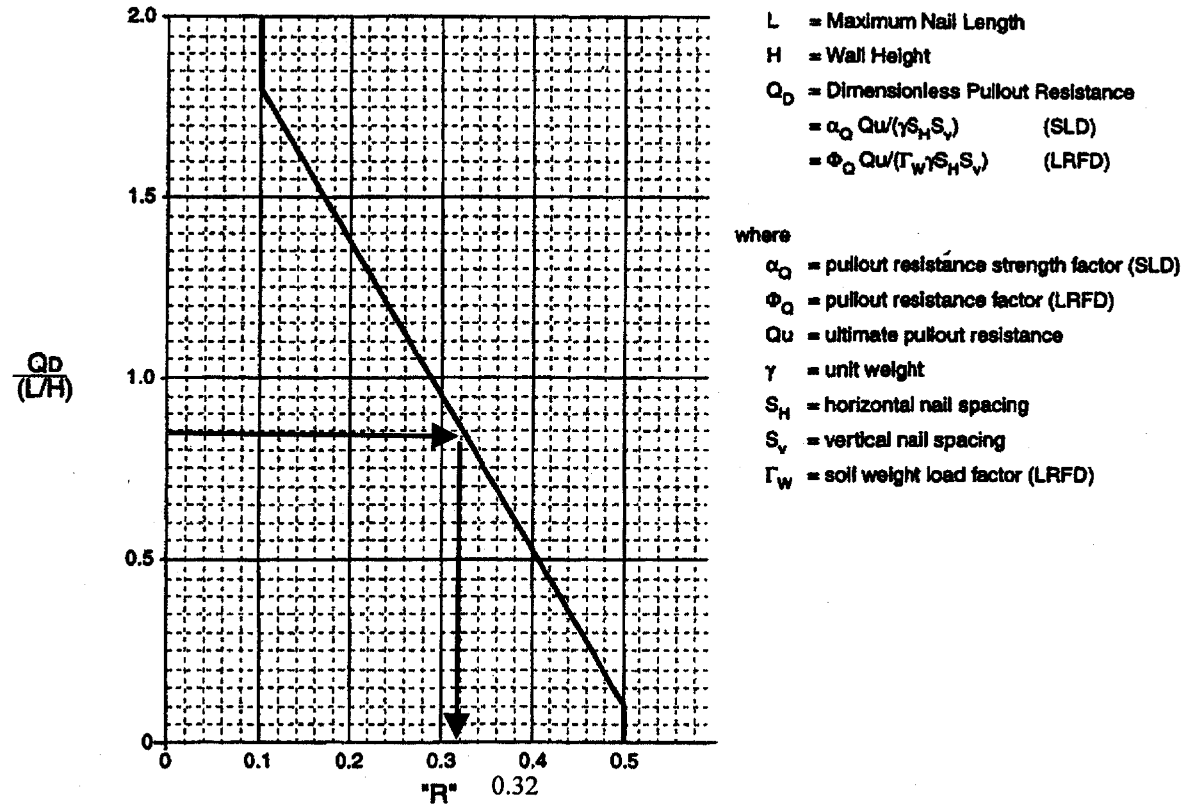
- The dimensionless nail pullout resistance, Q_D, is calculated:

$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) =$ (0.50)(60.0 kN/m)/[(18 kN/m^3)(1.50 m)(1.50 m)]
$=$ 0.74

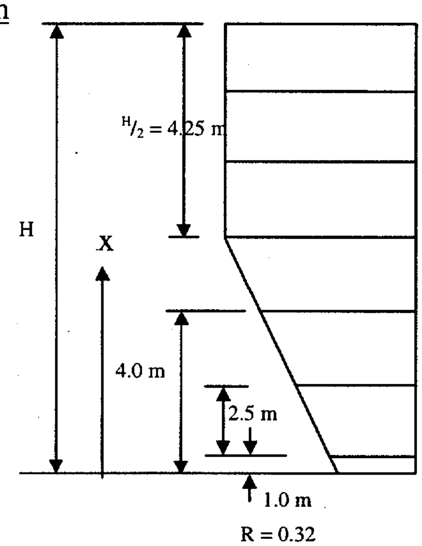
- The dimensionless nail length is:

$L/H =$ (8.3 m)/(9.5 m) $=$ 0.87

- Q_D / (L/H) = 0.74/0.87 = 0.85, giving an "R" factor of 0.32.
- Relative nail lengths are calculated from figure 4.11 for the nail head elevations shown on figure 5.1C and an "R" value of 0.32.



Nail No.	Trial Length,m	Rx	Trial Nail Length Distribution
1	8.3	1.0	8.3
2	8.3	1.0	8.3
3	8.3	1.0	8.3
4	8.3	0.89	7.4
5	8.3	0.68	5.6
6	8.3	0.46	3.8



Where $Rx = {}^{X}/_{(H/2)}\,(1-R) + R$

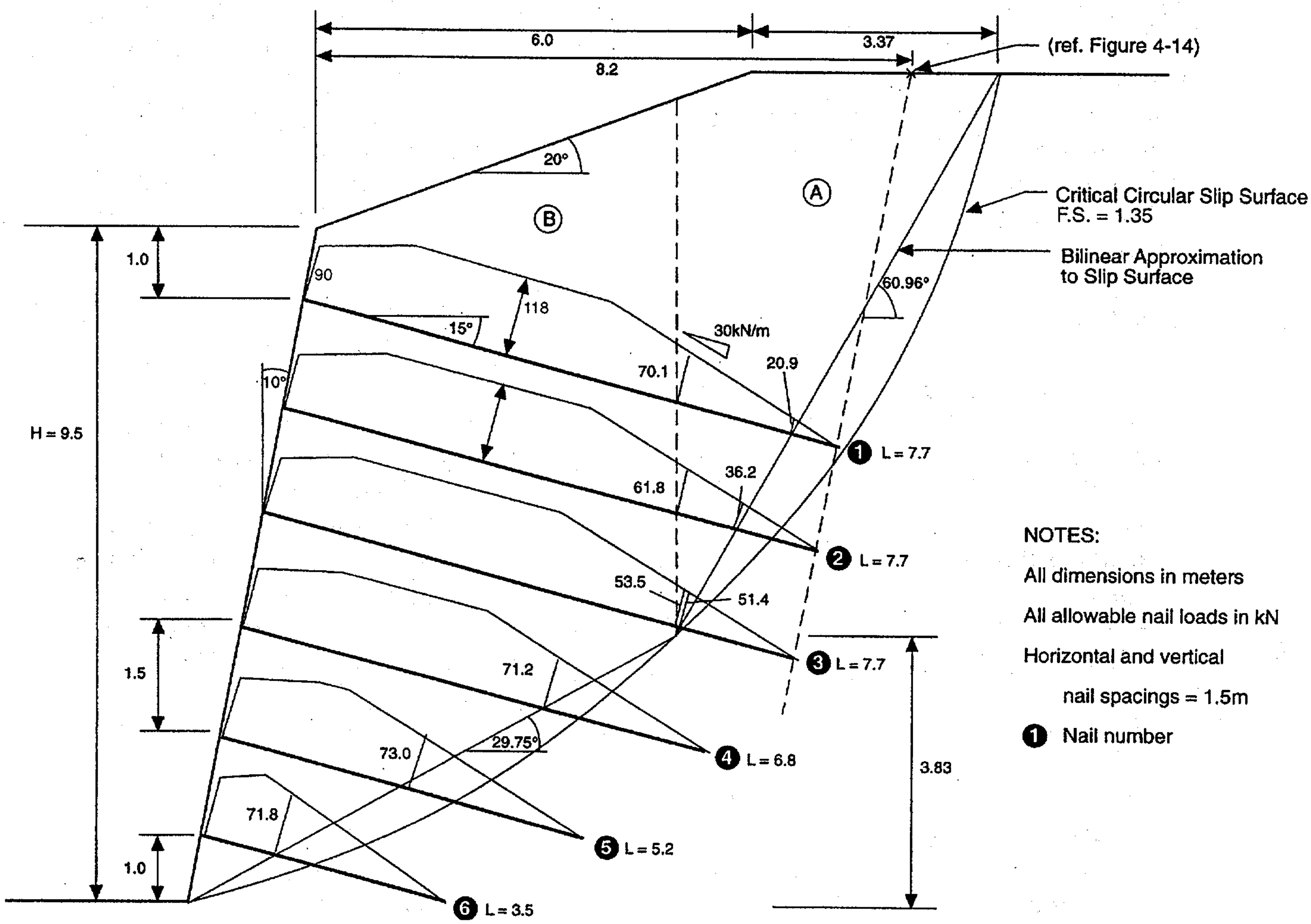


Figure 5.1C Cutslope Design Example
Trial Design Critical Cross Section
Static Loading (SLD)

Step 6 - Define the Ultimate Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0°
Ultimate Cohesion, c_U = $5.0\ \mathrm{kN/m^2}$

Step 7 - Calculate the Factor of Safety

An iterative limiting equilibrium analysis is performed using appropriate computer software to determine the actual nail lengths that are required for a global safety factor of 1.35 (table 4.5 Group I loading). The nail length distribution for design purposes and the allowable nail load support diagrams are shown on Figure 5.1C. Note that the maximum nail length has been calculated interactively to be 7.7 meters. This is slightly less than the 8.3 meters estimated from the design charts, partially due to the broken backslope adjacent the soil nail wall giving an "effective" backslope angle less than 20 degrees assumed in the design chart calculations.

Hand Calculation Check

In general, it will not be necessary to perform extensive hand calculation checks if a properly verified computer program is used. The following provides an approximate method for performing hand calculation checks, when they are required or when a verification check of a computer solution is desired. The method is approximate in that it is based on force balance only and more reliable results will generally be obtained using methods that address both force and moment equilibrium. In order to facilitate the hand calculation check, the critical circular slip surface may be approximated by a bilinear wedge, as shown on figure 5.1C. The forces on the 'active' Block A and the 'resisting' Block B are illustrated on figure 5.2 for the bilinear wedge, and are computed below:

Block A

Base Slope Angle, α_A = 60.96°
Base Length, L_A = 9.00 m
Block Weight, W_A = $(17.01\ \mathrm{m^2})(18.0\ \mathrm{kN/m^3})$ = 306.2 kN/m
Nail Force, T_A = $[(20.9\ \mathrm{kN})^{(1)} + (36.2\ \mathrm{kN})^{(2)} + (51.4\ \mathrm{kN})^{(3)}]/(1.50\ \mathrm{m})$
= 72.3 kN/m
$^{(n)}$ Nail Number

Block B

Base Slope Angle, α_B = 29.75°
Base Length, L_B = 7.72 m
Block Weight, W_B = $(47.3\ \mathrm{m^2})(18.0\ \mathrm{kN/m^3})$ = 851.8 kN/m
Nail Force, T_B = $[(71.2\ \mathrm{kN})^{(4)} + (73.0\ \mathrm{kN})^{(5)} + (71.8\ \mathrm{kN})^{(6)}]/(1.5\ \mathrm{m})$
= 144.0 kN/m

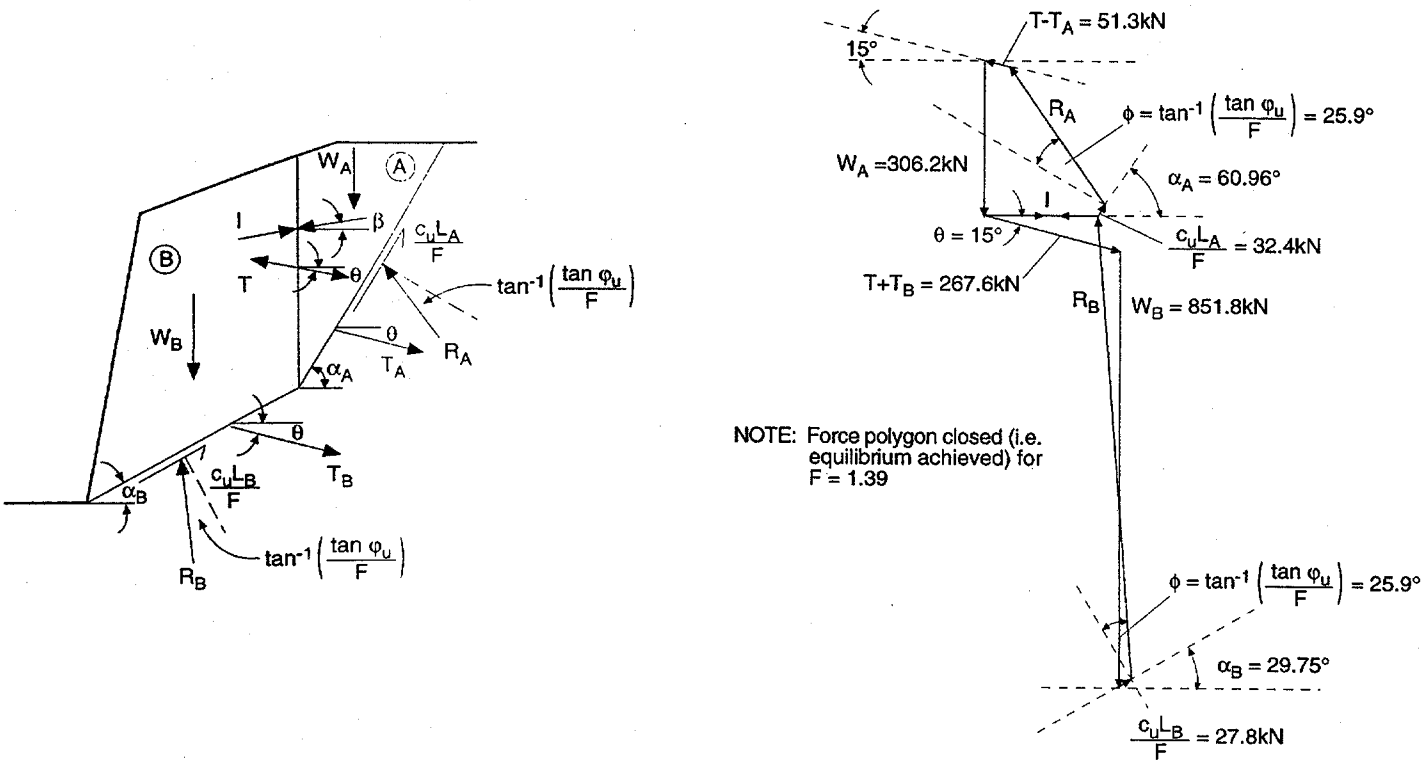


Figure 5.2. Cutslope Design Example Force Diagram and Polygon (SLD) Hand Calculation Check

Interslice

Nail Force, $T = [(70.1 \text{ kN})^{(1)} + (61.8 \text{ kN})^{(2)} + (53.5 \text{ kN})^{(3)}]/(1.5 \text{ m})$
$= 123.6 \text{ kN/m}$

Assume that the interslice soil force, I, is inclined at an angle 'β' to the horizontal (figure 5.2). The factor of safety F is then applied to the soil strengths, as indicated below.

Equilibrium

Define $\phi = \tan^{-1}[\tan(\phi_U)/F]$.

Block A

Vertical

$$W_A + (T_A - T)\sin(\theta) - (I)\sin(\beta) - c_U(L_A)\sin(\alpha_A)/F - (R_A)\cos(\alpha_A - \phi) = 0$$

Horizontal

$$(I)\cos(\beta) + (T_A - T)\cos(\theta) + c_U(L_A)\cos(\alpha_A)/F - (R_A)\sin(\alpha_A - \phi) = 0$$

Block B

Vertical

$$W_B + (T_B + T)\sin(\theta) + (I)\sin(\beta) - c_U(L_B)\sin(\alpha_B)/F - (R_B)\cos(\alpha_B - \phi) = 0$$

Horizontal

$$(I)\cos(\beta) - (T_B + T)\cos(\theta) - c_U(L_B)\cos(\alpha_B)/F + (R_B)\sin(\alpha_B - \phi) = 0$$

The above equations can be reduced to the following:

$$[1+\tan(\beta)\tan(\alpha_A-\phi)]\,[\{-W_B - (T_B + T)\sin(\theta)+ c_U(L_B)\sin(\alpha_B)/F\}\tan(\alpha_B-\phi)+ (T_B + T)\cos(\theta)+ c_U(L_B)\cos(\alpha_B)/F] = [1+\tan(\beta)\tan(\alpha_B-\phi)]\,[\{W_A + (T_A - T)\sin(\theta)- c_U(L_A)\sin(\alpha_A)/F\}\tan(\alpha_A-\phi)-(T_A - T)\cos(\theta)- c_U(L_A)\cos(\alpha_A)/F]$$

The above equation can be solved iteratively for the factor of safety F (e.g., by use of a spreadsheet), if the interslice force angle β is assumed. For example, assuming $\beta = 0$, the calculated factor of safety F = 1.39, which is similar to the computer-generated solution. The force polygon for the hand calculation is shown on figure 5.2.

The results of stability assessments during construction are shown on figure 5.3. Calculated factors of safety for each stage of construction, following excavation of each lift and prior to installation of the associated row of nails, are shown together with the corresponding critical slip surface in each case. It can be seen that for this problem,

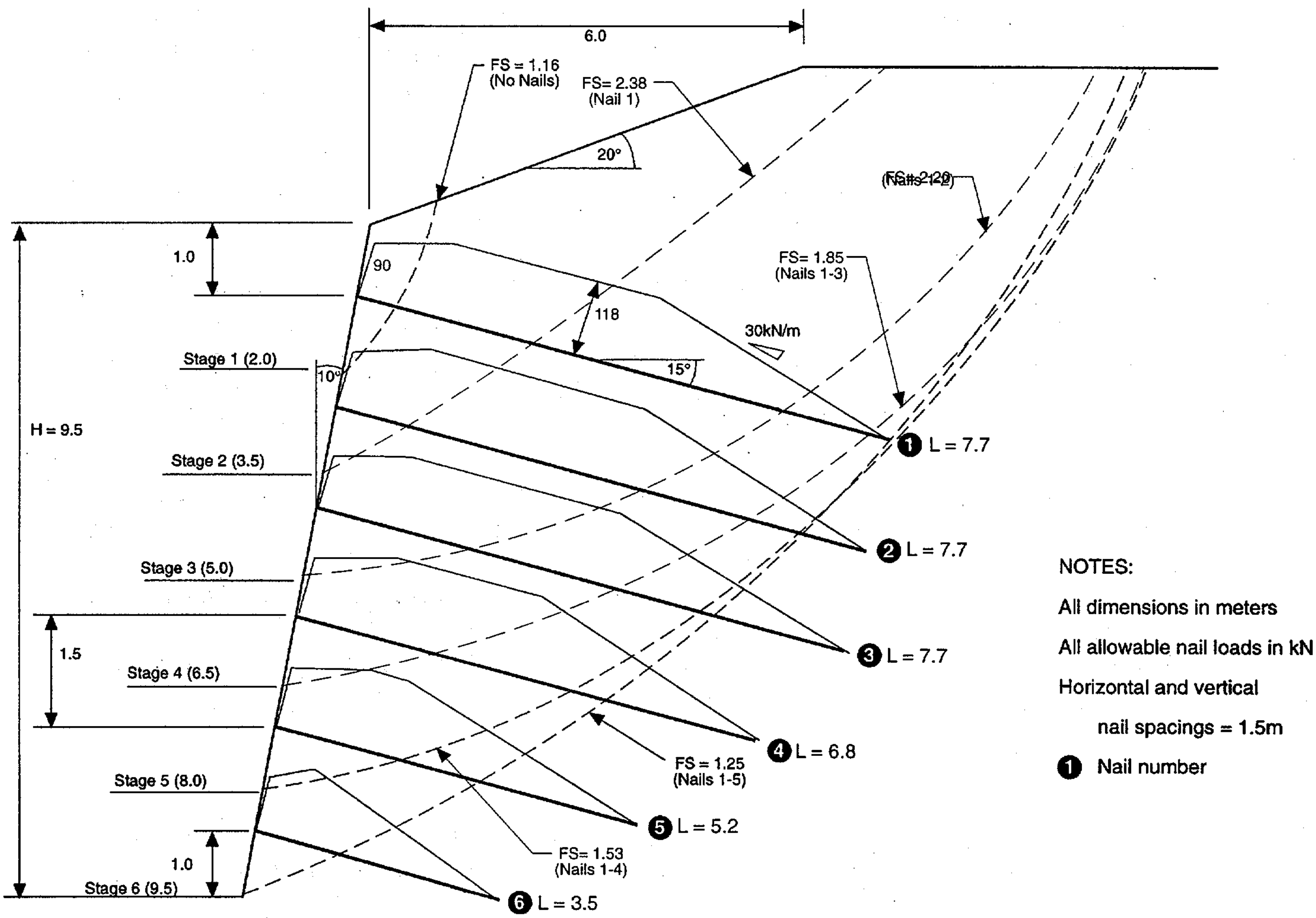


Figure 5.3 Cutslope Design Example Construction Stability (SLD)

following installation of the first row of nails, the factor of safety during construction decreases progressively as the height of the wall increases. However, the minimum calculated factor of safety is 1.25, which exceeds the minimum required value of 1.2 (table 4.5). Figure 5.3 does indicate that prior to the installation of the first row of nails, the calculated factor of safety is less than 1.2 but this construction condition has been evaluated in the field by test trenching during the site investigation stage.

Step 8 - External Stability Check of Nailed Block

Defer the external stability check of the nailed block until the final soil nail lengths are defined for the seismic loading condition. By inspection of the problem a bearing capacity check is not necessary for the static design. However, it will be presented later for the purpose of this example.

Step 9 - Check the Upper Cantilever

The height of the upper cantilever above the top nail is identical (1.0 meters) for both the temporary shotcrete and permanent CIP wall facings. Therefore, the static loading is identical in the two cases. Because both the facing thickness and steel content are increased in the permanent facing, the permanent facing is less critical by inspection. Therefore, for the static loading condition, only the construction facing upper cantilever needs to be evaluated.

For the method of construction, the appropriate earth pressure coefficient for the upper cantilever design is an active earth pressure coefficient. For a soil friction angle of 34°, zero cohesion (ignore it), a soil-to-wall interface friction angle of (2/3)(34°) = 22°, a wall batter of 10°, and a backslope angle of 20°, Coulomb's earth pressure theory may be used to derive an active earth pressure coefficient, K_A, equal to 0.247 (AASHTO, 15th Edition, 1992 - figure 5.5.2B [30]). The load component normal to the wall has a corresponding earth pressure coefficient equal to (0.247)cos(22°) = 0.229.

Shear Check

From force equilibrium, compute the one-way unit service shear force for the facing at the level of the upper row of nails (conservative), as indicated on figure 4.13:

Shear force $v_I = 0.5K_A\cos(\text{soil/wall friction angle})\gamma H^2$ (due to soil above 1st nail)

$$v = 0.5(0.229)(18.0\ \text{kN/m}^3)\ (1.0\ \text{m})^2$$

$$v = 2.06\ \text{kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 8-49 of the Standard Specifications for Highway Bridges, 15th Edition [30]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$

$$V_{NS} = 0.166\sqrt{28.0\ \text{MPa}}\ (0.05\ \text{m}) = 43.9\ \text{kN/m}$$

From table 4.4, the facing shear strength factor, α_F, equals 0.67, same factor for punching shear. Therefore, the allowable one-way unit shear is computed to be:

$$V = \alpha_F V_{NS} = (0.67)(43.9 \text{ kN/m}) = 29.4 \text{ kN/m}$$

Since v < V, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the one-way unit service moment for the facing at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever:

$$m_S = (0.33)(H/\cos(10^\circ))(v)$$
$$m_S = (0.33)(1.0 \text{ m}/\cos(10^\circ))(2.06 \text{ kN/m}) = 0.690 \text{ kN-m/m}$$

Compute the nominal unit moment resistance of the facing. From appendix F, the nominal unit moment resistance in the vertical direction over the nail locations, $m_{V,NEG}$, is computed to be 5.88 kN-m/m. From table 4.4, the strength factor, α_F, for facing flexure is 0.67. Therefore, the allowable one-way unit moment for the upper cantilever is:

$$M = \alpha_F m_{V,NEG} = (0.67)(5.88 \text{ kN-m/m}) = 3.94 \text{ kN-m/m}$$

Since $m_S < M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

Shotcrete Construction Facing

Check 1 - Waler Reinforcement

Two No. 13 horizontal waler bars, attached beneath the bearing plate, will be placed continuously between nail heads in each nail row.

Check 2 - Minimum Reinforcement Ratios

The minimum reinforcement ratio requirements in the Standard Specifications for Highway Bridges, 15th Edition [30] are waived for the temporary shotcrete construction facing per the discussion presented in section 4.7.1.

Check 3 - Minimum Cover Requirements

For the temporary shotcrete construction facing, the reinforcement is placed at the center of the facing (figure 5.1B).

Check 4 - Development and Splices of Reinforcement

Development of No. 13 Vertical Bearing Bars

For a uniformly loaded interior span with fixed-end supports, the point of zero moment theoretically occurs at $0.213L_C$ from the support. The vertical bearing bars are not needed all the way to the point of zero moment. However, per section 8.24 of AASHTO [30], they must extend the maximum of $L_C/20$, $15d_B$, or d, past the point at which they are no longer needed:

$$
\begin{aligned}
L_C/20 &= (1500\ \text{mm})/20 = 75\ \text{mm} \\
15d_B &= 15(12.7\ \text{mm}) = 191\ \text{mm} \\
d &= (100\ \text{mm})/2 = 50\ \text{mm}
\end{aligned}
$$

Since the vertical bearing bars comprise about $(258\ \text{mm}^2)/(443\ \text{mm}^2) = 58$ percent of the total vertical reinforcement, estimate the point at which the vertical bearing bars are no longer needed as $0.58(0.213L_C)$ from the support. The total bar length is computed to be:

$$L_{VB} = 2[0.58(0.213)(1.50\ \text{m}) + 0.191\ \text{m})] = 0.753\ \text{m} \quad \text{(Minimum)}$$

Section 8.24.3.3 of AASHTO [30] provides additional requirements for embedments beyond points of inflection. The only requirement that is potentially more critical than the above is an embedment at least equal to:

$$L_C/16 = (1500\ \text{mm})/16 = 93.8\ \text{mm}$$

This is less critical than the $15d_B = 191$ mm, computed above.

Splicing of No. 13 Waler Bars

Per section 8.32 of AASHTO [30], the splice length, L_S, for a Class C splice must equal or exceed the greater of 0.30 m or $L_D = 1.7L_{DB}$, where L_{DB} is computed from section 8.25:

$$
\begin{aligned}
L_{DB} &= 0.019(A_B\ (\text{mm}^2))(F_Y\ (\text{MPa}))/\sqrt{f'_C\ (\text{MPa})} \\
L_{DB} &= 0.019(129\ \text{mm}^2)(420\ \text{MPa})/\sqrt{28.0\ \text{MPa}} = 195\ \text{mm} \\
L_D = 1.7L_{DB} &= 1.7(195\ \text{mm}) = 332\ \text{mm}
\end{aligned}
$$

Therefore, provide 350 mm of splice length.

Splicing of MW 18.7 Mesh

Per section 8.30 of AASHTO [30], the splice length between outermost crosswires must equal or exceed the greater of (S_{WIRE} + 50.0 mm), $1.5L_{DB}$, or 150 mm. L_{DB} is computed from AASHTO equation 8-68:

$$L_{DB} = 3.25(A_{WIRE}\ (mm^2))(F_Y\ (MPa))/(((S_{WIRE}\ (mm))\sqrt{f'_C\ (MPa)}\ ))$$
$$L_{DB} = 3.25(18.7\ mm^2)(420\ MPa)/((152\ mm)\sqrt{28.0\ MPa}\ )$$
$$L_{DB} = 31.7\ mm$$
$$L_D = 1.5L_{DB} = 1.5(31.7\ mm) = 47.6\ mm$$
$$S_{WIRE} + 50.0\ mm = 152\ mm + 50.0\ mm = 202\ mm$$

Therefore, use a minimum of 200-mm splices for the wire mesh.

Permanent Facing

Check 1 - Waler Reinforcement

There are no applicable requirements.

Check 2 - Minimum Reinforcement Ratios

Per section 8.20 of AASHTO [30], the minimum required amount of shrinkage and temperature reinforcement near exposed surfaces of walls and slabs is 265 mm^2 per lineal meter. The No. 13 bars on 300-mm centers provides (129 mm^2)/(0.3 m) = 430 mm^2 per meter, which is adequate.

Check 3 - Minimum Cover Requirements

Per section 8.22 of AASHTO [30], the minimum cover on the front side of the facing is specified at 50 mm. Based on the design illustrated on figure 5.1B, the cover to the headed studs is as indicated below:

$$t_C = 200\ mm - t_{PL} - L_{HS} = 200\ mm - 25\ mm - 125\ mm = 50\ mm$$
$$50\ mm \geq 50\ mm \quad (OK)$$

The minimum required cover between the permanent facing reinforcing steel and the CIP concrete/temporary shotcrete interface is 38 mm. Based on figure 5.1B, this cover is:

$$t_c = 100\ mm - 12.7\ mm = 87.3\ mm > 38\ mm$$

For corrosion protection purposes, there must also be a minimum 75 mm of cover between the facing steel and the soil. Based on figure 5.1B, the 100 mm thick temporary shotcrete provides adequate cover for the permanent facing steel.

Check 4 - Development and Splices of Reinforcement

The permanent facing reinforcement consists entirely of No. 13 deformed bars. Therefore, the development lengths and splice lengths are identical to those computed for the temporary facing.

Step 11 - Serviceability Checks

Shotcrete Construction Facing

Because of the temporary nature of the wall, the serviceability requirements are waived for the construction facing.

Permanent Facing

Upper Cantilever Serviceability Check - Reinforcement Distribution

Per section 8.16.8 of AASHTO [30], the reinforcement must be distributed such that the steel stress does not exceed that given by the following equation:

$$F_S = \frac{z}{(d_C A_E)^{1/3}} \leq 0.6F_Y$$

$$z = 130 \text{ k/in} = 2.28 \times 10^4 \text{ kN/m}$$

$$d_C = 0.05 \text{ m}$$

$$A_E = (100 \text{ mm})(S_H)/(A_{TOTAL}/A_{LB})$$

$$A_E = (100 \text{ mm})(1500 \text{ mm})/((645 \text{ mm}^2)/(129 \text{ mm}^2))$$

$$A_E = 0.03 \text{ m}^2$$

$$0.6F_Y = 0.6(420 \text{ MPa}) = 252 \text{ MPa}$$

$$F_S = \frac{2.28 \times 10^4 \text{ kN/m}}{((0.05 \text{ m})(0.03 \text{ m}^2))^{1/3}} = 199 \text{ MPa}$$

From step 9, the service moment, m_S, is 0.690 kN-m/m. The corresponding service steel stress is determined from straight-line theory of reinforced concrete:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$

$$\rho = A_S/(bd)$$

$$n = E_S/E_C$$

$$j = 1 - k/3$$

$$f_S = m_S\, b/(A_S j d)$$

$$E_C = 4734\sqrt{f'_c\,(\text{MPa})} = 4734\sqrt{28 \text{ MPa}}$$

$$E_C = 2.50 \times 10^4 \text{ MPa}$$

$$E_S = (29{,}000{,}000 \text{ psi})(1 \text{ MPa})/(145 \text{ psi}) = 2.00 \times 10^5 \text{ MPa}$$

Substituting the correct values into the above expressions, the service steel stress is computed to be:

$$n = (2.00 \times 10^5 \text{ MPa})/(2.50 \times 10^4 \text{ MPa}) = 8.00$$

ρ = $A_S/(bd)$ = $(645\ mm^2)/((1500\ mm)(100\ mm))$ = 0.0043
ρn = $(0.0043)(8.00)$ = 0.0344
k = $\sqrt{2(0.0344) + (0.0344)^2} - 0.0344$ = 0.23
j = $1 - 0.230/3$ = 0.923
f_S = $(0.690\ kN\text{-}m/m)(1.50\ m)/((645\ mm^2)(0.923)(0.10\ m))$
f_S = 17.4 MPa

Since $f_S \leq F_S$, the steel distribution is adequate. (OK)

Overall Displacements of the Wall

Per section 2.4.6, the construction-induced vertical and horizontal permanent displacements at the top of the wall can be expected to be on the order of 0.2% of the height of the wall, or about 20 to 25 mm, for the given site soil conditions (medium dense to dense silty sands). Displacements can be anticipated to decrease back from the wall in general accordance with the recommendations given on figure 2.8.

5.1.1.2 Seismic Loading Condition

In accordance with AASHTO [30], Service Load Group VII defines the seismic loading condition. Because of the temporary nature of the shotcrete construction facing, only the permanent facing is considered for the seismic loading condition.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

Step 1 is identical to that presented for the static loading condition.

Step 2 - Compute the Allowable Nail Head Loads

The nominal nail head strengths for all credible failure mechanisms are as calculated for the static condition appendix F. However, the allowable nail head loads are computed from the nominal strengths in the following table for the Group VII loading condition the permanent CIP concrete facing:

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load T_F (kN)
Facing Flexure	278	$0.89^a(278) = 247$
Facing Punching Shear	219	$0.89^a(219) = 195$
Headed-Stud Tensile Fracture	639	$0.67^a(639) = 428$

[a] Per section 3.22.2 of AASHTO [30].
See Table 4.4 for Group VII loading factors.

Therefore, the allowable nail head load T_F is computed to be 195 kN. That is, facing punching shear of the permanent facing is the controlling mode of failure.

Step 3 - Minimum Allowable Nail Head Service Load Check

This check is not applied to extreme event loading combinations.

Step 4 - Define the Allowable Nail Load Support Diagrams

Develop the allowable nail load diagram for each nail, by determining the allowable pullout resistance, the allowable nail head loads, and the allowable nail tendon tensile loads.

Allowable Pullout Resistance (Ground-Grout Bond)

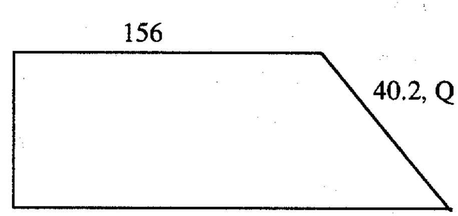
$$
\begin{aligned}
Q &= 1.33\alpha_Q Q_U \\
\alpha_Q &= 0.50 \\
Q_U &= 60.0 \text{ kN/m} \\
Q &= 0.67\,(60.0 \text{ kN/m}) \text{ (table 4.5)} \\
&= 40.2 \text{ kN/m}
\end{aligned}
$$

Allowable Nail Tendon Tensile Load

$$
\begin{aligned}
T_N &= 1.33\alpha_N T_{NN} \\
\alpha_N &= 0.55 \\
T_{NN} &= A_B F_Y = (510 \text{ mm}^2)(0.420 \text{ kN/mm}^2) = 214 \text{ kN} \\
T_N &= 0.73(214 \text{ kN}) \text{ (table 4.5)} \\
&= 156 \text{ kN}
\end{aligned}
$$

Allowable Nail Head Load

Per step 2, the allowable nail head load is 195 kN, which exceeds the allowable nail tendon load as shown below. The allowable nail tendon load therefore governs.



Step 5 - Select Trial Nail Lengths

The procedure to determine the nail distribution is the same as for the static case except that the Q_D value is revised to account for the 1.33 strength factor for Group VII loads.

- The dimensionless nail pullout resistance, Q_D, is calculated:

$$Q_D = 1.33\alpha_Q Q_U/(\gamma S_V S_H) = (0.67)(60.0\ \text{kN/m})/[(18\ \text{kN/m}^3)(1.50\ \text{m})(1.50\ \text{m})]$$
$$= 0.99$$

- The dimensionless nail length is:

$$L/H = (8.3\ \text{m})/(9.5\ \text{m}) = 0.87$$

- Q_D / (L/H) = 0.99/0.87 = 1.14, giving an "R" factor of 0.25.

- The trial nail length distribution is calculated from figure 4.11, for the nail head elevations and an "R" value of 0.25.

The allowable nail load support diagrams shown graphically in figure 5.4 were prepared in accordance with the procedure previously presented on figure 4.3.

Step 6 - Define the Ultimate Soil Strengths

Ultimate Friction Angle, $\phi_U = 34^\circ$
Ultimate Cohesion, $c_U = 5\ \text{kN/m}^2$

Step 7 - Calculate the Seismic Factor of Safety

In accordance with manual table 4.5 and section 3.22 of the Standard Specifications for Highway Bridges, 15th Edition [30], the required minimum global soil factor of safety for seismic loading is 1.35/1.33 = 1.01. An interactive computer solution for the design cross-section and nail pattern shown on figure 5.4 produced a calculated minimum factor of safety of 1.01 is obtained based on a computer solution for the following seismic loading conditions:

- For slip surfaces exiting the reinforced slope within a horizontal distance of 8.2 meters from the top of wall facing (determined from figure 5.1 in accordance with figure 4.14), the applied pseudo-static seismic coefficient for a 0.30 g peak ground acceleration was taken as:

$$A = (1.45 - A_{PK})A_{PK}$$
$$A = (1.45 - 0.30)(0.30\ g) = 0.345\ g$$

- For more deep-seated slip surfaces, an applied pseudo-static seismic coefficient of A = (1/2)(0.30 g) = 0.15 g was chosen, which is consistent with allowable overall

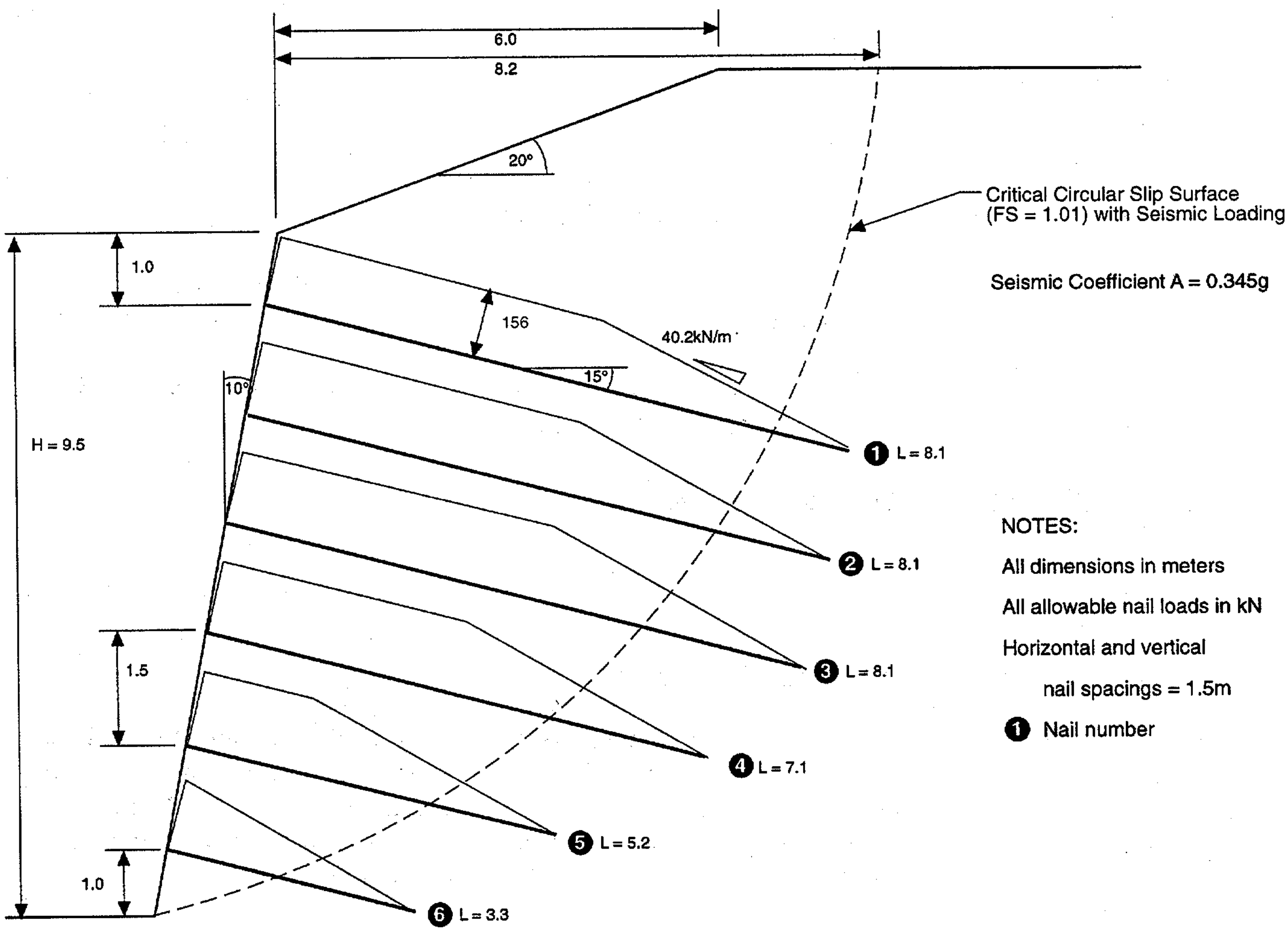


Figure 5.4 Cutslope Design Example Critical Cross-Section Seismic Loading (SLD)

displacements of up to 75 mm for the retaining wall system under design seismic loading.

Limiting-equilibrium analysis for seismic loading corresponding to a peak ground acceleration of 0.30 g has indicated that the same size nails as originally selected for the static design can be used, but the maximum nail lengths should be increased from 7.7 meters (for static loading) to 8.1 meters (for seismic loading), i.e., about a 5 percent increase in length. For this design example, seismic loading slightly controls the design.

Step 8 - External Stability Check of Nailed Block (Static and Seismic)

THE COMPUTATIONS FOR EXTERNAL STABILITY WHICH FOLLOW ARE SHOWN ONLY FOR ILLUSTRATIVE PURPOSES. As noted in the static computations, the check of static external stability is not generally required for granular soils. Also the seismic external stability is generally not checked unless the peak ground acceleration exceeds 0.25g as discussed in section 4.10.11.

For the design nail length of 8.1 m determined in step 7, the static and seismic bearing capacity checks are performed for the soil nail reinforced block of ground acting as a gravity wall structure (figure 4.12). Pertinent parameters for the analysis include the following:

Soil Unit Weight, γ = 18.0 kN/m^3
Ultimate Soil Friction Angle, ϕ_U = 34°
Ultimate Soil Cohesion, c_U = 5.0 kN/m^2
Seismic Coefficient, A = 0.15g

In accordance with section 6, Division I-A of the Standard Specifications for Highway Bridges, 15th Edition [30], the lateral forces due to the active block behind the reinforced soil are computed using the methods of Coulomb and Mononobe and Okabe (including the cohesive component of soil strength). Figure 5.5 shows the static and seismic earth pressure loads, inclined at an angle of 6.6 degrees above horizontal in accordance with the recommendations of figure 4.12. These are computed to be:

Static Active Earth Load, P_E	=	272.2 kN/m		
Horizontal component	=	$P_E \cos(6.6^o)$	=	270.4 kN/m
Vertical component	=	$P_E \sin(6.6^o)$	=	31.3 kN/m
Seismic Earth Load, P_{EQ}	=	112.1 kN/m		
Horizontal component	=	$P_{EQ} \cos(6.6^o)$	=	111.4 kN/m
Vertical component	=	$P_{EQ} \sin(6.6^o)$	=	12.9 kN/m

For moment equilibrium calculations, the assumed points of application of P_E and P_{EQ} are at the back of the reinforced block of ground at 0.33H and 0.6H above the base of the soil block, respectively:

Point of Application of P_E = 0.33H = 0.33(11.7 m) = 3.90 m

Point of Application of P_{EQ} = 0.6H = 0.6(11.7 m) = 7.02 m

Static Loading

For the static loading condition, Load Group I (table 4.3) controls. Referring to figure 5.5, forces and moments are computed below:

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_E(E)^a$	31.3	8.70	272.3
1 $(D)^a$	145.4	1.13	164.3
2 $(D)^a$	118.8	5.70	677.2
3 $(D)^a$	1026.0	4.70	4822.2
4 $(D)^a$	210.6	8.20	1726.9
Sum	1532.1 (N)	N/A	7662.9

Horizontal Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_E\ (E)^a$	270.4 (H)	3.90	-1054.6

[a] Notations used in the Standard Specifications for Highway Bridges, 15th Edition [30].

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

N = 1532.1 kN

X_O = (7662.9 kN-m/m- 1054.6 kN-m/m)/(1532.1 kN/m) = 4.31 m

q_{MAX} = $N/(2X_o)$ = (1532.1 kN/m)/[2(4.31 m)] = 177.6 kN/m^2

In accordance with equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30], the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) = 2(4.31 m) = 8.62 m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area

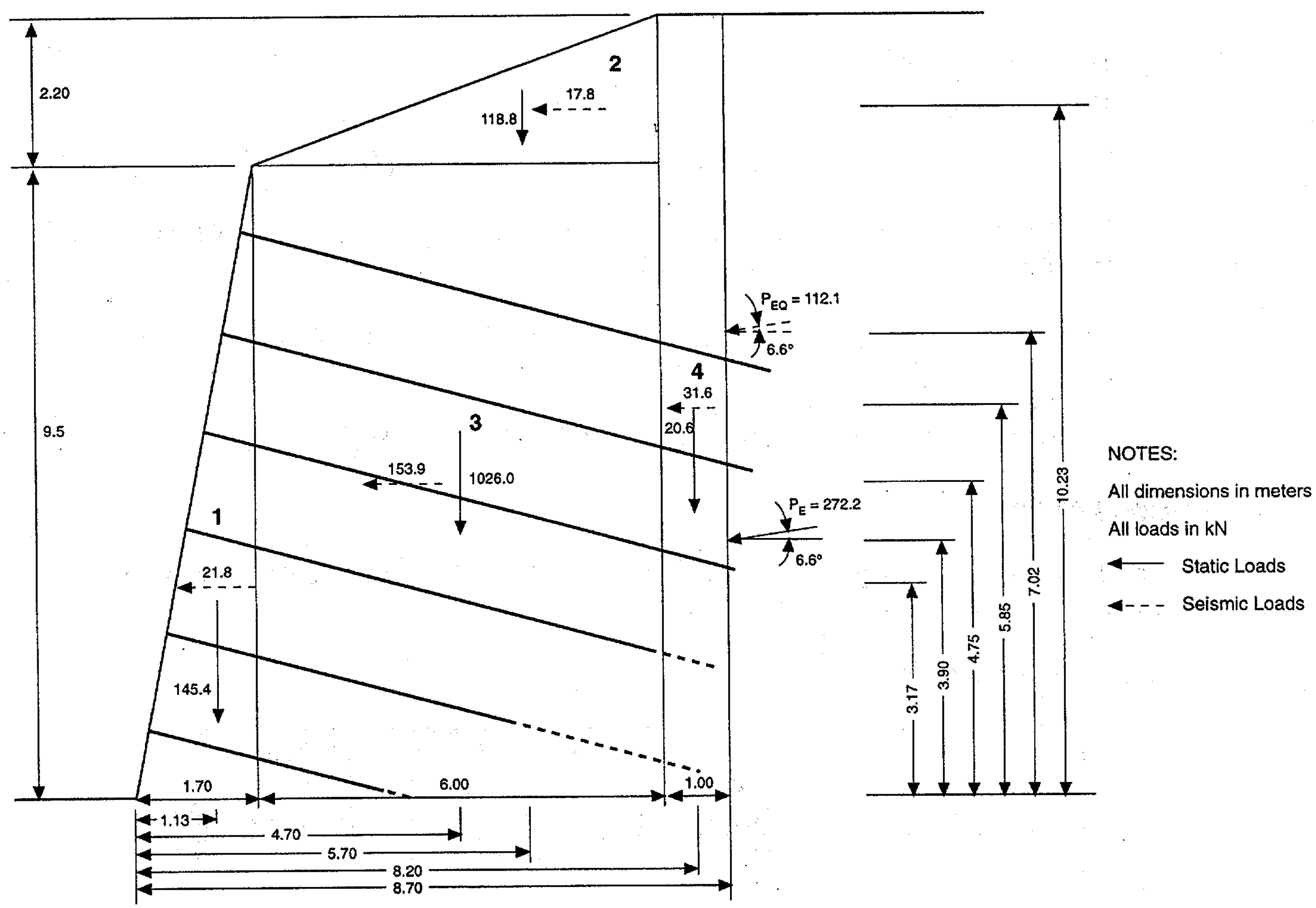


Figure 5.5. Cutslope Design Example External Stability Checks (SLD)

and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$$q_{ULT} = (5.0 \text{ kN/m}^2)(42.0)(1.06)(0.69) + 0.5(18.0 \text{ kN/m}^3)(8.62 \text{ m})(41.0)(0.97)(0.58)$$
$$q_{ULT} = 1{,}943 \text{ kN/m}^2$$

For a required factor of safety of 2.5 (see section 4.7.1, step 8), the allowable bearing pressure is:

$$q_{ALL} = (q_{ULT})/FS$$
$$q_{ALL} = (1{,}943 \text{ kN/m}^2)/2.5 = 777 \text{ kN/m}^2$$

Since $q_{MAX} < q_{ALL}$, the bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle third of the block of reinforced soil for static loading conditions:

$$e = B/2 - X_o = (8.7 \text{ m})/2 - 4.31 \text{ m} = 0.04 \text{ m}$$
$$B/6 = (8.7 \text{ m})/6 = 1.45 \text{ m}$$

Since $e < B/6$, the resultant is within the middle third. (OK)

Seismic Loading

For the seismic loading condition, Load Group VII (table 4.3) is of interest. The forces and moments are computed below (figure 5.5):

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
P_E(E)[a]	31.3	8.70	272.3
P_{EQ}(EQ)[a]	12.9	8.70	112.2
1 (D)[a]	145.4	1.13	164.3
2 (D)[a]	118.8	5.70	677.2
3 (D)[a]	1026.0	4.70	4822.2
4 (D)[a]	210.6	8.20	1726.9
Sum	1545.0 (N)	N/A	7775.1

Horizontal Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
P_E (E)[a]	270.4	3.90	-1054.6
P_{EQ} (EQ)[a]	111.4	7.02	-782.0
1 (EQ)[a]	21.8	3.17	-69.1
2 (EQ)[a]	17.8	10.23	-182.1
3 (EQ)[a]	153.9	4.75	-731.0
4 (EQ)[a]	31.6	5.85	-184.9
Sum	606.9 (H)	N/A	-3003.7

[a] Notations used in the Standard Specifications for Highway Bridges, 15th Edition [30].

Stability Criteria

Referring to figure 4.12, the maximum bearing pressure and the location of the resultant force are checked as follows:

$N = 1545.0$ kN
$X_O = (7775.1$ kN-m/m- 3003.7 kN-m/m$)/(1545.0$ kN/m$) = 3.09$ m
$q_{MAX} = N/(2X_o) = (1545.0$ kN/m$)/[2(3.09$ m$)] = 250.1$ kN/m^2

In accordance with equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30], the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) $= 2(3.09$ m$) = 6.18$ m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$q_{ULT} = (5.0$ kN/m$^2)(42.0)(1.04)(0.37) + 0.5(18.0$ kN/m$^3)(6.18$ m$)(41.0)(0.98)(0.24)$
$q_{ULT} = 617$ kN/m^2

For a required seismic factor of safety of 1.9 (see section 4.7.1, step 8), the allowable bearing pressure is:

$q_{ALL} = (q_{ULT})/FS$
$q_{ALL} = (617$ kN/m$^2)/1.9 = 325$ kN/m^2

Since $q_{MAX} < q_{ALL}$, the bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle third of the block of reinforced soil:

$$e = B/2 - X_o = (8.70\text{ m})/2 - 3.09\text{ m} = 1.26\text{ m}$$
$$B/6 = (8.70\text{ m})/6 = 1.45\text{ m}$$

Since e < B/6, the resultant is within the middle third. (OK)

Overall Stability

For the uniform nail length pattern selected (see figure 5.6), the deep ground water table, and the overall geometry of the slope, deep seated failure surfaces passing beneath the toe of the wall will not be critical.

Step 9 - Check the Upper Cantilever

In accordance with section 6 Commentary, division I-A of the Standard Specifications for Highway Bridges, 15th Edition [30], equation C6-4, the approach developed by Mononobe and Okabe for a free-standing retaining structure is used to develop the active seismic loading on the upper cantilever. For a soil friction angle of 34°, zero cohesion (ignore it), a soil-to-wall interface friction angle of (2/3)34° = 22°, a wall batter of 10°, a backslope angle of 20°, and a horizontal pseudo-static seismic coefficient of 0.15 g (i.e. $0.5A_{PK}$), the combined static and dynamic active earth pressure coefficient is calculated by the Mononobe and Okabe method to be 0.429. This can be considered to consist of a static earth pressure coefficient of 0.247 (see step 7 for the static loading condition) and a dynamic earth pressure coefficient of 0.182. The load components normal to the wall have corresponding earth pressure coefficients of (0.247)cos(22°) = 0.229 for the static load and (0.182)cos(22°) = 0.169 for the dynamic load.

Shear Check

From force equilibrium, compute the one-way unit service shear force at the level of the upper row of nails (conservative), as indicated on figure 4.13:

$$v = v_{STATIC} + v_{DYNAMIC}$$
$$v = 0.5(0.229 + 0.169)(18.0\text{ kN/m}^3)(1.0\text{ m})^2$$
$$v = 2.06\text{ kN/m} + 1.52\text{ kN/m} = 3.58\text{ kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 8-49 of AASHTO [30]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28\text{ MPa}}\,(0.1\text{ m}) = 87.8\text{ kN/m}$$

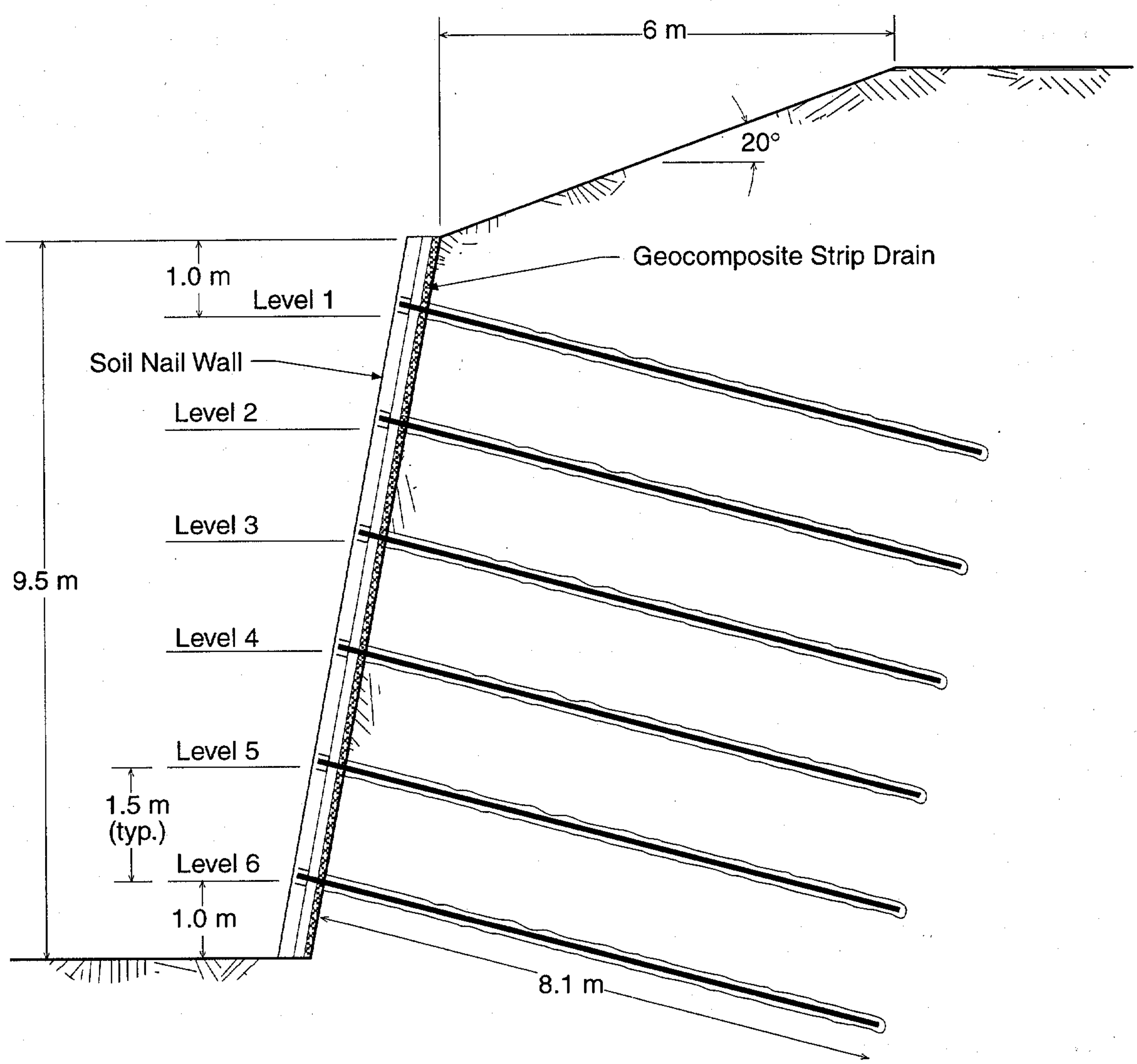


Figure 5.6 Cutslope Final Design Section (SLD)

From table 4.4, the facing shear strength factor equals 0.89. Therefore, the allowable one-way unit shear is computed to be:

$$V = (0.89)(87.8 \text{ kN/m}) = 78.1 \text{ kN/m}$$

Since $v < V$, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the one-way unit service moment at the level of the upper row of nails (conservative) as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever. The dynamic force is assumed to occur at 0.6H above the base of the cantilever:

$$m_S = [(0.33)(v_{STATIC}) + (0.6)(v_{DYNAMIC})](H)/\cos(10^\circ)$$
$$m_S = [(0.33)(2.06 \text{ kN/m}) + (0.6)(1.52 \text{ kN/m})](1.0 \text{ m}) /\cos(10^\circ)$$
$$m_S = 1.62 \text{ kN-m/m}$$

Compute the nominal unit moment resistance of the facing. From appendix F, the nominal unit moment resistance in the vertical direction over the nail locations, $m_{V,NEG}$, is computed to be 17.4 kN-m/m. From table 4.4, the (seismic loading) strength factor for facing flexure is 0.89. Therefore, the allowable one-way unit moment for the upper cantilever is:

$$M = (0.89)\, m_{V,NEG} = (0.89)(17.4 \text{ kN-m/m}) = 15.5 \text{ kN-m/m}$$

Since $m_S < M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

The facing reinforcement details, being independent of type of loading, have been previously considered for the static loading condition and need not be repeated for the seismic loading condition.

Step 11 - Serviceability Checks

The serviceability checks are not applicable to seismic loading conditions because of the extreme nature of the limit state.

Final Design Section for Contract Plans

Based on the above analyses, the final design cross-section is shown on figure 5.6. It should be noted that, to simplify for construction, the chosen final plan nail lengths are uniform, independent of the nail length distributions used in the analyses. The uniform nail lengths equal the maximum nail length indicated by the above analyses. Based on the designer's discretion, and subject to overall stability analyses, the nails in the bottom three rows could be shortened, taking into account any constructability issues and/or economic savings related to the use of varying nail lengths.

5.1.2 Load and Resistance Factor Design (LRFD)

5.1.2.1 Static Loading Condition

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Strength I Limit State (table 4.6) defines the static loading condition for this problem.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

The technical and cost feasibility of a soil nail solution for a road cut through medium dense silty sands has been established. The location is remote from any buried utilities and there are no adjacent structures that will be impacted by the modest settlements that would be associated with construction of a soil nail wall within these materials. The ground water table is below the base of the planned cut, which has a maximum wall height of 9.50 meters, but minor perched ground water is expected and provision will be made for intercepting any seepage to the face and conducting the collected water to a footing drain through the use of geotextile strip face drains. The seismic design condition corresponds to a peak ground acceleration of 0.30 g.

The site investigation consisted of soil borings with standard penetration testing, laboratory classification testing, test pitting, and in-situ density measurements. The investigation confirmed the subsurface soil and ground water conditions, and established that 2.5 meter high vertical cuts will stand unsupported for a minimum of several days. The soil profile consists of medium dense to dense silty sand. Average in-situ densities are 18.0 kN/m^3, and the soil strength parameters are estimated at a friction angle of 34.0 degrees and a cohesion of 5.0 kN/m^2.

Although no nail pullout testing was performed during the site investigation, previous experience suggested that an ultimate pullout resistance on the order of 60.0 kN/m should be achievable with drillhole diameters on the order of 200 mm. This pullout resistance will have to be demonstrated for the contractor's proposed installation method during the initial stages of the wall construction. Although the ground has not been determined to be aggressive in nature, the cut is adjacent to a major lifeline transportation corridor, and encapsulated nails will be used for corrosion protection. The site investigation has also confirmed that there will be no requirement for horizontal drains to address possible seasonal increases in the ground water table as the ground water table is located well below the base of the proposed wall.

The wall will have a maximum vertical height of 9.50 meters, with a face batter of 10.0 degrees from vertical. The critical design cross section is shown on figure 5.7A and will have a 20.0 degree slope at the top of the wall, as shown. The nail spacing will be at 1.50 meters, vertically and horizontally, and the nails installed at 15.0 degrees below horizontal for constructability reasons.

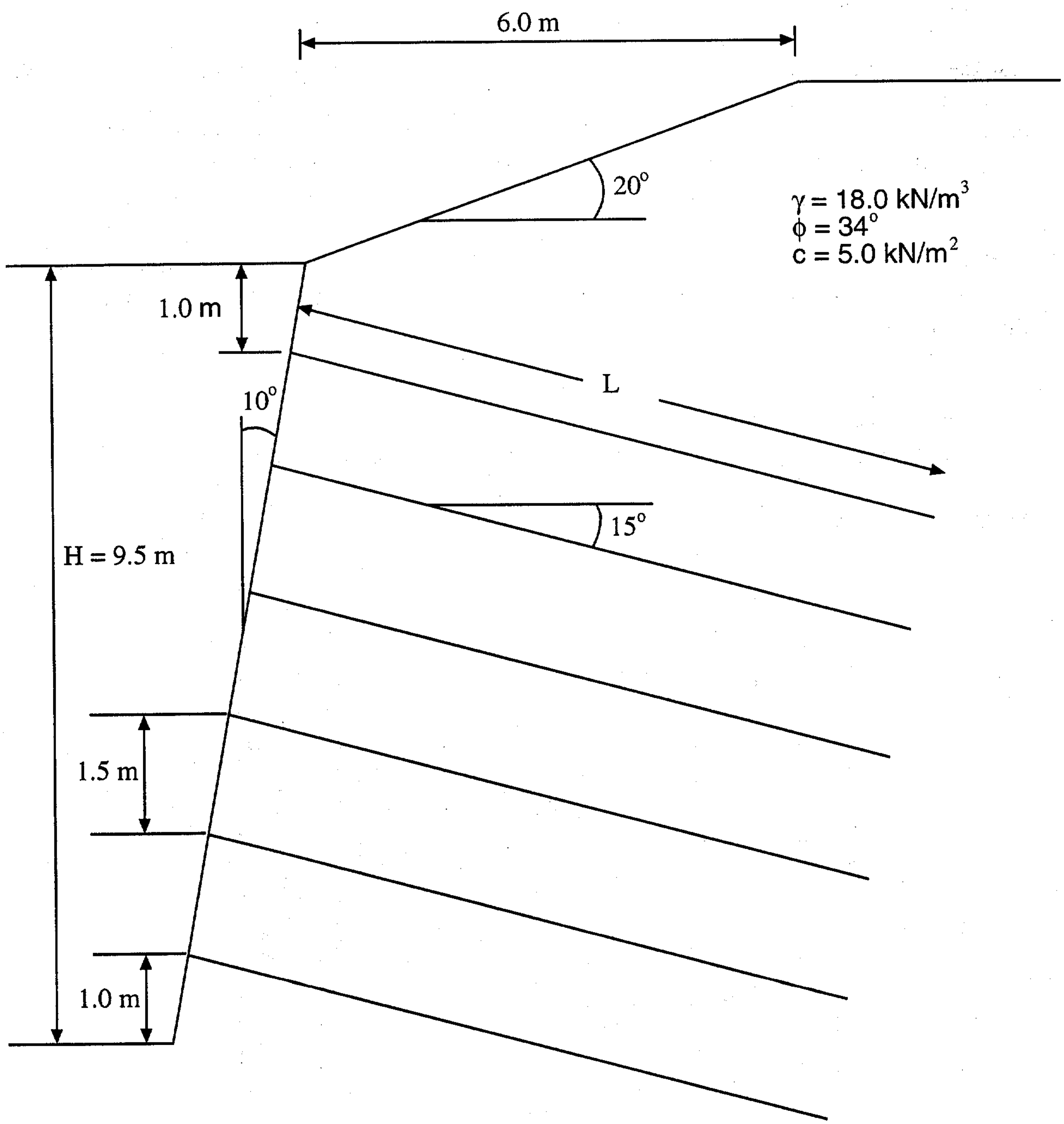


Figure 5.7A Cutslope Design Example

Based on the design charts presented in section 5.3, it is anticipated that No. 25 Grade 420 bars (Soft Metric designation - see table 4.8A), approximately 8.4 meters long, will be required for support of the cut. Furthermore, based on local practice and material availability, the trial design will assume use of a "standard" temporary shotcrete construction facing (28-day compressive strength of 28 Mpa) having a nominal thickness of 100 mm reinforced with a single layer of 152x152 MW18.7xMW18.7 (6x6 - W2.9xW2.9) welded-wire mesh, two No. 13 Grade 420 continuous horizontal waler bars along each row of nails, and two No. 13 vertical bearing bars at each nail head.

Use the preliminary design charts in section 4.12 to determine the preliminary values for nail length and nail bar size.Select the design chart set corresponding to the appropriate backslope angle. If necessary, interpolate results for intermediate backslope angles to those given in the charts. Figure 5.7A shows that the design section has a face batter of 10° and a backslope angle of 20°. Therefore use the design chart set presented in figure 4.31 and 4.32.

Compute the factored soil friction angle ϕ_D and the dimensionless factored soil cohesion c_D as defined above (equations 4.0A and 4.0C). From the appropriate Chart A, determine the dimensionless nail tensile capacity T_D.

$$\phi_D = \tan^{-1}[\Phi_\phi \tan(\phi_U)] = \tan^{-1}[0.75\tan(34°)] = 26.8°$$
$$\tan(\phi_D) = \tan(26.8°) = 0.51$$
$$c_D = \Phi_c c_U/(\Gamma_W \gamma H) = (0.9)(5.0\ \text{kN/m}^2)/[1.35(18.0\ \text{kN/m}^3)(9.50\ \text{m})] = 0.019$$

From Chart A (figure 4.31), $T_D = 0.23$

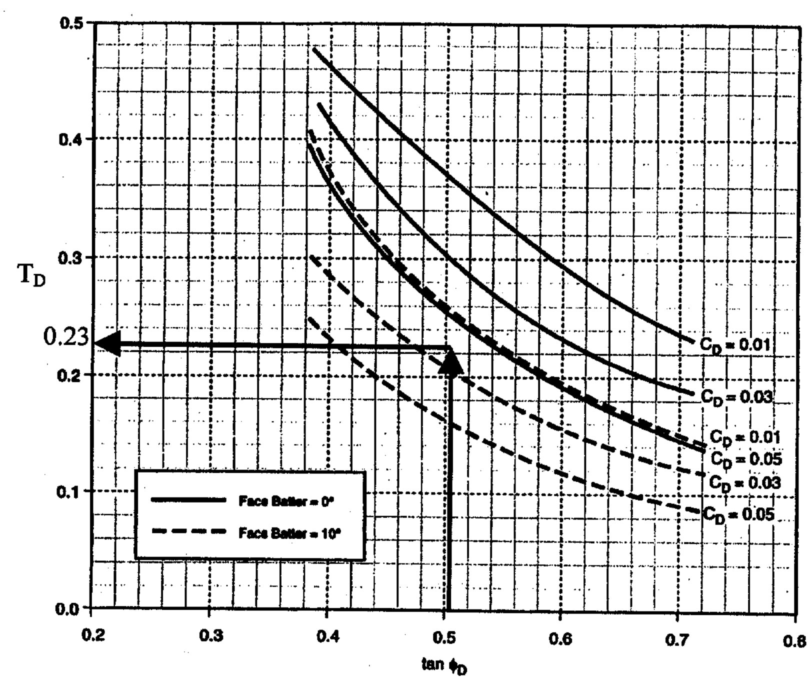
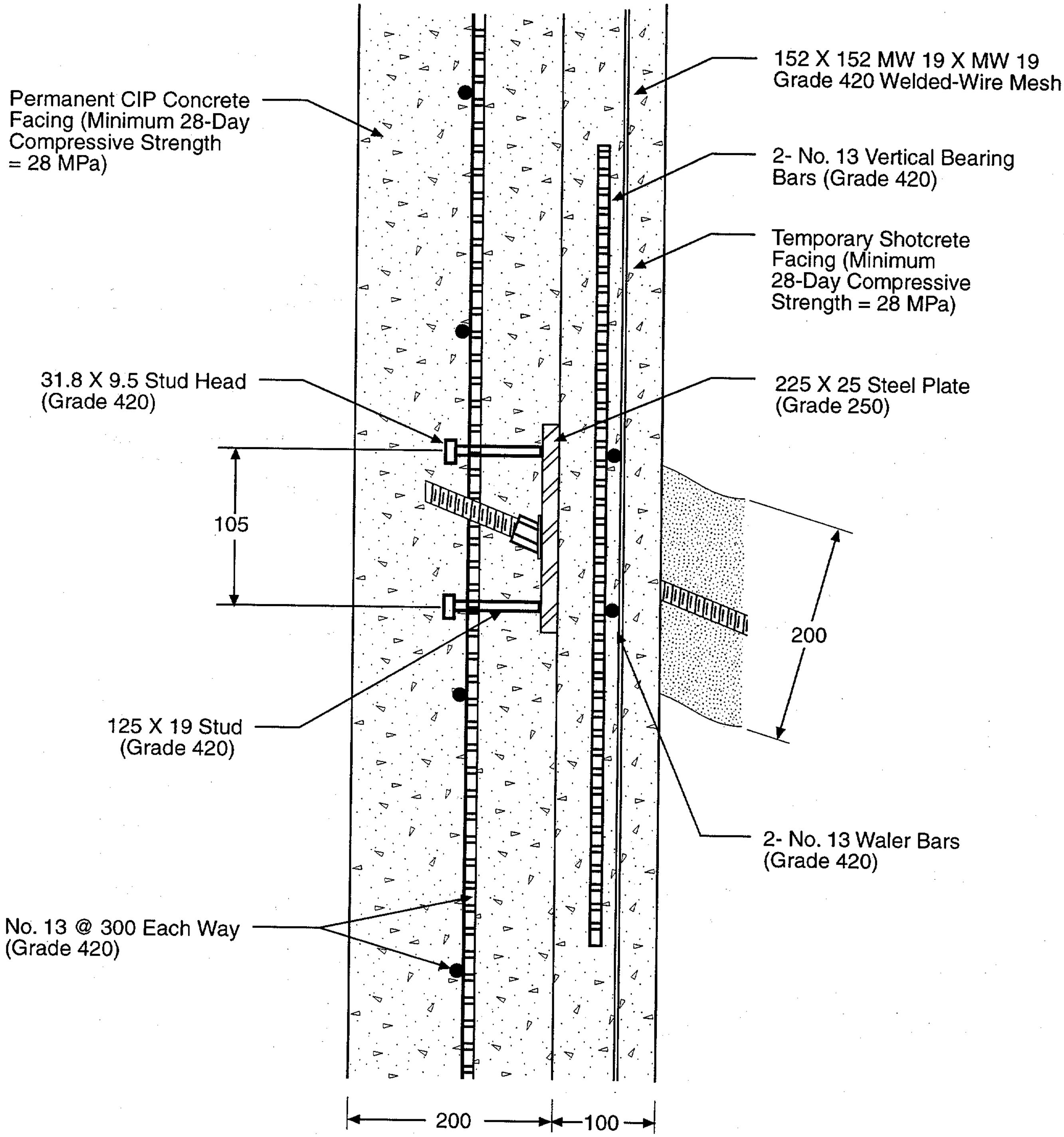


Figure 4.31 **Preliminary Design Chart 3A Backslope = 20° (Chart A)**



All dimensions in millimeters

Figure 5.7B Facing Details

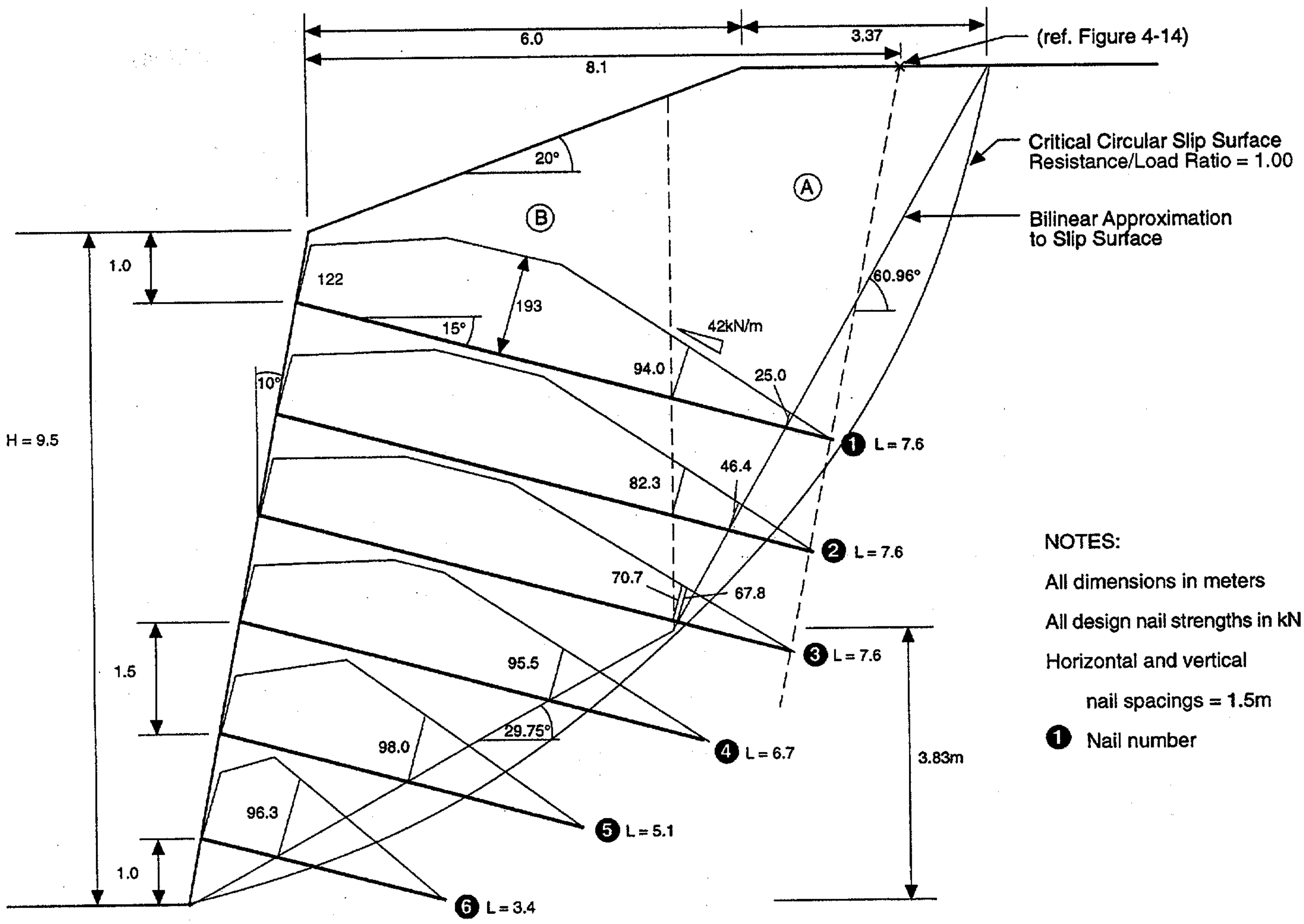


Figure 5.7C Cutslope Design Example
Trial Design Critical Cross-Section Static Loading (LRFD)

Based on soil and constructibility considerations as well as local practice, the vertical and horizontal nail spacings have both been selected to be 1.50 meters. The connection plate will be 225 mm wide and 25 mm thick. The facing reinforcement is selected by considering the various available mesh sizes, local practice, and strength requirements. Try a 152x152-MW19xMW19 mesh (steel area = 122.8 mm^2/m, table 4.1A), with two No. 13 (Soft metric designation) waler bars and two No. 13 vertical bearing bars (steel area = 129 mm^2, table 4.8A). All facing steel is assumed to be nominally located at the center of the section. The yield stress of the reinforcement is specified as 420 MPa and the specified design concrete compressive strength at 28 days is 28 MPa.

The first step to evaluate equation 4.1A is to compute the negative and positive nominal unit moment resistances of the facing in the vertical direction. The equation for the nominal unit moment resistance of a singly-reinforced, rectangular-shaped reinforced-concrete beam is as follows [1]:

$$m = \frac{A_S F_Y}{b}\left(d - \frac{A_S F_Y}{1.70 f'_c\, b}\right)$$

The areas of vertical steel over the supports (2 No. 13 vertical bars and mesh vertical wires) and at midspan (mesh vertical wires) for a facing width b equal to 1.5 m, are computed as follows:

$$A_{S,NEG} = (122.8\ mm^2/m)(1.5m) + 2(129\ mm^2) = 443\ mm^2$$

$$A_{S,POS} = (122.8\ mm^2/m)(1.5m) = 185\ mm^2$$

The corresponding average nominal unit moment resistances are computed as indicated below:

$$m_{V,NEG} = \frac{(443mm^2)(420MPa)}{1500mm}\left(50.0mm - \frac{(443mm^2)(420MPa)}{1.70(28.0MPa)(1500mm)}\right)$$

$$= 5.88\ kN\text{-}m/m$$

$$m_{V,POS} = \frac{(185mm^2)(420MPa)}{1500mm}\left(50.0mm - \frac{(185mm^2)(420MPa)}{1.70(28.0MPa)(1500mm)}\right)$$

$$= 2.53\ kN\text{-}m/m$$

From table 4.2, the facing flexure pressure factor C_F for a 100 mm thick temporary facing is 2.0. Substituting the corresponding values into, equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as follows:

$$T_{FN} = C_F(m_{v,neg}+m_{v,pos})(8)(S_H)/(S_V)$$

T_{FN} = 2.0(5.88 kN-m/m + 2.53 kN-m/m)(8)(1.50 m)/(1.50 m) = 135 kN

(b) Strength Criteria 2: Facing Punching Shear (section 4.5.3, equation 4.3)

The nominal internal punching shear strength of the facing is computed from equation 4.2, where h_C and D'_C are as indicated below:

h_C = 100 mm

$D'_C = b_{PL} + h_C$ = 225 mm + 100 mm = 325 mm

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_N = 0.33\sqrt{28.0\text{MPa}}\,(\pi)(325\text{mm})(100\text{mm}) = 178\text{kN}$$

From table 4.2, the pressure factor for punching shear C_S for a 100 mm thick temporary facing is 2.5. The punching cone bottom diameter D_C is equal to $D'_C + h_C$ = 425 mm. The diameter of the grout column is estimated to be about 200 mm. The corresponding areas are computed as follows:

$A_C = 0.25(\pi)(D_C)^2 = 0.25(\pi)(425\text{ mm})^2 = 1.42 \times 10^5\text{ mm}^2$

$A_{GC} = 0.25(\pi)(D_{GC})^2 = 0.25(\pi)(200\text{ mm})^2 = 3.14 \times 10^4\text{ mm}^2$

Substituting the above values into equation 4.3, the nominal nail head strength for the criteria of punching shear is computed as follows:

$$T_{FN} = (178\text{kN})\left(\frac{1}{1-2.5\dfrac{(1.42\text{x}10^5\text{mm}^2 - 3.14\text{x}10^4\text{mm}^2)}{[(1500\text{mm})(1500\text{mm}) - 3.14\text{x}10^4\text{mm}^2]}}\right) = 204\text{kN}$$

Permanent Cast-In-Place (CIP) Concrete Facing

(a) Strength Criteria 1: Facing Flexure (section 4.5.2, equation 4.1)

The same nail spacings and connection plate dimensions used for temporary facing apply to the permanent facing. Based on considerations including corrosion, durability, and quality of construction practice, the temporary facing is neglected in the permanent design. The thickness of the permanent facing is selected to be 200 mm.

By considering the available steel bar sizes and local practice, the facing reinforcement is selected to be No. 13 deformed bars at 300 mm centers in both the horizontal and vertical directions. All facing steel is assumed to be nominally located at the center of the section. The design yield stress of the reinforcement is again specified as 420 MPa and the specified 28-day design concrete compressive strength is 28.0 MPa.

The first step to evaluate equation 4.1 is to compute the negative and positive nominal unit moment resistances of the facing in the vertical direction. The areas of vertical steel over the supports and at the midspan for a width of 1500 mm are computed as follows:

$$A_{S,NEG} = A_{S,POS} = (129\ \text{mm}^2)(1500\ \text{mm})/(300\ \text{mm}) = 645\ \text{mm}^2$$

Using equation 4.1A, the average nominal unit moment resistances are computed as follows:

$$m_{V,NEG} = \frac{(645\text{mm}^2)(420\text{MPa})}{1500\text{mm}}\left(100\text{mm} - \frac{(645\text{mm}^2)(420\text{MPa})}{1.70(28.0\text{MPa})(1500\text{mm})}\right)$$

$$= 17.4\ \text{kN-m/m}$$

$$m_{V,POS} = m_{V,NEG} = 17.4\ \text{kN-m/m}$$

From table 4.2, the facing flexure pressure factor C_F for a 200 mm thick permanent facing is 1.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as follows:

$$T_{FN} = 10(17.4\ \text{kN-m/m} + 17.4\ \text{kN-m/m})(8)(1.50\ \text{m})/(1.50\ \text{m}) = 278\ \text{kN}$$

(d) Strength Criteria 2: Facing Punching Shear (section 4.5.3, equation 4.3)

The dimensions of the headed studs must be selected in order to calculate the geometry of the potential punching cone. Per Section 4.5.4, the headed-stud dimensions are first selected to satisfy the provisions of ACI Committee 349 [2]. Try a 22.0 mm body diameter, a 35.0 mm head diameter, a 9.5 mm head thickness, an overall length of about 125 mm (corresponding to a $^7/_8$ x $5^3/_{16}$ anchor size, table 4.2A), and a stud spacing of 105 mm. The provisions are checked as shown below:

<u>Provision 1:</u>	d_H	$\geq (2.5)^{0.5}(22.0\ \text{mm})$	
	35.0 mm	$\geq (2.5)^{0.5}(22.0\ \text{mm})$	
	35.0 mm	≥ 35.0 mm	(O.K.)
<u>Provision 2:</u>	t_H	$\geq 0.5\ (d_H - d_{HS})$	
	9.5 mm	$\geq 0.5(35.0\ \text{mm} - 22.0\ \text{mm})$	
	9.5 mm	≥ 6.5 mm	(O.K.)

The nominal internal punching shear strength of the facing is computed from equation 4.2, where h_C and D'_C are indicated below:

h_C = plate thickness + stud length

$h_C = 25 \text{ mm} + 125 \text{ mm} = 150 \text{ mm}$

$D'_C = S_{HS} + h_C = 105 \text{ mm} + 150 \text{ mm} = 255 \text{ mm}$

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_N = 0.33\sqrt{28.0\text{MPa}}(\pi)(255\text{mm})(150\text{mm}) = 210\text{kN}$$

From table 4.2, the pressure factor for punching shear for a 200 mm thick permanent facing is 1.0. The punching cone bottom diameter D_C is equal to $D'_C + h_C = 405$ mm. The diameter of the grout column is estimated to be about 200 mm. The corresponding areas are computed as follows:

$A_{GC} = 0.25(\pi)(D_{GC})^2 = 0.25(\pi)(200 \text{ mm})^2 = 1.29 \times 10^5 \text{ mm}^2$

Substituting the above values into equation 4.3, the nominal nail head strength for the criteria of punching shear is computed as follows:

$$T_{FN} = (210\text{kN})\left(\frac{1}{1-1.0\dfrac{(1.29\text{x}10^5\text{ mm}^2 - 3.14\text{x}10^4\text{ mm}^2)}{[(1500\text{mm})(1500\text{mm}) - 3.14\text{x}10^4\text{ mm}^2]}}\right) = 219\text{kN}$$

(c) Strength Criteria 3: Headed-Stud Tension (section 4.5.6, equation 4.4)

The nominal nail head strength associated with the criteria of headed-stud tension is computed by equation 4.4. For 22.0 mm diameter studs and a specified headed-stud ultimate tensile stress of 420 MPa, the nominal nail head strength is computed to be:

$T_{FN} = 4(0.25)(\pi)(22.0 \text{ mm})^2(420 \text{ MPa}) = 639 \text{ kN}$

The nominal nail head strengths for all credible failure mechanisms are calculated in appendix F and tabulated below for both the shotcrete and CIP facings. The design nail head strengths are computed from the nominal strengths as indicated in the tables below for both the temporary shotcrete construction facing and the permanent CIP concrete facing.

SHOTCRETE CONSTRUCTION FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Nail Head Strength T_F (kN)
Facing Flexure	135	$0.90^a(135) = 122$
Facing Punching Shear	204	$0.90^a(204) = 184$

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Nail Head Strength T_F (kN)
Facing Flexure	278	$0.90^a(278) = 250$
Facing Punching Shear	219	$0.90^a(219) = 197$
Headed-Stud Tensile Fracture	639	$0.67^a(639) = 428$

[a] See Table 4.7.

Therefore, the design nail head strength T_F is computed to be 122 kN. That is, facing flexure of the temporary facing is the controlling mode of failure.

Step 3 - Minimum Design Nail Head Strength Check

Since the design problem has a relatively simple geometry and uniform soil conditions, the active earth pressure coefficient can be analytically determined from a Coulomb analysis (AASHTO, 1st Edition, 1994 - section 3.11.5.3 [29]). For a friction angle of 34 degrees, a face slope angle of 10 degrees from vertical and a backslope angle of 20 degrees, the active earth pressure coefficient corresponding to a horizontal, triangular earth pressure distribution is given by K_A equal to 0.257 (ignoring the cohesive component of soil strength in accordance with the discussion of section 2.4.5). Values for other parameters are as follows:

$$S_H = S_V = 1.50\text{m}$$
$$H = 9.50\text{ m}$$
$$F_F = 0.50$$

Substituting the above values into equation 4.6, to determine the nail head service load:

$$t_F = F_F K_A \gamma H S_H S_V = (0.50)(0.257)(18\text{ kN/m}^3)(9.5\text{ m})(1.50\text{ m})^2$$
$$t_F = 49\text{ kN}$$

The factored nail head service load is computed using the load factor from table 4.6 (equal to 1.5 for active earth pressure), and is checked against the design nail head strength as indicated below:

$$\Gamma_{EH}(t_F) = 1.5(49.0\ \text{kN}) = 73.5\ \text{kN} < 122\ \text{kN}$$

(OK - the design nail head strength exceeds the estimated factored nail head service load)

Step 4 - Define the Design Nail Strength Diagrams

Develop the design nail strength diagram for each nail by determining the design pullout resistance, the design nail head strength and the design nail tendon tensile strength.

Design Nail Head Strength, T_F

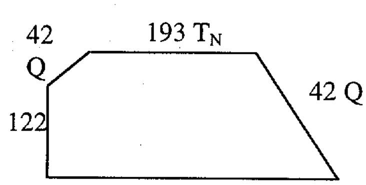
Per step 2, the design nail head strength is 122 kN.

Design Pullout Resistance, Q, (Ground-Grout Bond)

$$
\begin{aligned}
Q &= \Phi_Q Q_U \\
\Phi_Q &= 0.70 \quad \text{(table 4.8)} \\
Q_U &= 60.0\ \text{kN/m} \\
Q &= (0.70)(60.0\ \text{kN/m}) = 42.0\ \text{kN/m}
\end{aligned}
$$

Design Nail Tendon Tensile Strength

$$
\begin{aligned}
T_N &= \Phi_N T_{NN} \\
\Phi_N &= 0.90 \quad \text{(table 4.8)} \\
T_{NN} &= A_B F_Y = (510\ \text{mm}^2)(0.420\ \text{kN/mm}^2) = 214\ \text{kN} \\
T_N &= (0.90)(214\ \text{kN}) = 193\ \text{kN}
\end{aligned}
$$



Step 5 - Select Trial Nail Spacings and Lengths

In step 1, a preliminary nail length of 8.4 m at a horizontal and vertical spacing of 1.5 m was selected. However as noted in section 4.7.1, this length only represents the nail length in the upper half of the wall. The nail lengths in the lower half of the wall need to be artificially shortened prior to performing a limiting equilibrium analysis in order that the upper nail lengths are adequate to resist the anticipated loads at small deflections. Figure 4.11 is used as follows to determine the distributions of nail lengths with depth.

- The dimensionless nail pullout resistance, Q_D, is calculated:

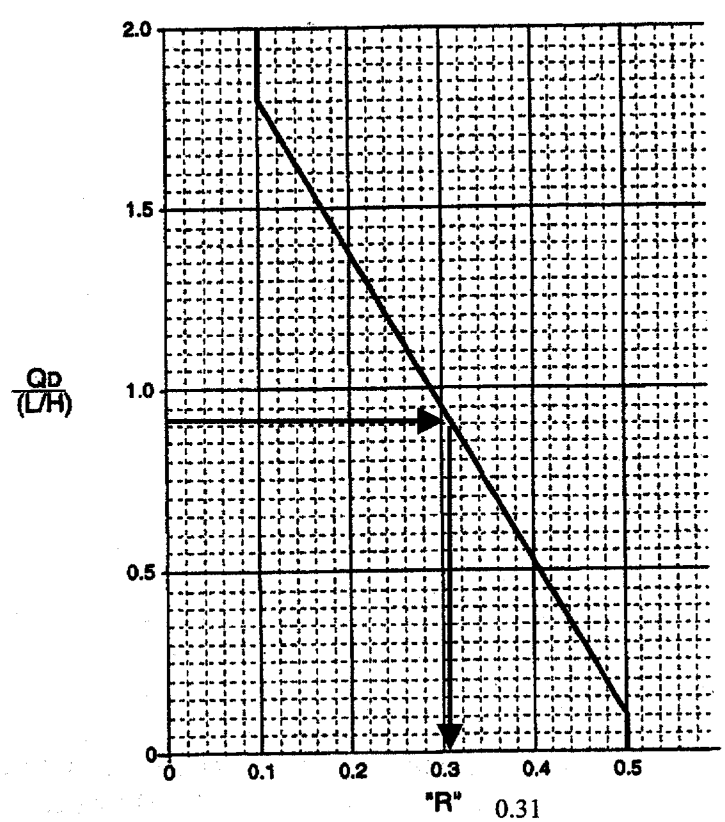
  $$Q_D = \Phi_Q Q_U/(\Gamma_W \gamma S_V S_H) = (0.70)(60.0\ \text{kN/m})/[(1.35)(18\ \text{kN/m}^3)(1.50\ \text{m})(1.50\ \text{m})] = 0.77$$

- The dimensionless nail length is:

  $$L/H = (8.4\ \text{m})/(9.5\ \text{m}) = 0.88$$

- Q_D / (L/H) = 0.74/0.88 = 0.88, giving an "R" factor of 0.31.

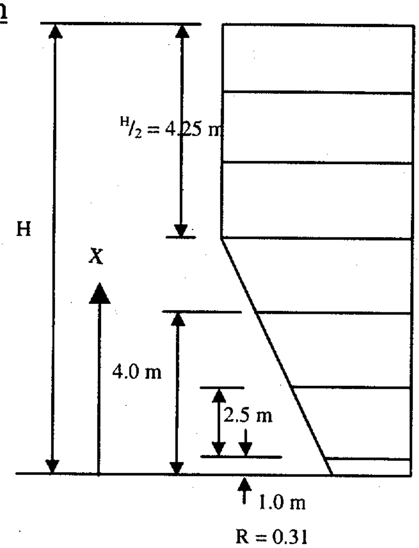
- Relative nail lengths are calculated from figure 4.11 for the nail head elevations shown on figure 5.1 and an "R" value of 0.31.



L = Maximum Nail Length
H = Wall Height
Q_D = Dimensionless Pullout Resistance
= α_Q Qu/($\gamma S_H S_v$) (SLD)
= Φ_Q Qu/($\Gamma_W \gamma S_H S_v$) (LRFD)

where
α_Q = pullout resistance strength factor (SLD)
Φ_Q = pullout resistance factor (LRFD)
Qu = ultimate pullout resistance
γ = unit weight
S_H = horizontal nail spacing
S_v = vertical nail spacing
Γ_W = soil weight load factor (LRFD)

Nail No.	Trial Length,m	Rx	Trial Nail Length Distribution
1	8.4	1.0	8.3
2	8.4	1.0	8.3
3	8.4	1.0	8.3
4	8.4	0.89	7.5
5	8.4	0.67	5.6
6	8.4	0.46	3.9



Where $Rx = {}^{x}/_{(H/2)}\ (1\text{-}R) + R$

Step 6 - Define the Design Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0°
Ultimate Cohesion, c_U= 5.0 kN/m^2

From table 4.8, the soil resistance factors are as follows:

Φ_ϕ = 0.75
Φ_C = 0.90

Design Friction Angle, $\phi = \tan^{-1}(\Phi_\phi \tan(\phi_U)) = \tan^{-1}(0.75[\tan(34.0^\circ)]) = 26.8^\circ$
Design Cohesion, $c = \Phi_C c_U = 0.90(5.0\ \text{kN/m}^2) = 4.5\ \text{kN/m}^2$

Step 7 - Calculate the Resistance/Load Ratio

A limiting equilibrium analysis is performed using appropriate computer software to determine the actual nail lengths which are required for a minimum resistance / load ratio of 1.00 (table 4.8 Group I loading). The nail length distribution for design purposes and the allowable nail load support diagrams are shown on figure 5.7C. Note that the maximum nail length has been calculated iteratively to be 7.6 meters. This is slightly less than the 8.4 estimated from the design charts, partially due to the broken backslope adjacent to the soil nail wall giving an "effective" backslope angle less than 20 degrees assumed in the design chart calculations.

Hand Calculation Check

In general, it will not be necessary to perform extensive hand calculation checks if a properly verified computer program is used. The following provides a method for performing hand calculation checks when they are required or when a verification check of a computer solution is desired. The method is approximate in that it is based on force balance only and more reliable results will generally be obtained using methods that address both force and moment equilibrium. In order to facilitate the hand calculation check, the critical circular slip surface may be approximated by a bilinear wedge, as shown on figure 5.7C. The forces on the 'active' Block A and the 'resisting' Block B are illustrated on figure 5.8 for the bilinear wedge and are computed below:

Block A

Base Slope Angle, α_A = 60.96°
Base Length, L_A = 9.00 m
Block Weight, W_A = $(17.01\ m^2)(18.0\ kN/m^3)$ = 306.2 kN/m
Soil Weight Load Factor, Γ_W =1.35 (table 4.6)
Factored Soil Block Weight, $\Gamma_W W_A$ = 1.35(306.2 kN/m) = 413.4 kN/m
Design Nail Force, T_A = $[(25.0\ kN)^{(1)} + (46.4\ kN)^{(2)} + (67.8\ kN)^{(3)}]/(1.50\ m)$
= 92.8 kN/m

$^{(n)}$ Nail Number

Block B

Base Slope Angle, α_B = 29.75°
Base Length, L_B = 7.72 m
Block Weight, W_B = $(47.3\ m^2)(18.0\ kN/m^3)$ = 851.8 kN/m
Soil Weight Load Factor, Γ_W =1.35 (table 4.6)
Factored Soil Block Weight, $\Gamma_W W_B$ = 1.35(851.8 kN/m) = 1149.9 kN/m
Design Nail Force, T_B = $[(95.5\ kN)^{(4)} + (98.0\ kN)^{(5)} + (96.3\ kN)^{(6)}]/(1.5\ m)$
= 193.2 kN/m

Interslice

Design Nail Force, T = $[(94.0\ kN)^{(1)} + (82.3\ kN)^{(2)} + (70.7\ kN)^{(3)}]/(1.5\ m)$
= 164.7 kN/m

Assume that the interslice soil force, I, is inclined at an angle 'β' to the horizontal (figure 5.8). The resistance/load ratio (R/L) is then applied to the design soil strengths, as indicated below.

Equilibrium

Define $\phi = \tan^{-1}[\Phi_\phi \tan(\phi_U)/(R/L)]$

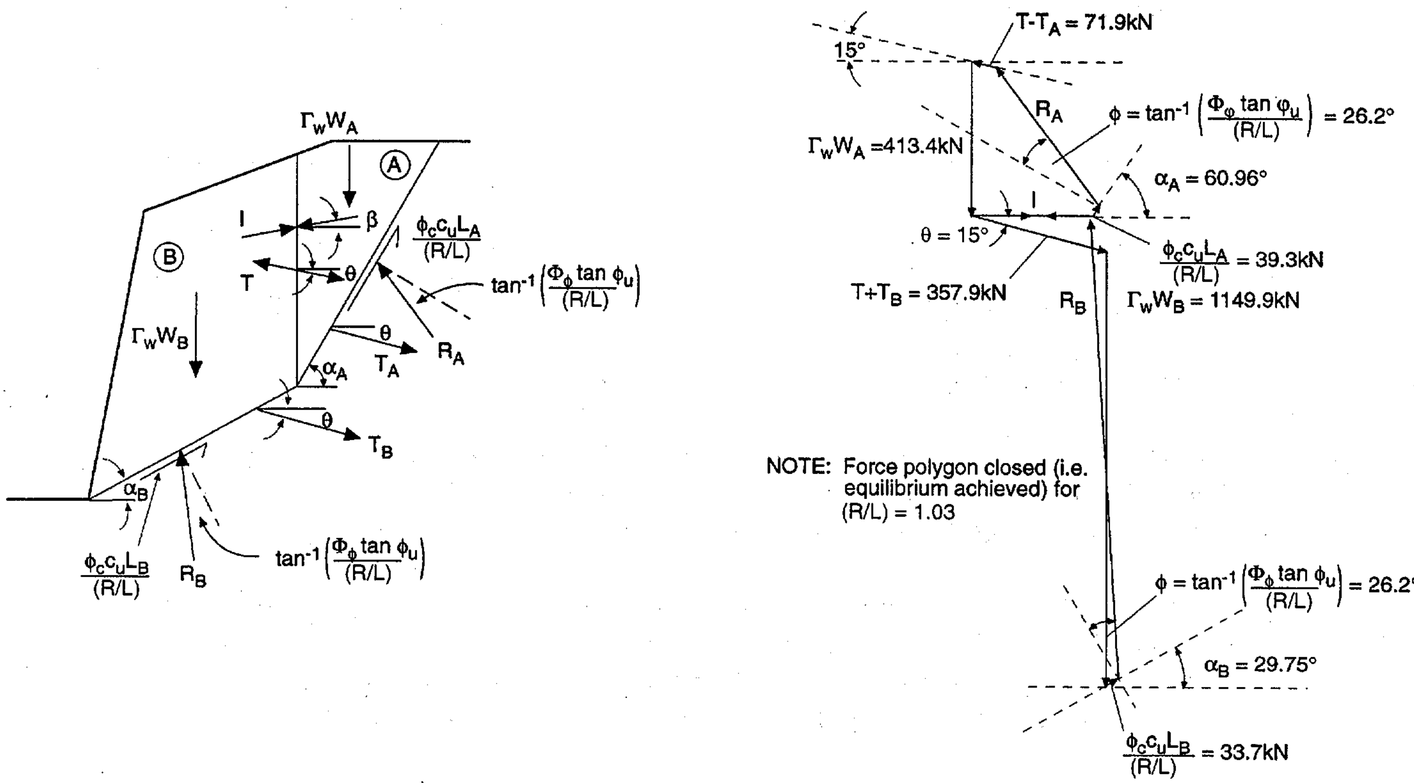


Figure 5.8. Cutslope Design Example Force Diagram and Polygon (LRFD) Hand Calculation Check

Block A

Vertical

$$\Gamma_W W_A+(T_A-T)\sin(\theta)-(I)\sin(\beta)-(\Phi_C)(c_U)(L_A)\sin(\alpha_A)/(R/L)-(R_A)\cos(\alpha_A-\phi)=0$$

Horizontal

$$(I)\cos(\beta)+(T_A-T)\cos(\theta)+(\Phi_C)(c_U)(L_A)\cos(\alpha_A)/(R/L)-(R_A)\sin(\alpha_A-\phi)=0$$

Block B

Vertical

$$\Gamma_W W_B+(T_B+T)\sin(\theta)+(I)\sin(\beta)-(\Phi_C)(c_U)(L_B)\sin(\alpha_B)/(R/L)-(R_B)\cos(\alpha_B-\phi)=0$$

Horizontal

$$(I)\cos(\beta)-(T_B+T)\cos(\theta)-(\Phi_C)(c_U)(L_B)\cos(\alpha_B)/(R/L)+(R_B)\sin(\alpha_B-\phi)=0$$

The previous equations can be reduced to the following:

$$[1+\tan(\beta)\tan(\alpha_A-\phi)]\,[\{-\Gamma_W W_B-(T_B+T)\sin(\theta)+\Phi_C\, c_U(L_B)\sin(\alpha_B)/(R/L)\}\tan(\alpha_B-\phi)+(T_B+T)\cos(\theta)+\Phi_C\, c_U(L_B)\cos(\alpha_B)/(R/L)]=[1+\tan(\beta)\tan(\alpha_B-\phi)]\,[\{\Gamma_W W_A+(T_A-T)\sin(\theta)-\Phi_C\, c_U(L_A)\sin(\alpha_A)/(R/L)\}\tan(\alpha_A-\phi)-(T_A-T)\cos(\theta)-\Phi_C\, c_U(L_A)\cos(\alpha_A)/(R/L)]$$

The above equation can be solved iteratively for resistance/load ratio (R/L) (e.g., by use of a spreadsheet), if the interslice force angle β is assumed. For example, assuming $\beta = 0$, the calculated resistance/load ratio (R/L) = 1.03, which is similar to the computer-generated solution. The force polygon for the hand calculation is shown on figure 5.8.

The results of stability assessments during construction are shown on figure 5.9. Calculated resistance/load ratios for each stage of construction following excavation of each lift and prior to installation of the associated row of nails, are shown together with the corresponding critical slip surface in each case. It can be seen that for this problem, following installation of the first row of nails, the resistance/load ratio during construction decreases progressively as the height of the wall increases. However, noting the increased soil strength resistance factors for the construction condition (table 4.8), the minimum calculated resistance/load ratio exceeds the minimum required value of 1.0. Figure 5.9 does indicate that prior to the installation of the first row of nails, the calculated resistance/load ratio is less than 1.0 but this construction condition has been evaluated in the field by test trenching during the site investigation stage.

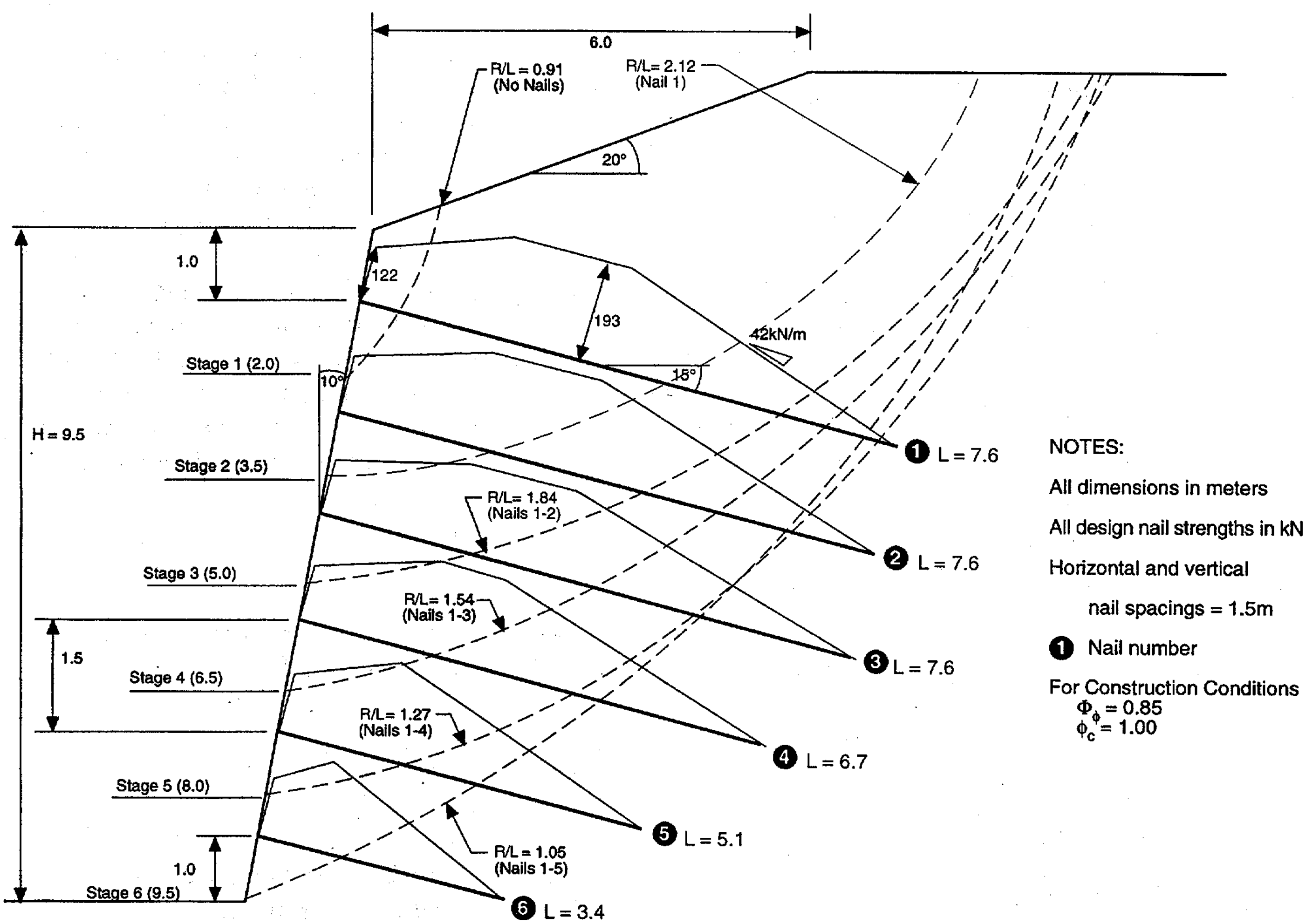


Figure 5.9 Cutslope Design Example Construction Stability (LRFD)

Step 8 - External Stability Check of Nailed Block

Defer the external stability check until the final soil nail lengths are defined for the seismic loading condition. An external stability check is not necessary for this wall as the subsurface soils are granular and no hydrostatic pressures are expected behind the wall.

Step 9 - Check the Upper Cantilever

The height of the upper cantilever above the top nail is identical (1.0 meters) for both the temporary shotcrete and permanent CIP wall facings. Therefore, the static loading is identical in the two cases. Because both the facing thickness and steel content are increased in the permanent facing, the permanent facing is less critical by inspection. Therefore, for the static loading condition, only the construction facing upper cantilever needs to be evaluated.

For the method of construction, the appropriate earth pressure coefficient for the upper cantilever design is an active earth pressure coefficient. For a soil friction angle of 34°, zero cohesion (ignore it), a soil-to-wall interface friction angle of (2/3)(34°) = 22°, a wall batter of 10°, and a backslope angle of 20°, Coulomb's earth pressure theory may be used to derive an active earth pressure coefficient, K_A, equal to 0.247 (AASHTO, 1st Edition, 1994 - section 3.11.5.3 [29]). The load component normal to the wall has a corresponding earth pressure coefficient equal to (0.247)cos(22°) = 0.229.

Shear Check

From force equilibrium, compute the factored one-way unit service shear force at the level of the upper row of nails (conservative), as indicated on figure 4.13:

$$v = 0.5(0.229)(18.0\ \text{kN/m}^3)(1.0\ \text{m})^2$$
$$v = 2.06\ \text{kN/m}$$
$$\Gamma_{EH} = 1.50 \quad \text{(table 4.6)}$$

Compute the nominal one-way unit shear strength of the facing based on equation 5.8.3.3-3 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(\text{d})$$
$$V_{NS} = 0.166\sqrt{28\ \text{MPa}}\ (0.05\ \text{m}) = 43.9\ \text{kN/m}$$

From table 4.7, the facing shear resistance factor, Φ_F, equals 0.90. Therefore, the design one-way unit shear strength is:

$$V = \Phi_F V_{NS} = 0.90(43.9\ \text{kN/m}) = 39.5\ \text{kN/m}$$

Since Γ_{EH} v = 3.09 kN/m < V, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the factored one-way unit service moment at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever:

$m_S = (0.33)(H/\cos(10^o))(v)$

$m_S = (0.33)(1.0\ m/\cos(10^o))(2.06\ kN/m) = 0.690\ kN\text{-}m/m$

$\Gamma_{EH} = 1.50$ (table 4.6)

Compute the nominal unit moment resistance of the facing. From appendix F, the nominal unit moment resistance in the vertical direction over the nail locations $m_{V,NEG}$ is computed to be 5.88 kN-m/m. From table 4.7, the resistance factor Φ_F for facing flexure is 0.90. Therefore, the design one-way unit moment resistance for the upper cantilever is:

$M = \Phi_F\, m_{V,NEG} = (0.90)(5.88\ kN\text{-}m/m) = 5.29\ kN\text{-}m/m$

Since $\Gamma_{EH}\, m_S = 1.04 < M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

Shotcrete Construction Facing

Check 1 - Waler Reinforcement

Two No. 13 horizontal waler bars attached beneath the bearing plate will be placed continuously between nail heads in each nail row.

Check 2 - Minimum/Maximum Reinforcement Ratios

Minimum Reinforcement Ratio Requirements

The minimum reinforcement ratio requirements in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] are waived for the temporary shotcrete construction facing per the discussion presented in section 4.7.2.

Maximum Reinforcement Ratio Requirements

Section 5.7.3.3 of AASHTO [29] contains a provision for the maximum amount of reinforcement allowed in a flexural member. Per the discussion presented in section 4.7.2, this provision is not applicable in the case of a soil nail wall.

Check 3 - Minimum Cover Requirements

For the temporary shotcrete construction facing, the reinforcement is placed at the center of the facing (figure 5.1B)

Check 4 - Development and Splices of Reinforcement

Development of No. 13 Vertical Bearing Bars

For a uniformly loaded interior span with fixed-end supports, the point of zero moment occurs at $0.213L_C$ from the support. The vertical bearing bars are not needed all the way to the point of zero moment. However, per section 5.11.1.2.1 of the AASHTO LRFD Bridge Specifications, 1st Edition [29], they must extend the maximum of $L_C/20$, $15d_B$, or d, past the point at which they are no longer needed:

$$L_C/20 = (1500 \text{ mm}/20 = 75 \text{ mm}$$
$$15d_B = 15(12.7 \text{ mm}) = 191 \text{ mm}$$
$$d = (100 \text{ mm})/2 = 50 \text{ mm}$$

Since the vertical bearing bars comprise about $(258 \text{ mm}^2)/(443 \text{ mm}^2) = 58$ percent of the total vertical reinforcement, estimate the point at which the vertical bearing bars are no longer needed as $0.58(0.213L_C)$ from the support. The total bar length is computed to be:

$$L_{VB} = 2[0.58(0.213)(1.50 \text{ m}) + 0.191 \text{ m})] = 0.753 \text{ m} \quad \text{(Minimum)}$$

Section 5.11.1.2.3 of AASHTO [29] provides additional requirements for embedments beyond points of inflection. The only requirement that is potentially more critical than the above is an embedment at least equal to:

$$L_C/16 = (1500 \text{ mm})/16 = 93.8 \text{ mm}$$

This is less critical than the $15d_B = 191$ mm, computed above.

Splicing of No. 13 Waler Bars

Per section 5.11.5.3 of AASHTO [29], the splice length, L_S, for a Class C splice must equal or exceed the greater of 0.30 m or $L_D = 1.7L_{DB}$, where L_{DB} is computed from section 5.11.2:

$$L_{DB} = 0.019(A_B \text{ (mm}^2))(F_Y \text{ (MPa)})/\sqrt{f'_C \text{ (MPa)}}$$
$$L_{DB} = 0.019(129 \text{ mm}^2)(420 \text{ MPa})/\sqrt{28.0 \text{ MPa}} = 195 \text{ mm}$$
$$L_D = 1.7L_{DB} = 1.7(195 \text{ mm}) = 332 \text{ mm}$$

Therefore, provide the 350 mm of splice length.

Splicing of MW 19 Mesh

Per section 5.11.6.2 of AASHTO [29], the splice length between outermost crosswires must equal or exceed the greater of (S_{WIRE} + 50 mm), $1.5L_{DB}$, or 150 mm. L_{DB} is computed from AASHTO equation 5.11.2.5.2-1:

$$L_{DB} = 3.25(A_{WIRE}\ (mm^2))(F_Y\ (MPa))/(((S_{WIRE}\ (mm))\sqrt{f'_C\ (MPa)}\ ))$$
$$L_{DB} = 3.25(18.7\ mm^2)(420\ MPa)/((152\ mm)\sqrt{28.0\ MPa}\ )$$
$$L_{DB} = 31.7\ mm$$
$$L_D = 1.5L_{DB} = 1.5(31.7\ mm) = 47.6\ mm$$
$$S_{WIRE} + 50.0\ mm = 152\ mm + 50.0\ mm = 202\ mm$$

Therefore, use a minimum of 200 mm splices for the wire mesh.

Permanent Facing

Check 1 - Waler Reinforcement

There are no applicable requirements.

Check 2 - Minimum and Maximum Reinforcement Ratios

Minimum Reinforcement Ratio Requirements

Per section 5.10.8 of AASHTO [29], the minimum required amount of shrinkage and temperature reinforcement near exposed surfaces of walls and slabs is given by equation 5.10.8.2-1, and must be greater than:

$$0.76A_G/F_Y$$

For one nail spacing:

$$0.76A_G/F_Y = 0.76(1500\ mm)(200\ mm)/(420\ MPa) = 543\ mm^2$$

A_{TOTAL} = 645 mm^2, which is adequate.

Maximum Reinforcement Ratio Requirements

Section 5.7.3.3 of AASHTO [29] contains a provision for the maximum amount of reinforcement allowed in a flexural member. Per the discussion presented in section 4.7.2, this provision is not applicable in the case of a soil nail wall.

Check 3 - Minimum Cover Requirements

Per section 5.12.3 of AASHTO [29], the minimum cover on the front side of the facing is specified at 50 mm. Based on the design arrangement illustrated on figure 5.1B, the cover to the headed studs is calculated below:

$$t_C = 200\ mm - t_{PL} - L_{HS} = 200\ mm - 25\ mm - 125\ mm = 50\ mm$$

50 mm ≥ 50 mm (OK)

The minimum required cover between the permanent facing reinforcing steel and the CIP concrete/temporary shotcrete interface is 38 mm. Based on figure 5.1B, this cover is:

$$t_c = 100 \text{ mm} - 12.7 \text{ mm} = 87.3 \text{ mm} > 38 \text{ mm}$$

For corrosion protection purposes, there must also be a minimum 75 mm of cover between the facing steel and the soil. Based on figure 5.1B, the 100 mm thick temporary shotcrete provides adequate cover for the permanent facing steel.

Check 4 - Development and Splices of Reinforcement

The permanent facing reinforcement consists entirely of No. 13 deformed bars. Therefore, the development lengths and splice lengths are identical to those computed for the temporary facing.

Step 11 - Serviceability Checks

Shotcrete Construction Facing

Because of the temporary nature of the wall, the serviceability requirements are waived for the construction facing.

Permanent Facing

Upper Cantilever Serviceability Check - Reinforcement Distribution

Per section 5.7.3.4 of AASHTO [29], the reinforcement must be distributed such that the steel stress does not exceed that given by the following equation:

$$F_S = \frac{z}{(d_C A_E)^{1/3}} \leq 0.6F_Y$$

$$z = 130 \text{ k/in} = 2.28 \times 10^4 \text{ kN/m}$$

$$d_C = 0.05 \text{ m}$$

$$A_E = (100 \text{ mm})\,(S_H)/(A_{TOTAL}/A_{LB})$$

$$A_E = (100 \text{ mm})(1500 \text{ mm})/((645 \text{ mm}^2)/(129 \text{ mm}^2))$$

$$A_E = 0.03 \text{ m}^2$$

$$0.6F_Y = 0.6(420 \text{ MPa}) = 252 \text{ MPa}$$

$$F_S = \frac{2.28 \times 10^4 \text{ kN/m}}{((0.05 \text{ m})(0.03 \text{ m}^2))^{1/3}} = 199 \text{ MPa}$$

From step 9, the service moment, m_S, is 0.690 kN-m/m. The corresponding service steel stress is determined from straight-line theory of reinforced concrete:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$

$\rho = A_S/(bd)$
$n = E_S/E_C$
$j = 1 - k/3$
$f_S = m_S\, b\, /(A_S jd)$
$E_C = 4734\sqrt{f'_c\,(\text{MPa})} = 4734\sqrt{28\ \text{MPa}}$
$E_C = 2.50 \times 10^4$ MPa
$E_S = (29{,}000{,}000\ \text{psi})(1\ \text{MPa})/(145\ \text{psi}) = 2.00 \times 10^5$ MPa

Substituting the correct values into the above expressions, the service steel stress is computed to be:

$n = (2.00 \times 10^5\ \text{MPa})/(2.50 \times 10^4\ \text{MPa}) = 8.00$
$\rho = A_S/(bd) = (645\ \text{mm}^2)/((1500\ \text{mm})(100\ \text{mm})) = 0.0043$
$\rho n = (0.0043)(8.00) = 0.0344$
$k = \sqrt{2(0.0344) + (0.0344)^2} - 0.0344 = 0.230$
$j = 1 - 0.230/3 = 0.923$
$f_S = (0.690\ \text{kN-m/m})(1.50\ \text{m})/((645\ \text{mm}^2)(0.923)(0.10\ \text{m}))$
$f_S = 17.4$ MPa

Since $f_S \leq F_S$, the steel distribution is adequate. (OK)

Overall Displacements of the Wall

Per section 2.4.6, the construction-induced vertical and horizontal permanent displacements at the top of the wall can be expected to be on the order of 0.2 percent of the height of the wall, or about 20 to 25 mm, for the given site soil conditions (medium dense to dense silty sands). Displacements can be anticipated to decrease back from the wall in general accordance with the recommendations given in figure 2.8.

5.1.2.2 Seismic Loading Condition

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Extreme Events I Limit State (table 4.6) considers seismic loading. Because of the temporary nature of the shotcrete construction facing, only the permanent facing is considered for the seismic loading condition.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

Step 1 is identical to that presented for the static loading condition.

Step 2 - Compute the Design Nail Head Strengths

The nominal nail head strengths for all credible failure modes are as calculated for the static conditions. The design nail head strengths for the permanent CIP concrete facing are computed from the nominal strengths for Group VII loading in the following table.

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Strength T_F (kN)
Facing Flexure	278	$(1.00)^a(278) = 278$
Facing Punching Shear	219	$(1.00)^a(219) = 219$
Headed-Stud Tensile Fracture	639	$(1.00)^a(639) = 639$

[a] Per section 1.3.2.1 of AASHTO [29].
See table 4.7 .

Therefore, the design nail head strength T_F is computed to be 219 kN. That is, facing punching shear of the permanent facing is the controlling mode of failure.

Step 3 - Minimum Design Nail Head Strength Check

This check is not applied to extreme event loading combinations.

Step 4 - Define the Design Nail Strength Diagrams

Develop the design nail strength diagram for each nail, by determining the design pullout resistance, the design nail head strength, and the design nail tendon tensile strength.

Design Pullout Resistance, Q, (Ground-Grout Bond)

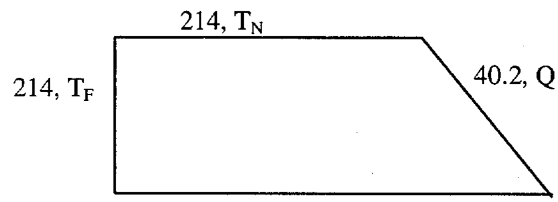
$Q = \Phi_Q Q_U$
$\Phi_Q = 0.80$ (table 4.8)
$Q_U = 60.0$ kN/m
$Q = (0.80)(60.0 \text{ kN/m}) = 48.0$ kN/m

Design Nail Tendon Tensile Strength, T_N

$T_N = \Phi_N T_{NN}$
$\Phi_N = 1.00$ (table 4.8)
$T_{NN} = A_B F_Y = (510 \text{ mm}^2)(0.42 \text{ kN/mm}^2) = 214$ kN
$T_N = (1.00)(214 \text{ kN}) = 214$ kN

Design Nail Head Strength, T_F

Per step 2, the design nail head strength is 219 kN, which exceeds the design nail tendon tensile strength as shown below. The design nail tendon tensile strength therefore governs.



Step 5 - Select Trial Nail Lengths

In accordance with section 4.7.2 and figure 4.11, the trial nail length distribution for design purposes is calculated by the same procedure used for the static case.

- The dimensionless nail pullout resistance, Q_D, is calculated:

$$Q_D = \phi_Q Q_U / (\Gamma_W \, \gamma S_V S_H) = (0.8)(60.0 \text{ kN/m})/[(1.0^a)(18 \text{ kN/m}^3)(1.50 \text{ m})(1.50 \text{ m})]$$
$$= 1.19$$

[a] Note that the minimum load factor of 1.0 (table 4.6) is applied, as this is more critical in this case for the seismic loading condition than is the maximum load factor of 1.35.

- The dimensionless nail length is:

$$L/H = (8.4 \text{ m})/(9.5 \text{ m})$$
$$= 0.88$$

- Q_D / (L/H) = 1.19/0.88 = 1.35, giving an "R" factor of 0.20.

- Relative nail lengths are calculated from figure 4.11, for the nail head elevations shown on figure 5.10 and an "R" value of 0.20 as shown below.

Nail No.	Rx	Trial Nail Length (m)
1	1.0	8.4
2	1.0	8.4
3	1.0	8.4
4	0.87	7.3
5	0.62	5.2
6	0.37	3.1

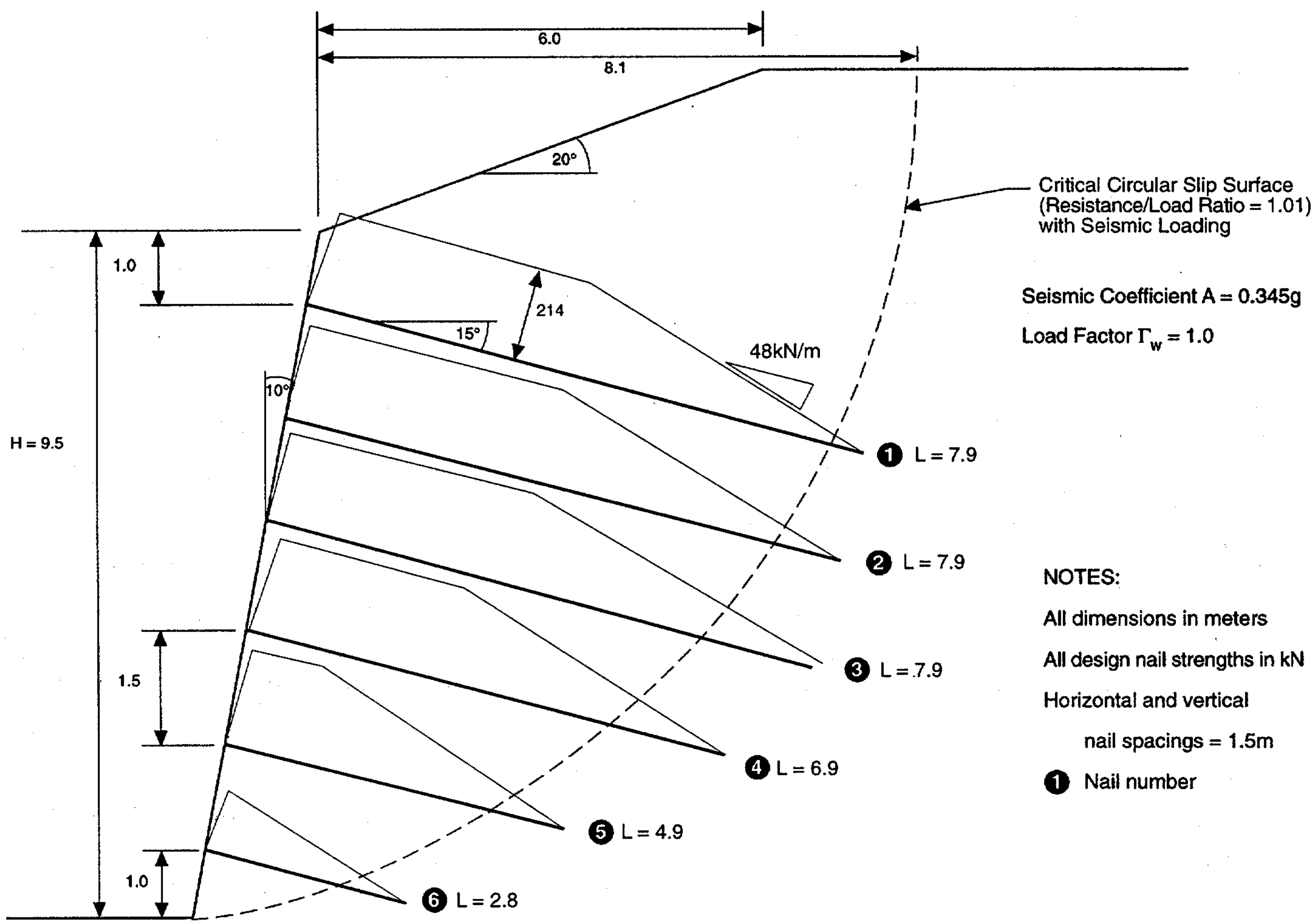


Figure 5.10 Cutslope Design Example Critical Cross-Section Seismic Loading (LRFD)

Step 6 - Define the Design Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0°
Ultimate Cohesion, c_U = 5.0 kN/m^2

From table 4.8, the soil resistance factors are as follows:

Φ_ϕ = 1.00
Φ_C = 1.00

Design Friction Angle, $\phi = \tan^{-1}(\Phi_\phi \tan(\phi_U)) = \tan^{-1}(1.00[\tan(34.0^\circ)]) = 34^\circ$
Design Cohesion, $c = \Phi_C c_U = 1.00(5.0\ \text{kN/m}^2) = 5\ \text{kN/m}^2$

Step 7 - Calculate the Resistance/Load Ratio

An iterative limiting equilibrium analysis is performed using appropriate computer software to determine the actual nail lengths which are required for a calculated minimum resistance/load ratio of 1.01.

- For slip surfaces exiting the reinforced slope within a horizontal distance of 8.1 meters from the top of wall facing (determined from figure 5.7 in accordance with figure 4.14), the applied pseudo-static seismic coefficient for a 0.30 g peak ground acceleration was taken as:

  $$A = (1.45 - A_{PK})A_{PK}$$
  $$A = (1.45 - 0.30)(0.30\ g) = 0.345\ g$$

- For more deep-seated slip surfaces, an applied pseudo-static seismic coefficient of A = (1/2)(0.30 g) = 0.15 g was chosen, consistent with allowable overall displacements of up to 75 mm for the retaining wall system under design seismic loading.

Limiting-equilibrium analysis for seismic loading corresponding to a peak ground acceleration of 0.30 g has indicated that the same size nails as originally selected for the static design can be used, but that the maximum nail lengths should be increased from 7.6 meters (for static loading) to 7.9 meters (for seismic loading) i.e., about a 4 percent increase in length. For this design example, seismic loading slightly controls the design.

Step 8 - External Stability Check of Nailed Block (Static and Seismic)

THE COMPUTATIONS FOR EXTERNAL STABILITY WHICH FOLLOW ARE SHOWN ONLY FR ILLUSTRATIVE PURPOSES. As noted in the static computations, the check of static external stability is not required for granular soils. Also the seismic external stability is generally not checked unless the peak ground acceleration exceeds 0.25g as discussed in section 4.10.11.

For the design nail length of 7.9 m, determined in step 7, the static and seismic bearing capacity checks are performed for the soil nail reinforced block of ground acting as a gravity wall structure (figure 4.12). Pertinent parameters for the analysis include the following:

Soil Unit Weight, γ = 18.0 kN/m^3
Ultimate Soil Friction Angle, ϕ_U = 34°
Ultimate Soil Cohesion, c_U = 5.0 kN/m^2
Seismic Coefficient, A = 0.15g

In accordance with appendix A, section 11 of the AASHTO [29], the lateral forces due to the active block behind the reinforced soil are computed using the methods of Coulomb and Mononobe and Okabe (including the cohesive component of soil strength). Figure 5.11 shows the static and seismic earth pressure loads, inclined at an angle of 6.6° above horizontal in accordance with the recommendations of figure 4.12, and these are computed to be:

Static Active Earth Load, P_{EH} = 272.2 kN/m
Horizontal component = $P_{EH}\cos(6.6^o)$ = 270.4 kN/m
Vertical component = $P_{EH}\sin(6.6^o)$ = 31.3 kN/m

Seismic Earth Load, P_{EQ} = 112.1 kN/m
Horizontal component = $P_{EQ}\cos(6.6^o)$ = 111.4 kN/m
Vertical component = $P_{EQ}\sin(6.6^o)$ = 12.9 kN/m

For moment equilibrium calculations, the assumed points of application of P_{EH} and P_{EQ} are at the back of the reinforced block of ground at 0.33H and 0.6H above the base of the soil block, respectively:

Point of Application of P_{EH} = 0.33H = 0.33(11.7 m) = 3.90 m

Point of Application of P_{EQ} = 0.6H = 0.6(11.7 m) = 7.02 m

<u>Static Loading</u>

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Strength I Limit State (table 4.6) considers static loading. Referring to figure 5.11, forces and moments are computed below:

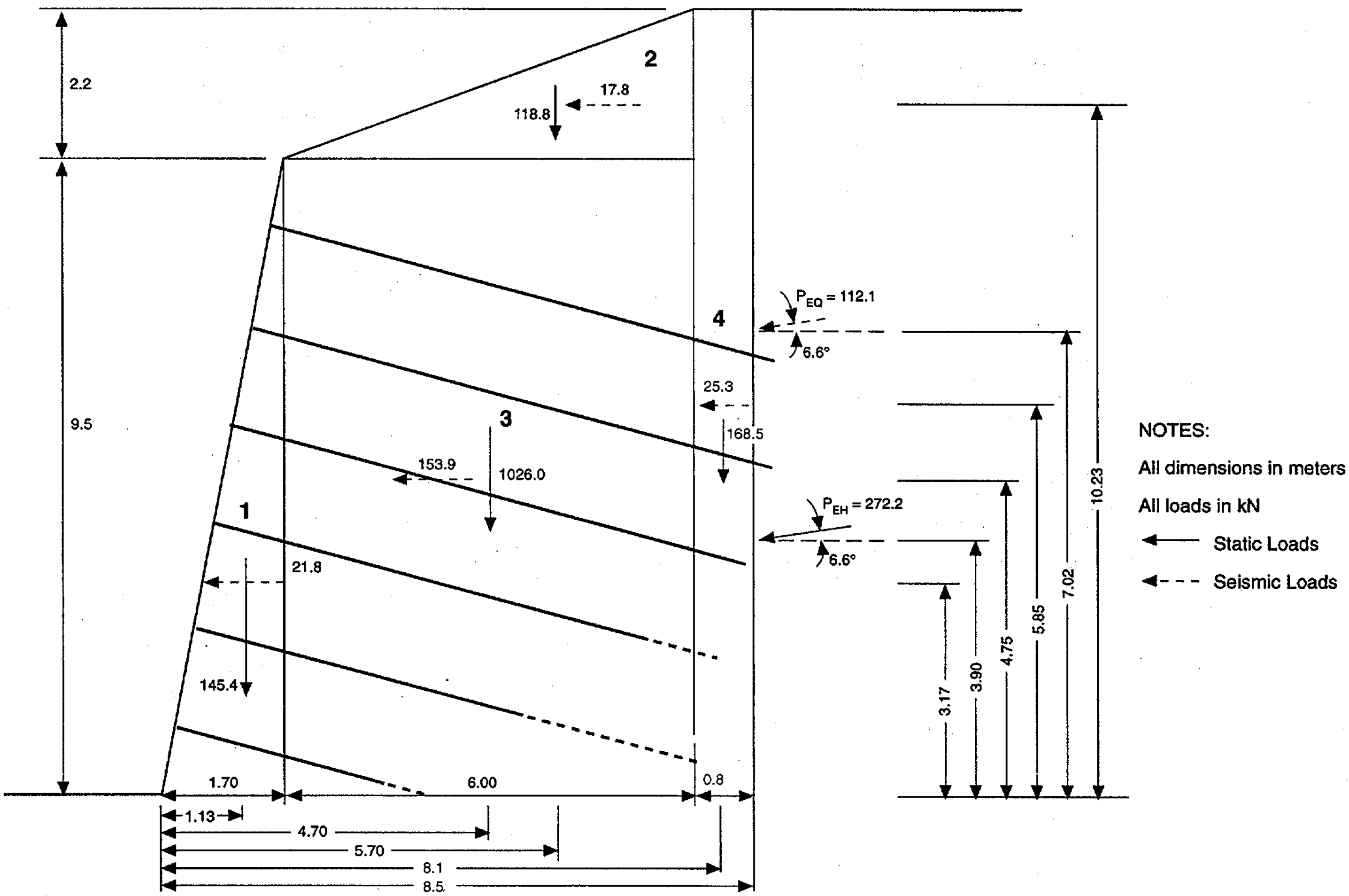


Figure 5.11. Cutslope Design Example External Stability Checks (LRFD)

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.5^b(31.3)$	8.50	399.1
1 $(EV)^a$	$1.35^c(145.4)$	1.13	221.8
2 $(EV)^a$	$1.35^c(118.8)$	5.70	914.2
3 $(EV)^a$	$1.35^c(1026.0)$	4.70	6510.0
4 $(EV)^a$	$1.35^c(168.5)$	8.10	1842.6
Sum	2016.2 (N)	N/A	9887.6

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.50^b(270.4)$ (H)	3.90	-1581.8

[a] Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
[b] Load factor for lateral earth pressure (EH)
[c] Load factor for vertical earth pressure (EV)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

N = 2016.2 kN
X_O = (9887.6) kN-m/m - 1581.8 kN-m/m)/(2016.2 kN/m) = 4.12 m
q_{MAX} = $N/(2X_o)$ = (2016.2 kN/m)/[2(4.12 m)] = 245 kN/m^2

In accordance with section 10 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] (and by reference to equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30]), the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) = 2(4.12 m) = 8.24 m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$$q_{ULT} = (5.0\ kN/m^2)(42.0)(1.06)(0.68) + 0.5(18.0\ kN/m^3)(8.24\ m)(41.0)(0.97)(0.57)$$
$$q_{ULT} = 1{,}832\ kN/m^2$$

It should be noted that unfactored loads are used in computing the load inclination factor "i." For a bearing capacity resistance factor Φ_q of 0.45 (section 10.5.4 of AASHTO [29], the design bearing capacity is:

$$\Phi_q q_{ULT} = 0.45(1{,}832\ kN/m^2) = 824\ kN/m^2$$

Since $q_{MAX} < \Phi_q q_{ULT}$, the design bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle half of the block of reinforced soil, per section 10.6.3.1.5 of AASHTO [29]. The critical eccentricity check occurs for the minimum EV load factor of 1.0 (see table 4.6). Referring to figure 5.11, forces and moments are computed below:

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.5^b(31.3)$	8.50	399.1
1 $(EV)^a$	$1.00^c(145.4)$	1.13	164.3
2 $(EV)^a$	$1.00^c(118.8)$	5.70	677.2
3 $(EV)^a$	$1.00^c(1026.0)$	4.70	4822.2
4 $(EV)^a$	$1.00^c(168.5)$	8.10	1364.9
Sum	1505.7 (N)	N/A	7427.6

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.50^b(270.4)$ (H)	3.90	-1581.8

^{a} Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
^{b} Load factor for lateral earth pressure (EH)
^{c} Load factor for vertical earth pressure (EV)

$$X_O = (7427.6\ kN\text{-}m/m - 1581.8\ kN\text{-}m/m)/(1505.7\ kN/m) = 3.88\ m$$

$e = B/2 - X_o = (8.50 \text{ m})/2 - 3.88 \text{ m} = 0.37 \text{ m}$
$B/4 = (8.50 \text{ m})/4 = 2.13 \text{ m}$

Since e < B/4, the resultant is within the middle half. (OK)

Seismic Loading

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Extreme Events I Limit State (table 4.6) considers seismic loading. The forces and moments are computed below (figure 5.11). The critical bearing and eccentricity check for the seismic loading condition occurs for the minimum EV load factor of 1.0 (see table 4.6).

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.5^b(31.3)$	8.50	399.1
$P_{EQ}(EQ)^a$	$1.0^d(12.9)$	8.50	109.7
$1\ (EV)^a$	$1.0^c(145.4)$	1.13	164.3
$2\ (EV)^a$	$1.0^c(118.8)$	5.70	677.2
$3\ (EV)^a$	$1.0^c(1026.0)$	4.70	4822.2
$4\ (EV)^a$	$1.0^c(168.5)$	8.10	1364.9
Sum	1518.6 (N)	N/A	7537.2

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
$P_{EH}(EH)^a$	$1.50^b(270.4)$	3.90	-1581.8
$P_{EQ}(EQ)^a$	$1.\ 0^d(111.4)$	7.02	-782.0
$1\ (EQ)^a$	$1.\ 0^d(21.8)$	3.17	-69.1
$2\ (EQ)^a$	$1.\ 0^d(17.8)$	10.23	-182.1
$3\ (EQ)^a$	$1.\ 0^d(153.9)$	4.75	-731.0
$4\ (EQ)^a$	$1.\ 0^d(25.3)$	5.85	-148.0
Sum	735.8 (H)	N/A	-3494.1.2

[a] Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
[b] Load factor for lateral earth pressure (EH)
[c] Load factor for vertical earth pressure (EV)
[d] Load factor for earthquake loads.

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

N = 1518.6 kN
X_O = (7537.2 kN-m/m- 3494.1 kN-m/m)/(1518.6 kN/m) = 2.66 m
q_{MAX} = $N/(2X_o)$ = (1518.6 kN/m)/[2(2.66 m)] = 285 kN/m

In accordance with section 10 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] (and by reference to equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30]), the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) = 2(2.66 m) = 5.32 m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

q_{ULT} = (5.0 kN/m^2)(42.0)(1.04)(0.36)+ 0.5(18.0 kN/m^3)(5.32 m)(41.0)(0.98)(0.23)
q_{ULT} = 521 kN/m^2

It should be noted that unfactored loads are used in computing the load inclination factor "i". For a bearing capacity resistance factor Φ_q of 1.00 for seismic loading (section 1.3.2.1 of AASHTO [29]), the design bearing capacity is:

$$\Phi_q q_{ULT} = 1.00(521 \text{ kN/m}^2) = 521 \text{ kN/m}^2$$

Since $q_{MAX} < \Phi_q q_{ULT}$, the design bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle half of the block of reinforced soil per section 10.6.3.1.5 of AASHTO [29].

e = $B/2 - X_o$ = (8.50 m)/2 - 2.66 m = 1.59 m
$B/4$ = (8.50 m)/4 = 2.13 m

Since $e < B/4$, the resultant is within the middle half. (OK)

Overall Stability

For the uniform nail length pattern selected (see figure 5.12), the deep ground water table, and the overall geometry of the slope, deep seated failure surfaces passing beneath the toe of the wall will not be critical.

Step 9 - Check the Upper Cantilever

In accordance with appendix A, section 11 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], equation A11.1.1.1-2, the approach developed by Mononobe and Okabe for a free-standing retaining structure is used to develop the active seismic loading on the upper cantilever. For a soil friction angle of 34°, zero cohesion (ignore it), a soil-to-wall interface friction angle of (2/3)34° = 22°, a wall batter of 10°, a backslope angle of 20°, and a horizontal pseudo-static seismic coefficient of 0.15 g (i.e., $0.5A_{PK}$), the combined static and dynamic active earth pressure coefficient is calculated by the Mononobe and Okabe method to be 0.429. This can be considered to consist of a static earth pressure coefficient of 0.247 (see step 7 for the static loading condition) and a dynamic earth pressure coefficient of 0.182. The load components normal to the wall have corresponding earth pressure coefficients of (0.247)cos(22°) = 0.229 for the static load and (0.182)cos(22°) = 0.169 for the dynamic load.

Shear Check

From force equilibrium, compute the factored one-way unit service shear force at the level of the upper row of nails (conservative), as indicated on figure 4.13:

$$v_{STATIC} = 0.5(0.229)(18.0\ \text{kN/m}^3)(1.0\ \text{m})^2 = 2.06\ \text{kN/m}$$
$$v_{DYNAMIC} = 0.5(0.169)(18.0\ \text{kN/m}^3)(1.0\ \text{m})^2 = 1.52\ \text{kN/m}$$
$$\Gamma_{EH} = 1.50; \quad \Gamma_{EQ} = 1.00 \quad \text{(table 4.6)}$$
$$\Gamma_{EH} v_{STATIC} + \Gamma_{EQ} v_{DYNAMIC} = 1.50(2.06\ \text{kN/m}) + 1.00(1.52\ \text{kN/m}) = 4.61\ \text{kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 5.8.3.3-3 of AASHTO [29]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28\ \text{MPa}}\ (0.1\ \text{m}) = 87.8\ \text{kN/m}$$

From table 4.7 and section 1.3.2.1 of AASHTO [29], the facing shear resistance factor, Φ_F, equals 1.0. Therefore, the design one-way unit shear strength of the facing is computed to be:

$$V = \Phi_F V_{NS} = 1.00(87.8\ \text{kN/m}) = 87.8\ \text{kN/m}$$

Since $\Gamma_{EH} v_{STATIC} + \Gamma_{EQ} v_{DYNAMIC} = 4.61\ \text{kN/m} < V$, the design for shear is adequate. (OK)

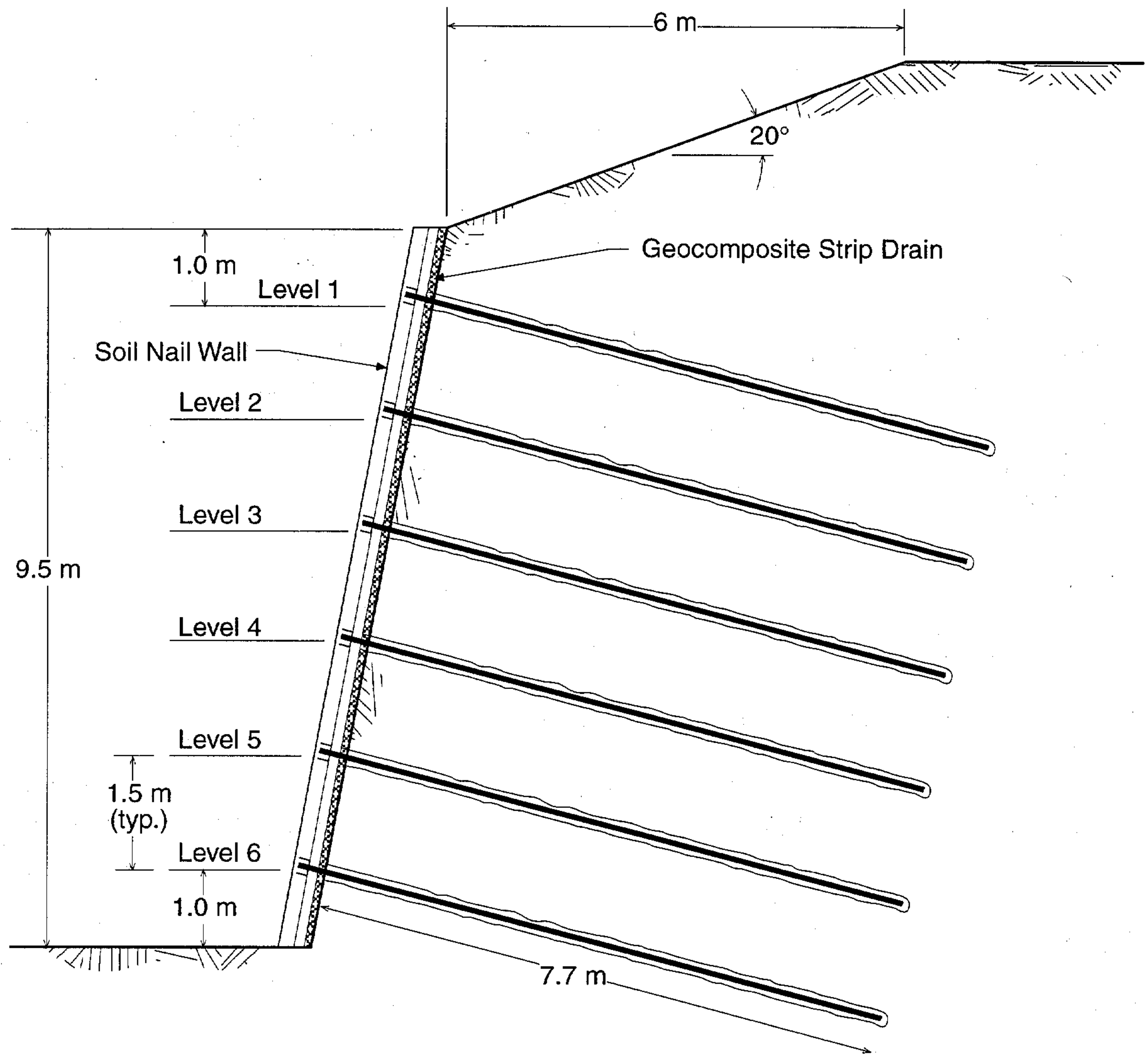


Figure 5.12 Cutslope Final Design Section (LRFD)

Flexure Check

From moment equilibrium, compute the factored one-way unit service moment at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever. The dynamic force is assumed to occur at 0.6H above the base of the cantilever:

$$m_{STATIC} = (0.33)(v_{STATIC})(H)/\cos(10^\circ) = (0.33)(2.06\text{ kN/m})(1.0\text{ m})/\cos(10^\circ) = 0.690\text{ kN-m/m}$$

$$m_{DYNAMIC} = (0.60)(v_{DYNAMIC})(H)/\cos(10^\circ) = (0.60)(1.52\text{ kN/m})(1.0\text{ m})/\cos(10^\circ) = 0.926\text{ kN-m/m}$$

$\Gamma_{EH} = 1.50$; $\Gamma_{EQ} = 1.00$ (table 4.6)

$$\Gamma_{EH}m_{STATIC} + \Gamma_{EQ}m_{DYNAMIC} = 1.50(0.690\text{ kN-m/m}) + 1.00(0.926\text{ kN-m/m}) = 1.96\text{ kN-m/m}$$

Compute the nominal unit moment resistance of the facing. From appendix F, the nominal unit moment resistance in the vertical direction over the nail locations $m_{V,NEG}$ is computed to be 17.4 kN-m/m. Per section 1.3.2.1 of AASHTO [29], the resistance factor Φ_F for facing flexure is 1.00. Therefore, the design one-way unit moment resistance for the upper cantilever is:

$$M = \Phi_F\, m_{V,NEG} = (1.00)(17.4\text{ kN-m/m}) = 17.4\text{ kN-m/m}$$

Since $\Gamma_{EH}m_{STATIC} + \Gamma_{EQ}m_{DYNAMIC} = 1.96$ kN-m/m < M, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

Being independent of type of loading, the facing reinforcement details, have been previously considered for the static loading condition and need not be repeated for the seismic loading condition.

Step 11 - Serviceability Checks

The serviceability checks are not applicable to seismic loading conditions because of the extreme nature of the limit state.

Final Design Section

Based on the above analyses, the final design cross-section is shown on figure 5.12. It should be noted that, to simplify for construction, the chosen final plan nail lengths are uniform, independent of the nail length distributions used in the analyses. The uniform lengths equal the maximum nail length indicated by the above analyses. Based on the designer's discretion, and subject to overall stability analyses, the nails in the bottom three rows could be shortened, taking into account any constructability issues and/or economic savings related to the use of varying nail lengths.

CHAPTER 6. SOIL NAIL WALL PERFORMANCE MONITORING

6.1 Introduction

Although several hundred soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data in support of design procedures. Confidence in the use of soil nail shoring structures and improvements in design will be enhanced by proven performance of such systems. To this end, it is important to monitor performance behavior of future soil nail structures. This chapter includes details necessary to plan and implement both limited and comprehensive monitoring programs for soil nail systems. Recommendations for appropriate instrumentation are included.

Safe and economic soil nail walls can currently be designed, yet the behavior of such walls is still being studied. It is considered that some current design techniques may incorporate considerable conservatism. Additional performance data are needed in order to refine the design and construction methodologies.

The United States Department of Transportation, Federal Highway Administration (FHWA) has published a document titled *Reinforced Soil Structures, Volume I., Design and Construction Guidelines*, FHWA-RD-89-043 [44]. Chapter 8 of this guideline, "Monitoring of Reinforced Soil Structures", provides a comprehensive discussion and details of appropriate instrumentation schemes, equipment requirements, etc. for providing soil nail wall performance monitoring. This chapter is recommended reading for all soil nail design engineers, inspectors, and specialty contractors.

On U.S. Federal-aid highway projects, it is recommended that the initial permanent soil nail wall constructed in each State and any initial critical or unusual installations (e.g., walls greater than 10 m high, widening under existing bridges, walls with high external surcharge loading, etc.) be designated an "Experimental Features Project" and have performance monitoring instrumentation installed. A major advantage of the Experimental Features designation is that it allows construction funds to be used to pay for the performance monitoring instrumentation and evaluation including data interpretation, plotting and report preparation. Performance monitoring instrumentation for such walls should preferably include slope inclinometers and top of wall survey points to measure wall movements during and after construction and load cells and strain gages installed on selected production nails to measure nail loads at the wall face and along the nail length. By strain gauging individual nails, the development and distribution of the nail forces may be measured to provide vital feedback to designers. Load cells at the nail head also provide data on facing loads. Monitoring for a period of at least 2 years after construction is recommended because the instrumentation monitors service behavior, i.e., structural deformation and stress development in the nails and wall facing as a function of both load and time and environmental changes such as winter freeze-thaw cycles.

6.2 Soil Nail Wall Performance Monitoring Methods

Monitoring during wall construction can be of limited nature with the intention of obtaining data on the overall wall performance. As a minimum, observations and monitoring should typically include:

- Face horizontal movements using surface markers on the face and surveying methods and inclinometer casings installed a short distance (typically 1 m) behind the facing.
- Vertical and horizontal movements of the top of wall facing and the ground surface behind the shotcrete facing, using optical surveying methods.
- Ground cracks and other signs of disturbance in the ground surface behind the top of wall, by daily visual inspection during construction and, if necessary, crack gages.
- Local movements and or deterioration of the facing using visual inspections and instruments such as crack gages.
- Drainage behavior of the structure, especially if groundwater was observed during construction. Drainage can be monitored visually by observing outflow points or through standpipe piezometers installed behind the facing.

Alternatively, soil nail wall performance monitoring can be more comprehensive and continued over a longer time period for one or more of the following purposes:

- Confirming design stress levels and monitoring safety during construction.
- Allowing construction procedures to be modified for safety or economy.
- Controlling construction rates.
- Enhancing knowledge of the behavior of soil nail structures to provide a base reference for future designs with the possibility of improving design procedures and/or reducing costs.
- Providing insight into seismic performance based on long-term performance monitoring through future earthquake events.

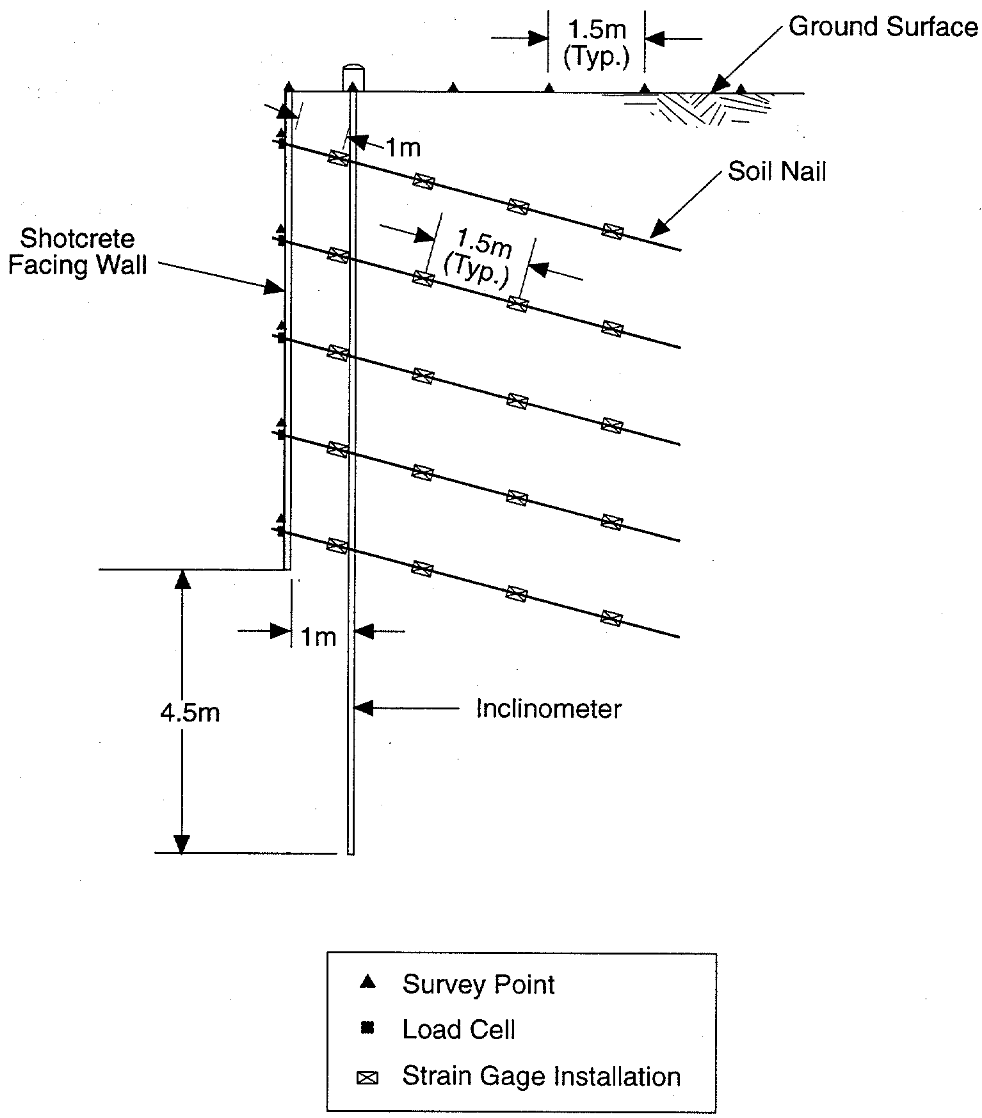
A more comprehensive monitoring plan might include the following:

- Strain gage monitoring along the length of the nail to determine the magnitude and location of the maximum nail load. Ideally, strain gages are attached to the nail tendon in pairs, and are mounted top and bottom at a 1.5m spacing circumferentially 180° apart to address bending effects. In either case the end of the bar should be inscribed so that the final orientation of the strain gage can be verified.
- Load cells to measure loads at the head of the nail. Higher quality nail load data near

the head of the nail can generally be obtained by load cells rather than by strain gages attached to the nail. The nail section immediately behind the facing is sometimes subjected to bending due to the weight of the shotcrete facing.

- Inclinometers installed from the ground surface at various horizontal distances up to one times the wall height behind the wall facing, to measure horizontal movements within the overall structure. Sometimes it might be preferable to make use of horizontal single- or multi-point borehole extensometers, particularly if access for the installation of inclinometers is difficult or if access is available for extensometer installation prior to the installation of the nails, e.g., through an existing bridge abutment or pile cap.

A typical instrumentation layout for a comprehensive monitoring plan is shown in figure 6.1.



Note:
Number and location of instruments may be adjusted to suit field conditions.

Figure 6.1 Typical Instrumentation

6.3 Soil Nail Wall Performance Monitoring Plan

A well defined, systematic plan should be developed for all monitoring programs, whether limited or comprehensive. The first step is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question. If there is no question, there should be no instrumentation. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be clearly established. More detailed discussions of appropriate instrumentation schemes, equipment requirements, etc., are contained in FHWA RD-89-043 [44], chapter 8.

The following list (based on Dunnicliff, [45]) provides the key steps that should be followed in developing a systematic approach to planning monitoring programs for soil nail walls using geotechnical instrumentation. All steps should be followed and, if possible, completed before instrumentation work commences in the field.

- Define purpose of the monitoring program.
- Define the project conditions.
- Predict mechanisms that control behavior.
- Select the parameters to be monitored.
- Predict magnitudes of change of measured parameters.
- Assign monitoring tasks for design, construction, and operation phases.
- Select instruments, based on reliability and simplicity.
- Select instrument locations.
- Plan recording of factors that may influence measured data.
- Establish procedures for ensuring reading correctness.
- Plan installation.
- Plan regular calibration and maintenance.
- Plan data collection, processing, presentation, interpretation, reporting, and implementation.
- Write instrument procurement and installation specifications.
- Prepare budget.
- Write contractual arrangements for field instrumentation services.

6.4 Parameters To Be Monitored

As part of developing the monitoring plan, the Engineer must select the parameters to be monitored. The most significant parameters should be identified, with care taken to identify secondary parameters that should be measured if they could influence the primary parameters. **The most significant measurement of overall performance of the soil nail wall system is the deformation of the wall or slope during and after construction**. Slope inclinometers at various distances back from the face provide the most comprehensive data on ground deformations.

The following list provides the important parameters that should be considered during development of a systematic approach to planning soil nail wall performance monitoring programs using geotechnical instrumentation:

- Vertical and horizontal movements of the wall face.
- Vertical and horizontal movements of the surface of the overall structure.
- Local movements or deterioration of the facing elements.
- Drainage behavior of the ground.
- Performance of any structure supported by the reinforced ground, such as roadways, bridge abutments or footings, slopes above the wall, etc.
- Loads in the nails, with special attention to the magnitude and location of the maximum load.
- Load distribution in the nails due to surcharge loads.
- Load change in the nails as a function of time.
- Nail loads at the wall face.
- Temperature (may cause real changes in other parameters and also affect instrument readings).
- Rainfall (often a cause of real changes in other parameters).

6.5 Soil Nail Wall Performance Monitoring Instruments

Soil nail wall performance monitoring instruments should be selected based on the parameters to be measured, the instrument's reliability and simplicity, and the instrument's compatibility with the readout devices specified for the project or already the property of the owner. Other factors to be considered include the influence of the instrument's installation on construction (e.g., access and time required) and the skills of the personnel who will read the instruments. A brief discussion of the various types of monitoring instruments typically employed for assessing soil nail wall performance is provided below.

6.5.1 Slope Inclinometers

The most significant measurement of overall performance of the wall system is the deformation of the soil nail wall during and after construction. Slope inclinometers, preferably installed at about one meter behind the soil nail wall face, provide the most comprehensive data on wall deformations.

Inclinometers are a well-established technology and are commercially available from several manufacturers. The measuring system typically consists of a portable probe that measures its own orientation relative to vertical. The probe is mounted on wheels and is raised or lowered within a grooved casing installed vertically in the ground. Readings are recorded by hand or on data loggers.

Detailed specifications for inclinometer selection and installation are presented in appendix D - Guide Specification for Soil Nail Wall Instrumentation. The following general guidelines should be followed for all installations:

- The bottom of the grooved casing should be installed in a stable portion of the ground mass, typically several meters beneath the lowest expected zone of movement.
- The casing should be aligned so that the grooves, which are typically at 90-degree intervals, are parallel and perpendicular to the wall face.
- The borehole containing the casing should be completely backfilled so that no voids are present around the casing. This is typically done with a grout mixture tremmied from the bottom of the inclinometer borehole. The grout mixture should be relatively weak (e.g., a lean cement) so that it does not provide any structural reinforcement of the surrounding ground.
- The top of the casing should be in firm contact with the surrounding soil and should be adequately protected from damage.

Details of a typical inclinometer installation are shown in appendix D.

6.5.2 Survey Points

Soil nail wall face deformation can be measured directly by optical surveying methods or with electronic distance measuring (EDM) equipment. Also, ground movements behind the soil nail wall can be assessed by monitoring an array or pattern of ground surface points established behind the wall face and extending for a horizontal distance at least equal to the wall height. In addition, reflector prisms attached to selected nails permit electronic deformation measurements of discreet points on the soil nail wall face. Frequent monitoring of the ground during the progress of construction allows the actual performance to be checked against the design assumptions. It also provides a real-time record of performance, thereby allowing modification of the construction procedure in response to changed conditions. This can be particularly useful if wall deformations become significant because poorer ground than originally anticipated is encountered or the contractor uses inappropriate construction methods.

The survey system should be capable of measuring horizontal and vertical displacements to an accuracy of 3 mm or better.

6.5.3 Soil Nail Strain Gages

Soil nails instrumented with strain gages allow assessment of the soil nail load distribution as the excavation progresses and following completion of the soil nail wall installation. By strain gauging individual nails in the laboratory and during field tests, the development and distribution of the nail forces may be measured.

Detailed specifications for strain gage selection and installation are presented in the Appendix D Guide Specification. The following general guidelines should be followed for all installations:

- Weldable vibrating wire strain gages are recommended for ruggedness and low susceptibility to electrical signal degradation.

- Temperature sensors should be included at each strain gage location to allow temperature correction. These are particularly important near the face. (Some gages contain integral temperature sensors.)

- At each measuring location on the nail, gages should be installed in pairs 180° apart in a vertical plane (top and bottom) to evaluate bending and provide an average strain reading.

- The surface of the nail should be carefully prepared and strain gage installation performed in accordance with the manufacturer's recommendations.

- The completed strain gage installation should be covered with a soft, waterproof material to ensure that (1) moisture does not penetrate the gage, and (2) the gage is mechanically decoupled from the surrounding grout.

- Preferably, some type of mechanical assembly should be installed at each gage location to break the grout column and ensure that all load is transferred to the nail bar at this point. This approach will eliminate data interpretation problems associated with grout/nail interaction. Because the grout has some tensile strength, it will carry a portion of the total load. This load will depend to a large degree on the in-place deformational characteristics of the grout and the interaction between the grout and borehole wall, both of which are difficult to evaluate. Thus, while strain measurements in the grout and nail are readily achievable, conversion of these measurements into nail loads is difficult to achieve with accuracy. An alternative approach for obtaining accurate measurements of nail load is to introduce a break in the grout at the location of the strain gage, thereby forcing the nail to carry the entire load at this location. Examples of potential approaches include a disk coated with a release agent, two disks butted together, or a soft zone around the gages formed with foam. See further discussion in the Appendix D Guide Specification commentary 2.3.

- Signal cables should be protected by installation in conduit, from construction damage and vandalism, for example.

- In some areas of the country, lightning protection may be required.

Details of a recommended strain gage installation are shown in the Appendix D Guide Specification.

6.5.4 Load Cells at the Nail Head

Load cells installed at the soil nail head are used to provide reliable information on the actual loads that are developed at the facing.

Specifications for load cell selection and installation are presented in appendix D. The following general guidelines should be followed for all installations:

- Unless the load cells are temperature-compensated, temperature sensors should be included with each cell to allow temperature correction.

- The axis of the load cell should be aligned with the axis of the soil nail, to prevent eccentric, lateral, or other types of non-uniform loading. Hardened steel spherical bearings are preferred. Bearings and other support hardware should be well-lubricated prior to installation.

- The bearing surface under the load cell should be firm and unyielding. This will prevent apparent loss of load that could otherwise result from creep underneath the cell. High-early strength cement mortar and steel bearing plates (12-mm-thick minimum) are typically used.

- Load cells should be protected from construction damage and vandalism. Signal cables should be placed in a conduit for protection.

- In some areas of the country, lightning protection may be required.

Details of a typical load cell installation are shown in the Appendix D Guide Specification.

CHAPTER 7. SHOTCRETE

7.1 Introduction

7.1.1 General

Shotcrete is concrete or mortar projected at high velocity onto a surface.

The typical highway **permanent** soil nail wall will consist of a temporary shotcrete "construction facing" and a final permanent cast-in-place facing. A **temporary** soil nail wall will have only a temporary shotcrete construction facing. The shotcrete construction facing is typically considered sacrificial in terms of its long-term structural contribution, but must support the face loadings until the permanent facing is installed. An analogy would be use of timber lagging which provides the temporary support prior to placement of a permanent CIP facing for a tieback wall. In some applications the shotcrete facing may be designed for permanent support, particularly where an architectural finish is not required or where a non-structural pre-cast tilt-up panel facing is employed.

A basic description of the shotcrete process, the materials, and the physical properties of the hardened product are included in this chapter. Also included are guidelines on shotcrete quality control and testing. Emphasis is placed on the field aspects of shotcreting in soil nailing and on the wet-mix placement process because wet-mix is currently the most common shotcrete used for soil nail wall facings. Guide specifications for both temporary shotcrete construction facing and permanent shotcrete facing are included in Appendix C.

7.1.2 The Function of Shotcrete in Soil Nailing

The function of shotcrete in soil nailing is both to transfer the earth pressure reaching the wall face from the soil to the nails and to prevent deterioration of the excavated soil face. Shotcrete is usually applied soon after excavation of a lift and placement of the nails, but may also be applied before nail installation. Its initial function is to stabilize the surface from raveling. Subsequently, as the ground relaxes against the shotcrete facing, and further excavation lifts are made, the reinforced shotcrete resists and transfers soil pressures to the nails.

In soil nailing, the flexural and shear strength of the facing dictate its load carrying capacity. The shotcrete must restrict the movement of the surrounding ground and be able to adapt to some ground movement. Both the early strength and the toughness characteristics (ability to absorb energy and support load after cracking) of the shotcrete facing are important.

From a quality perspective, the construction facing is less critical than the permanent facing, except from worker safety perspectives. Because it is the backing for the permanent facing, final quality of the construction facing shotcrete is important only to the degree that it will not degrade excessively due to aggressive groundwater or freezing and thawing, will protect embedded steel from corrosion, and will retain integrity around the nail head plates. Permanent shotcrete facing has to be of a structural quality corresponding to that required by ACI 318 [38] or other design codes for the particular exposure.

The critical shotcrete properties are:

- Shear and flexural strength (to prevent punching around nail-facing connections).
- Bond to reinforcement (for stress transfer).

7.1.3 Definitions

Following are some common definitions unique to shotcrete. They are primarily extracted from ACI 506.R [46] in which a more extensive list is presented.

Air ring. Perforated manifold in nozzle of wet-mix shotcrete equipment through which high pressure air is introduced into the material flow.

Blowpipe. Air jet operated by nozzleman's helper in shotcrete gunning to assist in keeping rebound or other loose material out of the work.

Dry-mix shotcrete. Shotcrete in which most of the mixing water is added at the nozzle. Use in soil nailing limited mostly to remote sites with difficult access.

Flash coat. Thin shotcrete coat applied from a distance greater than normal for use either as an initial coat to stabilize the ground surface or a final coat for finishing; also called Flashing. Nozzle work usually done rapidly.

Ground wire. Small-gage, high-strength steel wire used to establish line and grade for shotcrete work; also called alignment wire, guide wire, screed wire, or shooting wire.

Gun finish. Undisturbed layer of shotcrete as applied from nozzle, without hand finishing. Also known as as-shot finish.

Nozzleman. Worker on shotcrete crew who manipulates the nozzle, controls consistency with the dry process and controls final disposition of the material.

Overspray. Shotcrete material deposited away from intended receiving surface.

Rebound. Shotcrete material leaner than the original mixture and consisting of predominantly coarser aggregate that ricochets off the receiving surface and falls to surfaces below.

Sand pocket. A zone in the shotcrete containing fine aggregate with little or no cement.

Sloughing. Subsidence of shotcrete, due generally to excessive water in mixture; also called Sagging.

Wet-mix shotcrete. Shotcrete in which all of the ingredients including water, are mixed before introduction into the delivery hose. Compressed air is introduced to the material flow at the nozzle. If a strength accelerating admixture is used, it is normally added at the nozzle. Wet-mix is most commonly batched at conventional concrete batch plants and delivered via conventional ready-mix trucks. More commonly used for soil nail wall facings than is dry-mix.

7.1.4 Types of Shotcrete

There are two methods of placing shotcrete - the wet-mix and dry-mix processes. In dry-mix, aggregate and cement are blended and deposited in the gun; the mix water is added at the nozzle and is therefore instantaneously adjustable at the work face; the material is conveyed by compressed air from the gun through the nozzle. In wet-mix, a plastic mix of aggregate, cement water and admixtures are conveyed to the nozzle by hydraulic pump and nozzle velocity is achieved by compressed air (see figure 7.1).

Wet-mix is often preferred for soil nailing because of:

- Higher volume throughput, typically 6 to 8 m^3/hour for wet-mix versus 4 to 6 for dry;
- Less rebound, typically 5 percent for wet-mix and 15 percent for dry-mix;
- Less reliance on the competence of the nozzlemen because of the additional function of adding water at the dry-mix nozzle; and
- More readily available equipment-a concrete pump versus the gun and possibly moisturizer for dry-mix, and the convenience of ready-mix supply from commercial batch plants.

Wet-mix is generally simpler, faster and more economical. However, dry-mix may still find use in soil nailing under the following conditions:

- Extremely wet or difficult soil conditions where the ability to adjust the wetness of the mix, or to add powdered accelerators on demand, will minimize sloughing. Dry-mix is preferable for flash coat application.
- Some geographical areas where the shotcrete contractors are more familiar with dry-mix.
- In areas of difficult access or where small volumes of shotcrete are required, in which case the versatility of dry-mix (rapid stopping and starting) is attractive.

7.1.5 Development of Shotcrete in the USA

Historically, many problems with shotcrete quality stemmed from poor nozzling techniques resulting in sand lenses, layering, and trapped rebound and overspray. These were more prominent in dry-mix, leading to the more demanding nozzlemen workmanship for that process.

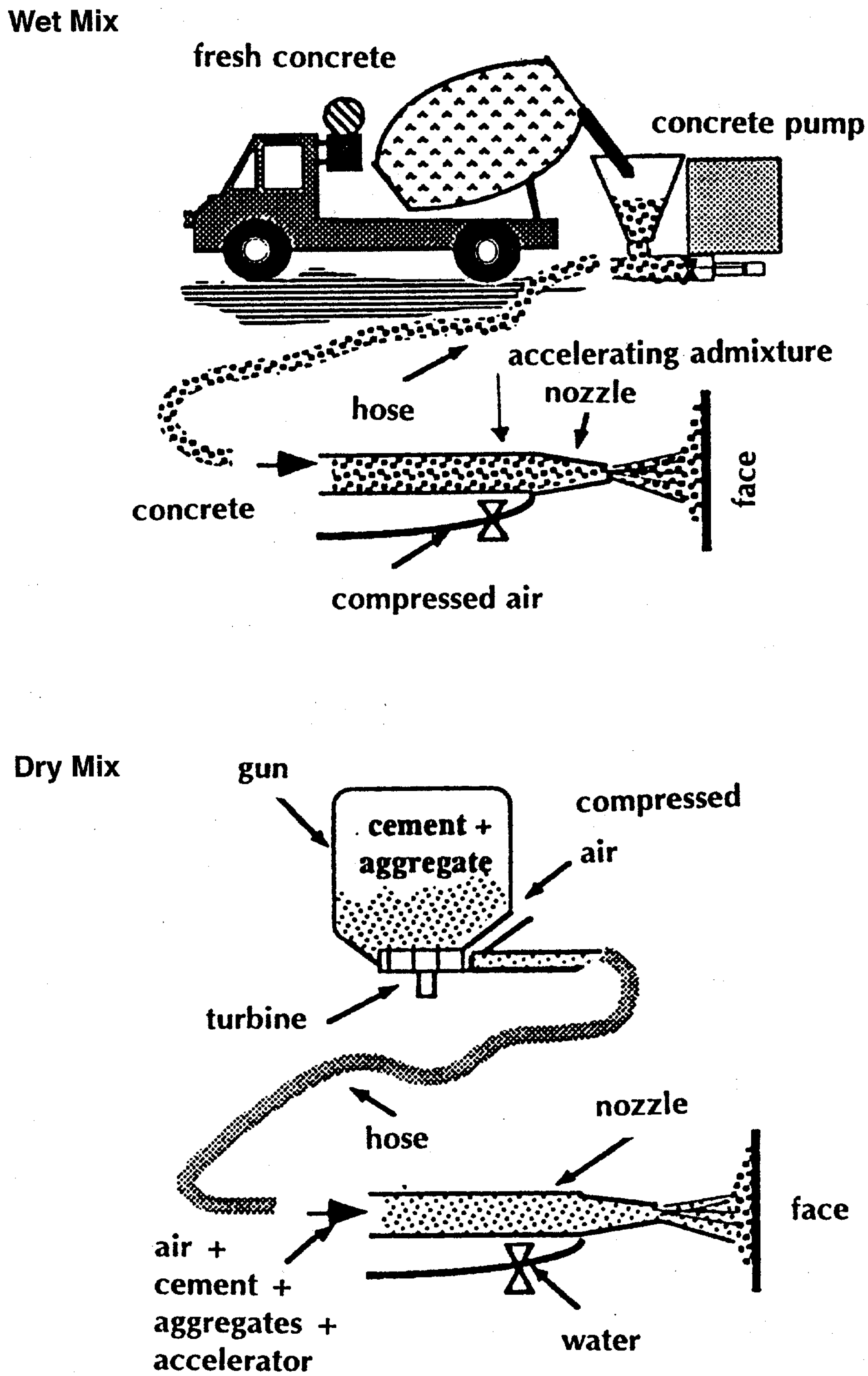


Figure 7.1 Schematic of Wet-Mix and Dry-Mix Processes

Improved recognition of what constitutes quality shotcrete, more demanding specifications, shotcrete nozzleman training and certification, and better techniques for evaluating in-situ shotcrete have led to minimization of problems.

Today, shotcrete is a recognized structural material. Its use is increasing in North America as engineers become both more comfortable with the process and more confident that a quality product can be produced.

7.2 Shotcrete Equipment

Figure 7.1 shows schematics of the shotcrete wet-mix and dry-mix equipment. Basic wet-mix equipment typically consists of:

- A concrete pump with a 75-mm diameter delivery line reduced to a 40 to 50-mm diameter rubber hose for practicality of handling by the shotcrete crew.
- An air compressor with at least 60 l/sec capacity.
- A nozzle. These are proprietary in nature and of various materials (hard rubber, plastic, or steel) and configurations (straight, tapered, or bulbous).

There may also be ancillary equipment such as dispensers for fibers and admixtures.

7.3 Shotcrete Materials

Shotcrete, like concrete, consists basically of a paste (cement plus water) and an inert fine and coarse aggregate filler. To the paste can be added mineral and chemical admixtures to reduce water demand, entrain air, disperse particles, and increase or decrease the time of set. Specifications for these materials are generally the same as for concrete.

Shotcrete may include the following materials:

Cement Portland cements of all Types are used in shotcrete-Type I and II for normal use, Type III where early strength is required, and Type V for sulfate exposure. Cements should meet AASHTO M85 [47].

For hot weather shotcreting, it is preferable to limit the cement temperature to 65° C maximum to reduce rapid set.

Specialty cements such as rapid hardening may find use in shotcrete requiring early set or early strength.

Aggregate Uniformly graded concrete aggregate meeting the requirements of AASHTO M6 [48] for fine aggregate and M80 for coarse is suitable. A commonly used gradation specification for soil nailing shotcrete is:

Metric Sieve	Percentage Passing by Weight
12 mm	100
10 mm	90 - 100
5 mm	70 - 85
2.5 mm	50 - 70
1.25 mm	35 - 55
0.63 mm	20 - 35
0 - 315 mm	8 - 20
0 - 160 mm	2 - 10

Ref: ACI 506.2 [49]

Optimum wet-mix aggregate proportioning has at least 25 percent greater than 5-mm size. The advantages of using larger size aggregate, or higher quantities of coarse aggregate, are increased impact on the fresh shotcrete during shooting (which increases compaction and therefore density) and increased volume stability, particularly reduced shrinkage.

Reinforcing Steel

Wire Mesh Mesh should be no smaller than 100 mm square so that it does not act as a barrier for the shotcrete nozzle stream. In soil nailing use of sheet mesh rather than rolled mesh will simplify placement in the desired plane.

Bar Any normal concrete rebar can be used in shotcrete. Ideally, it should be limited to No. 16 (Soft Metric designation - see appendix F) size to minimize interruption with the shotcrete nozzle stream.

Fibers The reinforcement of shotcrete with discrete steel fibers is now becoming more common. Historically, wire mesh has been used, but fibers have been substituted when it is difficult to attach or support the mesh at the proper distance from the face. ASTM C1116 [50] presents a performance-based specification for fiber reinforced shotcrete. ACI 506.1R [51] presents a state-of-art description.

Admixtures Satisfactory wet-mix shotcrete can be produced without admixtures. However, for reasons of economy, (i.e. reduction in mixing water, control of set times, and modification of plastic properties), admixtures are sometimes used for soil nailing shotcrete.

The various types of water reduction and set control admixtures in AASHTO M194 [52] can be used in wet-mix. For reduced set time, normal plant-added accelerators find little use because it is more effective to add accelerator at the nozzle and obtain set within a few minutes or even less than a minute. Rapid accelerators are used to speed shotcrete construction. They permit shotcreting over wet ground, increased thicknesses of lift, and faster additions of shotcrete without sloughing. Shotcrete accelerators are known to reduce the later age strength gain, some by significant amounts, so both compatibility and strength need to be checked. Such rapid accelerators typically depend for their effectiveness on reaction with the tricalcium aluminate in the cement and therefore they are sensitive to the cement chemistry. ASTM C1117 [53] provides a procedure for evaluating shotcrete accelerator effectiveness. Shotcrete accelerators can be costly, adding up to 10 to 15 percent to the shotcrete cost.

Accelerators containing chloride ions are discouraged because they may contribute to reinforcing steel corrosion and reduce the long term shotcrete quality. A limit of 0.10 percent chloride as a percentage of the cement mass is often used.

The use of air-entrainment in wet-mix shotcrete is discussed in section 7.4.3. Air entraining agents should comply with AASHTO M194 [52].

Superplasticizers (high range water reducing admixtures) are now commonly used to achieve better dispersion of both cement and mineral admixtures, further reductions in mixing water content and temporary increased pumpability without increased water. The increased slump with superplasticizer is lost in ½ to 1½ hours. Superplasticizers can be added at the plant or site and are available with both normal and extended window slump loss characteristics; the former is preferable for soil nailing shotcrete because they result in early in-place stiffening. However, because of shortened plastic life, these must be added on site. Superplasticizers are mandatory for dispersion of the fine particles if silica fume is used. Many superplasticizers act as set retarders; some at high dosages extend set for many hours.

ASTM C1141 [54], "Specification for Admixtures for Shotcrete," presents properties for all shotcrete admixtures.

7.4 Shotcrete Properties

7.4.1 Factors Determining Shotcrete Quality

Two of the most significant factors determining shotcrete quality are its paste content and in-situ density. Shotcrete mixtures have high cement factors, typically 355 to 415 kg/m^3, dictated by the need to provide fines for pumpability and shootability. Therefore, corresponding low water:cement ratios can be expected, with resulting high strength and durability and low permeability if proper in situ compaction is achieved. The richness of the mixture is further enhanced in the loss of some coarser aggregate particles during shooting by rebound, resulting in final in-place proportions that are higher in paste than the original mix. Given proper mixture proportioning and reasonably well graded aggregate, the in situ density depends on field compaction, primarily proper nozzle technique, and velocity of the nozzle stream. Consequently, emphasis must be placed on assuring that the nozzleman is competent.

7.4.2 Common Specification Requirements

Typical specifications for soil nailing shotcrete address two basic properties:

- **Strength**. The critical strengths are flexure and shear, which are related to the more easily tested compressive strength. As a conservative rough guideline, the flexural strength is taken as 10 to 12 percent of the 28-day compressive strength.
- **Durability**. The commonly specified 28 MPa (4000 psi) 28-day compressive strength requirement normally assures a water:cement ratio of less than 0.50, which is adequate for all but the most demanding exposures.

Table 7.1 presents suggested material specification guidelines for soil nailing wet-mix shotcrete. Figure 7.2 illustrates typical range of shotcrete compressive strength in relation to curing time.

7.4.3 Shotcrete Mixture Proportioning and Air Entrainment

Wet-mix proportioning can be done using the normal concrete procedures, such as those in ACI 211.1 [55], with adjustments to such items as mixing water content for the higher fine aggregate content and pumpability through small diameter hoses. A theoretically ideal mixture proportion is of no value in the field unless it can be pumped and shot with the Contractor's equipment and unless it results in high in situ density.

Slump is largely self-controlling in wet-mix shotcrete-too wet and it sloughs on the face; too dry and it will not pump.

Air can be entrained into wet-mix shotcrete. Recent research has shown that the loss of entrained air from wet-mix shotcrete during pumping and application onto the face can occur as follows and the final in-place air content will typically be between 4-5% irrespective of how high the initial air content is:

Mixer Truck	Pump	Nozzle	Final In-Place Facing
Initial 7 to 10%	Lose 1 to 1½%	Lose up to 4%	Residual typically 4 to 5%

TABLE 7.1
SUGGESTED SPECIFICATION GUIDELINES
WET-MIX SHOTCRETE FOR SOIL NAILING

Property	Standard Or Method Of Test	Requirement	Comments
Materials			
Cement	AASHTO M85 [47]		Specify Type(s)
Admixtures	AASHTO M194 [52] or ASTM C1141 [54]	Compatibility to ASTM C1117 [48]	May specify limit on Chloride to 0.15% of cement weight.
Aggregate general quality gradation	AASHTO M6 [48], M80 [56] AASHTO M43 [57]	Grading No. 2 (ACI 506.2) [44]	Desirable to include some 10 mm coarse aggregate
Supply			
Ready-mix	ASTM C94 [58]		If volume batching permitted, use ASTM C685. Site volume batching may be a preferred method of supply for small projects. It assures a fresh and interruptible supply of mix.
Plastic Properties			
Slump	AASHTO T119 [59]	50 to 100 mm	Controlled largely by pumping and sloughing restraints so testing is not normally required.
Air content	AASHTO T152 [60] ASTM C231 [61]	7 to 10%	Measured at the truck. Air entrainment usually only required when there is freezing and thawing exposure (e.g., permanent shotcrete in cold climates).
Maximum water/cement ratio		For construction facing _ 0.50; for permanent facing _ 0.45.	Lower values may be required for severe exposure, eg. many freeze-thaw cycles and exposure to de-icing salts or acidic soils.
Temperature of mix		10 to 30°C	Shotcrete is often a "thin" section so more stringent temperature limits than normal concrete apply.
Hardened Properties			
Compressive strength	ASTM C42 [62] on cores from test panels or in situ shotcrete	28 MPa at 28 days; 15 MPa at 3 days	Samples tested in wet condition; 3 cores averaged for a test. Acceptable if: Average of 3 cores $\geq 0.85\ f'_c$ No individual Core $\leq 0.75\ f'_c$
Porosity	ASTM C642 [63] on cores from panels or in situ. Average 2 cores per test.	Test at 7 days. Boiled absorption $\leq$ 8%.	Only required for permanent shotcrete.

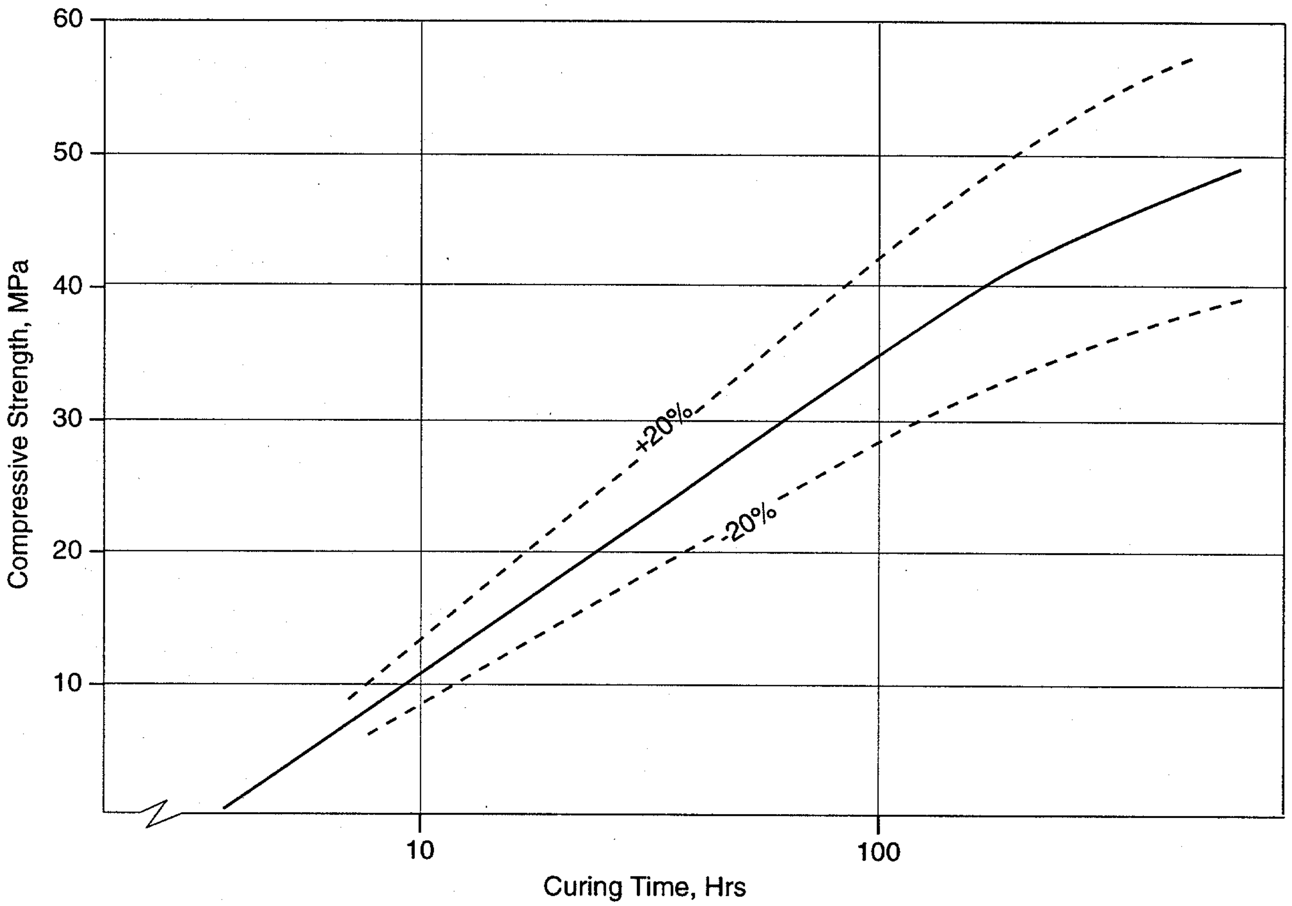


Figure 7.2 Typical Age Compressive Strength Relationships of Shotcrete

Hence, loss of air through wet-mix nozzling is inevitable. Trials of up to 20 percent initial air content have still resulted in 4 to 5 percent on the face. Research has shown that adequate air void factors are normally achieved and this amount of residual air is sufficient to provide good long-term durability.

Air entrainment usually improves workability and reduces both mixing water and bleeding in a concrete or shotcrete mix. Increasing air content will proportionally decrease strength (1 percent air commonly reduces compressive strength about 5 percent). Unless air is needed for long-term freezing and thawing durability, it is not essential, and is typically not required in temporary shotcrete construction facings. However, the loss of air through the nozzle described above results in a corresponding loss of slump so the shootability of the mix is improved. For this reason, many contractors are choosing to add air entrainment to increase productivity.

Typical shotcrete wet-mix mixture proportions for one cubic meter are presented in table 7.2.

For dry-mix, the shooting process entrains a certain amount of air bubbles of sufficient size to provide freezing and thawing protection. It certainly will not produce spacing factors less than the 0.20 mm normally required; typical spacing factors in the order of 0.33 mm have been recorded. A combination of this small amount of bubbles of entrainment size plus the low water:cement ratio will provide adequate freezing and thawing protection. Much of the dry-mix shotcrete tested to the relatively severe ASTM C666 [64], Procedure A, has shown good durability. The only exception is dry-mix with high accelerator dosages.

Therefore, in summary, air entrainment is not required in wet-mix used for temporary shotcrete construction facings or in dry-mix used for temporary or permanent shotcrete facings. Air entrainment is recommended in wet-mix used for permanent shotcrete facings.

TABLE 7.2
TYPICAL WET-MIX SHOTCRETE MIXTURE PROPORTIONS

Material	1 Cubic Meter Batch 28 MPa* Specification
Cement	390 kg
Water	160 Liters
Fine aggregate	1350 kg + moisture correction
Coarse aggregate, 10 to 15 mm max. size	400 kg
Admixtures	
Water reducing	Yes
Air entraining	Yes, if air is specified
Superplasticizer	Only if silica fume used
Silica Fume	Not normally used unless difficult shooting, then substitute for 30 to 40 kg cement
Steel fibers	50 to 70 kg if specified

* Required 28-day compressive strength. This mix will typically produce compressive strengths over 28 MPa in most parts of the USA. The 390 kg/m^3 is a common minimum specified cement content.

For shotcrete mixtures there are two opposing requirements-"shootability" and "pumpability". Shootability is the ability of a mix to stick to a surface. It includes build-up thickness and resistance to sloughing. For pumping, a low flow resistance and low viscosity are ideal; for shooting, a high flow resistance and high viscosity are ideal. Once on a face, a shotcrete mix with high flow resistance and high cohesion or viscosity will tend to stay there. Contractors want high shootability to increase one-pass thicknesses. With the proper mix design, shootability to 300-mm thickness can readily be achieved without sloughing or sag cracks below rebar. Flow resistance at the face is markedly increased by compaction of the shotcrete mix, which is achieved by the impact of the nozzle stream on the surface and results in driving out the copious amounts of air in the stream. This air is both entrapped and entrained.

7.5 Proper Shooting Techniques

7.5.1 Nozzling

The key to good shotcrete quality is good nozzle work. The velocity of the material at impact is an important factor in determining the ultimate properties of the shotcrete. For most applications where standard nozzle distances of 0.6 to 2.0 m are used, material velocity at the nozzle and impact velocity of the material particles are almost identical. However, there can be pressure loss in long air lines, reducing the impact velocity.

Shotcrete may be applied in layers or in single pass, depending on the position of the work, the cohesiveness and stiffening characteristics of the mix and the thickness required. Vertical surfaces may be applied in layers or a single thickness; as noted above, thicknesses to 300 mm can be accomplished in a single pass. In any case, thickness of a layer is governed mainly by the requirement that the shotcrete should not sag. Sags or sloughs that go undetected and are not removed can hide internal cracks and voids.

Each layer of shotcrete is built up by making several passes of the nozzle over a section of the work area. Whenever possible, sections should be gunned to their full design thickness in one layer, thereby reducing the possibility of cold joints and laminations. Joints are often porous. The nozzle must be continuously manipulated so that material is dispersed and mixed as it hits the face. As a general rule, the nozzle should be held perpendicular to the receiving surface, but never inclined more than 45° to the surface (figures 7.3 and 7.4). The shotcrete should emerge from the nozzle in a steady, uninterrupted flow. When the nozzle is held at too great an angle from the perpendicular, the shotcrete rolls or folds over, creating an uneven, wavy textured surface that can trap rebound and overspray.

In gunning walls, application should begin at the bottom. The first layer should, if possible, completely encase the reinforcement adjacent to the form or soil face. In soil nailing, shotcrete lift heights of 1 to 2 m are common.

An important technique for applying a thick single layer of shotcrete against a vertical surface is "shelf" or "bench" gunning. Instead of gunning directly against the surface, a thick layer of material is built up, the top surface of which is maintained at approximately a 45° slope (figure 7.5).

Proper nozzle techniques for shooting into formed corners such as bulkheads in soil nail walls is shown in figure 7.6.

7.5.2 Minimizing Rebound

The Achilles heel of shotcrete is rebound. When shotcrete strikes the excavation face or the reinforcement, some of the larger aggregate particles tend to ricochet. As shotcrete builds up it acts as a bed and rebound is reduced. Parameters influencing the amount of rebound can be separated into two categories: parameters related to the shooting technique, and those related to the mix composition. The most important parameters related to the shooting technique are:

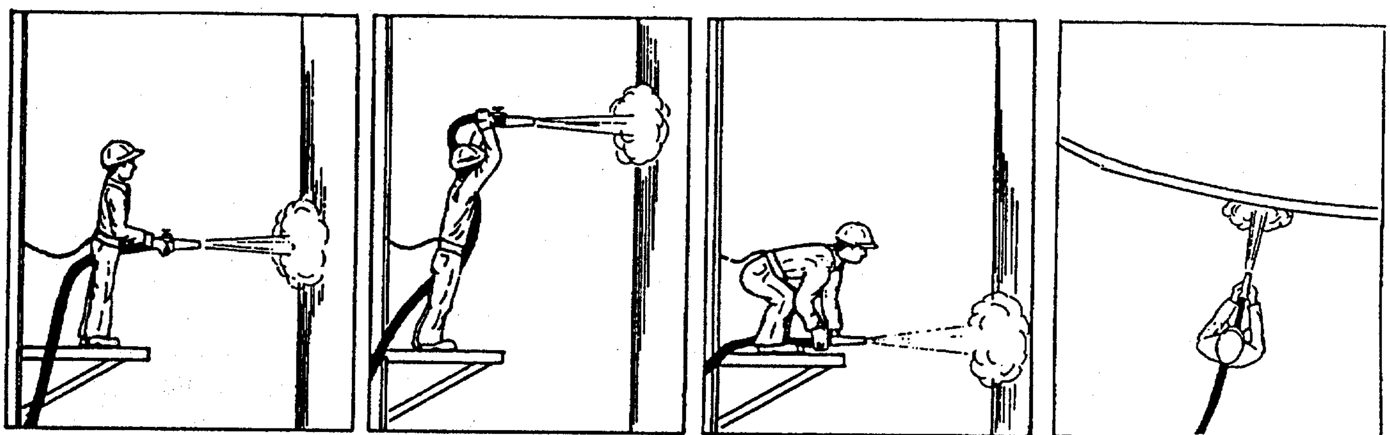


Figure 7.3 Nozzle Gunning Position

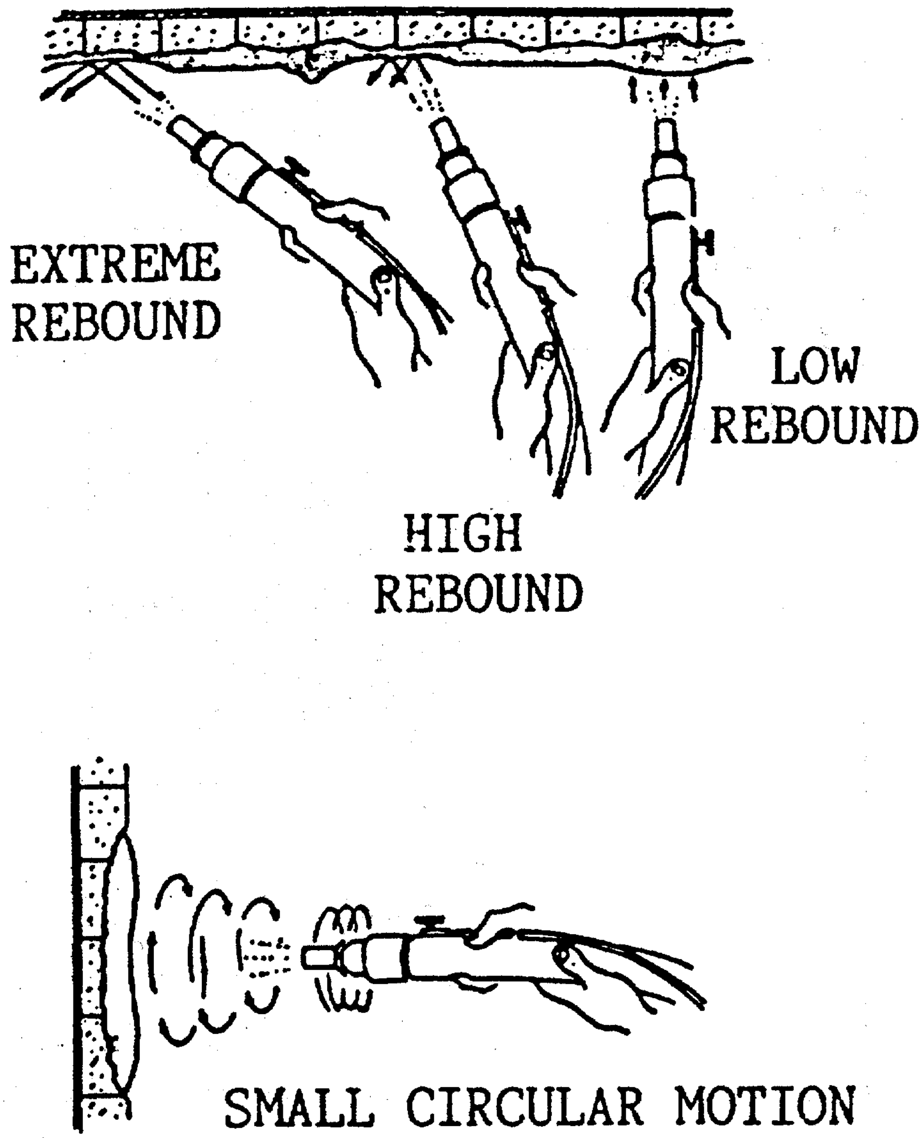
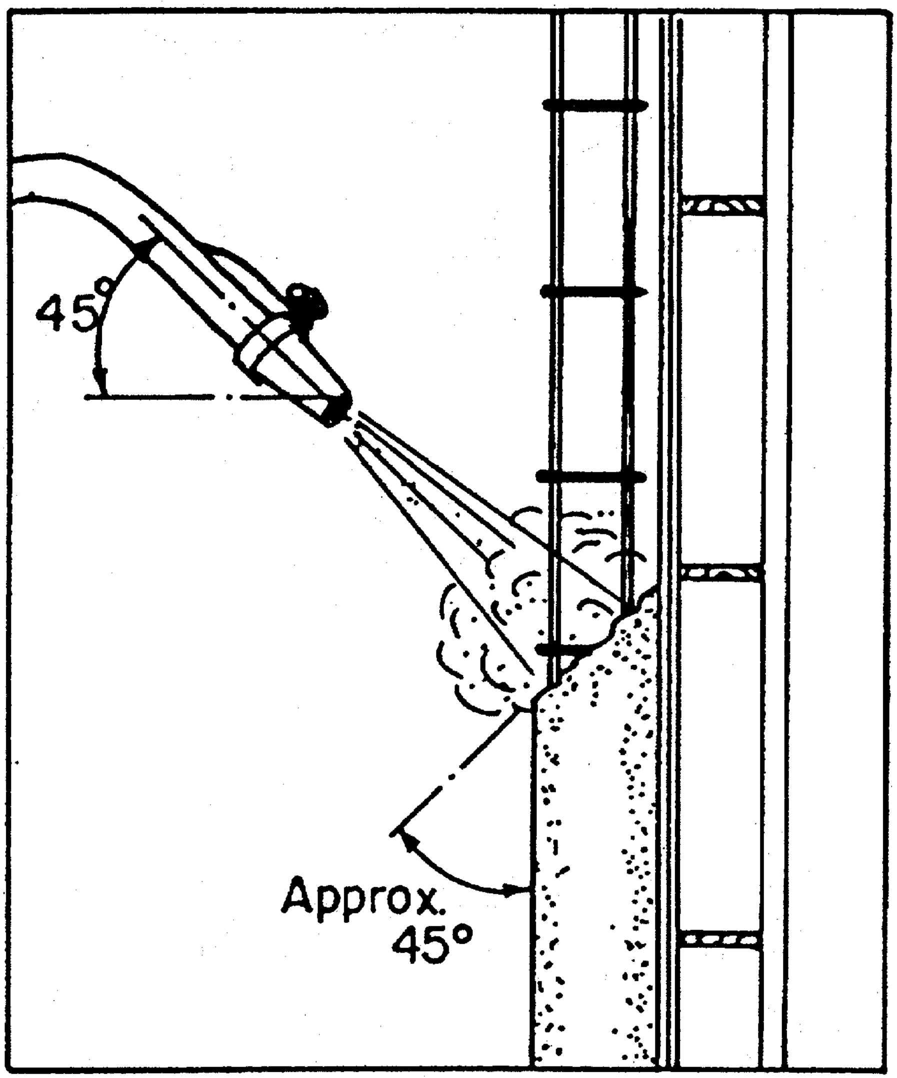


Figure 7.4 Nozzling Techniques



CORRECT SHOOTING
THICK APPLICATIONS

Figure 7.5 Placing Shotcrete in Thick Walls by Bench Gunning

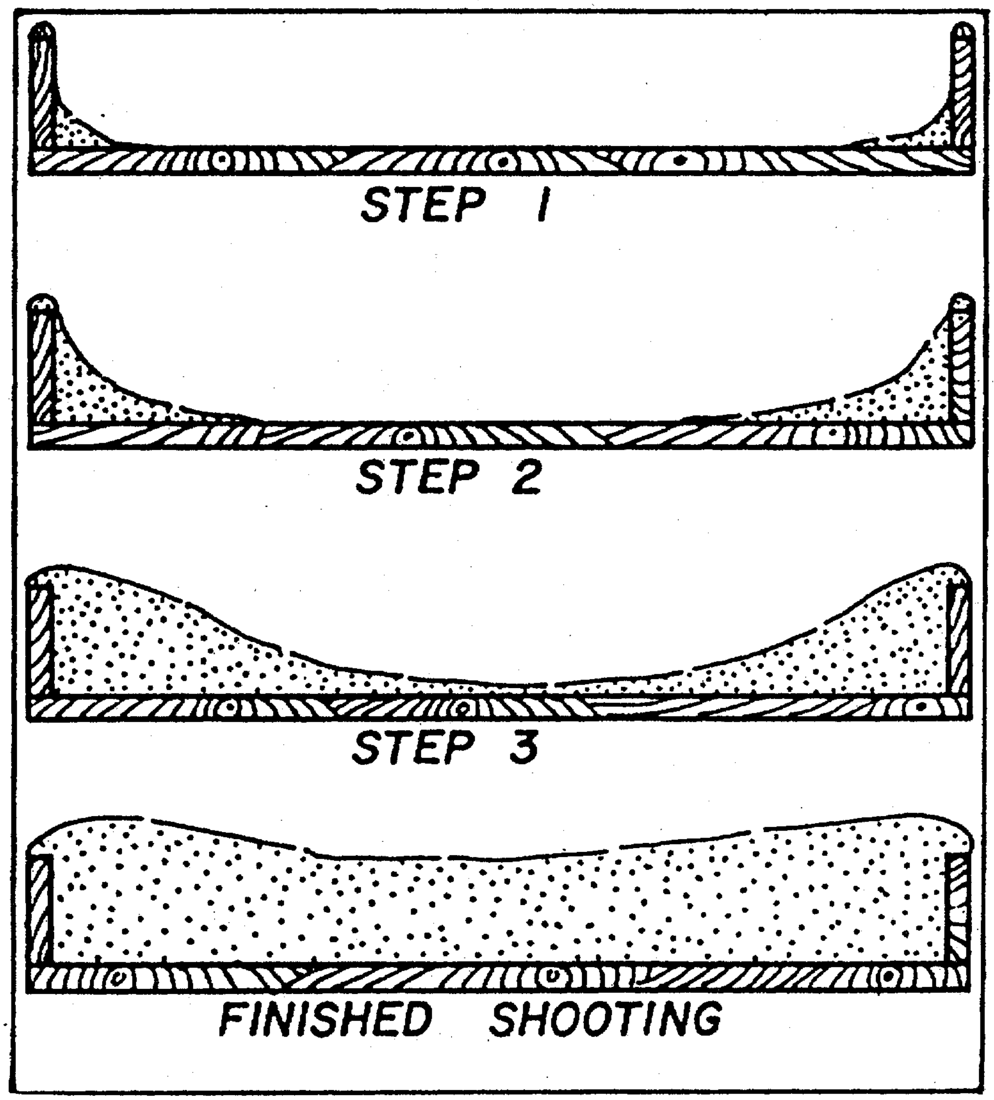


Figure 7.6 Gunning Corners of Formwork

shooting position, angle, thickness and presence of reinforcement (bars or mesh). Loose mesh or rebar will also increase rebound. The most important parameters with respect to the mix composition are aggregate characteristics and content, cement content, and presence of silica fume if used.

In recent years, shotcreting techniques have been developed to minimize rebound. For wet-mix, some Contractors report virtually no rebound, although values of 5 percent on vertical surfaces are still common. Reductions can be achieved by:

- Reducing nozzle velocity,
- Reducing amount of coarse aggregate or fiber in the mix,
- Increasing nozzle distance,
- Adding silica fume to the mix to improve cohesion, and
- Adding accelerators.

Of these, all will reduce the in-situ quality of the shotcrete except the addition of silica fume and possibly accelerator.

7.5.3 Shooting Reinforcement in Shotcrete

The soundest shotcrete is usually obtained when the reinforcement is designed and positioned to cause the least interference with the nozzle stream. This can be accomplished by:

- Tying intersecting wire mesh or rebar laps firmly; tie to pins or anchors to avoid vibration or displacement during shooting;

- Fabricating splices so that the reinforcements are aligned behind one another in a plane parallel to the direction of shooting;

- Using non-contact splices; where bars overlap, do not tie together but leave apart a minimum of 50 mm but not more than 1/5 the lap length or 150 mm; and

- In two layer systems, leave the outside layer open (i.e., with a rebar spacing greater than 150 mm).

A related problem with soil nailing is displacement of drainage mats due to inadequate support of the mat.

7.6 Shotcrete Quality Control and Testing

7.6.1 Basic Purpose

The fundamental purpose of the owner's quality control is to ensure both that the Contractor's quality assurance is being implemented and that work is being conducted in reasonable accordance with plans and specifications. This is particularly relevant for a performance-based specification such as is commonly used for soil nailing.

Experience has shown that prequalifying the shotcrete nozzleman, equipment and mix design will help ensure a reasonable quality product. A properly conducted pre-qualification program will identify problems or deficiencies and allow for correction before they are incorporated into the work.

7.6.2 Shotcrete Submittals

Submittals for the shotcrete portion of a soil nailing project could be expected to include materials qualifications, mix designs, reinforcement details, and nozzleman qualifications.

Materials qualification normally take the form of mill certificates for cement and reinforcement and aggregate test reports showing compliance to the specifications. Owners should check that they are current and should request updates during construction if the volume of shotcrete is significant. Admixture submittals are normally technical literature from the manufacturer. Mix designs should be checked for specification compliance.

7.6.3 Qualifications

7.6.3.1 Contractors

It is common to require the Contractor to present his qualifications to do the work. This is part of basic quality control. Items which may be requested as qualification submittals are:

- Reference shotcrete soil nailing projects completed by the Contractor - volume/size, location, resident engineer, type of shotcrete (construction vs. permanent), finishing.
- Name of proposed superintendent and resumé of relevant experience with project engineer contact person(s).
- Quality control procedures to be employed.

7.6.3.2 Nozzlemen

It has been emphasized previously that quality shotcrete requires a competent and conscientious nozzleman. ACI 506.3R [65] Guide to Certification of Shotcrete Nozzlemen presents proven procedures for nozzleman certification. They consist of two stages - a written examination on shotcrete principles (examination can be presented orally to circumvent literacy problems), and a field demonstration that evaluates both shooting techniques, including encapsulation of reinforcing steel, and ability to produce shotcrete of a given compressive strength.

The qualification process can be project specific using the same mix, equipment, and personnel proposed for the work.

Certification can be a positive experience providing both guidance and recognition to the nozzleman. Any competent nozzleman will readily qualify. Currently, the examination must be done by a testing laboratory or inspector familiar with shotcrete; ACI is considering setting up the program under their auspices. Certification is often combined with pre-construction testing. Nozzlemen who qualify should be given an appropriate written record. The specification requirement for nozzlemen to shoot pre-construction test panels can be waived for nozzlemen who have been certified in accordance with ACI 506.3R [65] requirements.

7.6.4 Pre-Construction Testing

Detailed guidelines for shotcrete pre-construction testing are given in ACI 506.2 [49] and ACI 506R [46]. They cover shooting of test panels (which can also provide qualification of the mix design) with or without reinforcement. Reinforced test panels are not normally required for temporary shotcrete construction facing.

Test panels are typically 0.5-to 1.0-m square. ACI 506.2 [49] permits 0.5 m square and 75 mm thickness. Panels should have sloped 45° sides to release rebound. Test panels should be shot using the vertical orientation of the face.

The test panel without reinforcement is used for compressive strength determination. Three cores, usually 75 mm diameter are required for a test. Panels are normally field cured until they are shipped to the testing laboratory. The outside 150 mm of the panel is not used for testing. Recommended acceptance criteria are given in table 7.1.

Panels with reinforcement are cored at 100-mm diameter; cores should intersect reinforcement overlaps. These are usually required only for permanent shotcrete. Cores and panel formed surfaces are examined for voids, rebound, rock pocket, lenses, and general structure. Core structure is compared against a set of five standard photographs to grade the core for shotcrete structure and reinforcement encapsulation. A minimum Core Grade No. 2 - is required for the reinforced panels to be considered acceptable.

The key to successful work is preparedness. However, pre-construction testing can be expensive and consideration can be given to the previous history of the Contractor and/or mix design and previous nozzleman certification as an alternate acceptance.

7.6.5 Quality Control During Production

7.6.5.1 Testing

ACI 506.2 [49] details testing procedures during production. These are straightforward and focus on compressive strength only. Often, this is adequate for soil nailing but the Owner may add additional requirements for such properties as those in table 7.1. ASTM C1140 [66] provides further guidance on shotcrete testing.

ACI 506.2 [49] provides the option of obtaining core test samples from panels or from the wall face. Test panels should be shot during production using the vertical orientation of the face.

ACI 506.2 [49] requires the panels to be moist cured until testing. Many agencies prefer to cure in conditions simulating the field, then soak test cores immediately before testing. Shotcrete tested wet will have lower compressive strength than dry; differences to 15 percent are reported.

The shotcrete facing guide specifications contained in Appendix C give suggested testing frequencies.

7.6.5.2 Inspection and Acceptance

If the contractor's quality assurance is functional, the owner's quality control role will be visual inspection accompanied by hammer sounding (a dull sound indicates freedom from delaminations and debonding) and some system for confirming thickness. Inspectors may satisfy themselves that thickness is adequate by observations during shooting assisted by wire lines or the use of "pogo stick" probes.

The fundamentals of shotcrete quality are given in 7.4.1. To ensure these are achieved, the inspector would observe:

- Uniform delivery of mix; minimize cold joints.
- Reasonable nozzle technique - particularly encapsulating mesh or rebar without build-up on their surfaces. It will be apparent if good in-situ compaction is being achieved. This will also result in some, but not excessive, rebound. For wet-mix, there may be a tendency to "lob" the mix onto the face with low air pressure. This is not good practice as compaction by impact is required.
- The nozzleman should be observed to be moving towards the face to encapsulate rebar.
- Removal of foreign material.

As a quality control checklist, the Inspector may wish to record some or all of the following:

- Nozzleman's name and certification details.
- Types and orientation of test panels prepared.
- Reinforcing in qualification panels (for permanent shotcrete)_a photograph is ideal. Confirm the same reinforcing layout is used on the face.
- Mix design, delivery tickets, delivery time, supplier.

- Equipment-compressor, pump, nozzle.
- Presence of and proper support for rebar and mesh. Eliminate localized congestion.
- Appearance of as-delivered mix segregation, oversize, slump.
- Installation of ground wires, forms, bulkheads.
- On-site test results-slump, air, temperature (if required by specification).
- Adequate surface preparation.
- Shotcreting nozzle work-pattern, distance, rebound, encapsulation.
- Operation of blow pipe (if required).
- Removal of sags and sloughs.
- Curing and protection used (if required).
- Surface finish and planeness (if required).
- Compliance with dimensional tolerances.

7.7 Shotcrete Application - Temporary Shotcrete Facings

Section 7.9 deals with additional requirements for quality permanent shotcrete. For temporary construction facings, a lower standard is usually allowed because of the short service life. As long as worker safety is not jeopardized, soil nailing is more cost effective if the quality standards for construction facings are commensurate with its function.

7.7.1 Finishing

As-shot or rough-screeded finish is satisfactory for temporary construction facings. An as-shot finish should be used where conditions permit. This provides economy and avoids the possibility of producing a weak surface from tearing or over-finishing.

7.7.2 Tolerances

Tolerances typically used for formed concrete cannot be easily achieved in shotcrete and they are not necessary for adequate performance of the structure. Tight tolerances will increase construction costs significantly. Similarly, tolerances for cover and mesh or bar location desired by designers for permanent facing are not essential for construction facings. Suggested tolerances for temporary shotcrete construction facings are given in table 7.6.

7.7.3 Joints

In temporary construction facings, no contraction joints or special construction joints are normally required. No significant strength loss is experienced if the face suffers some shrinkage cracks.

7.7.4 Protection and Curing

Ideally, the mix should be heated for cold weather shotcreting and cooled for hot. Heating of aggregates and mix water is practical but cooling is normally not. Basic temperature placement limits are:

- Ambient temperature limits 0 °C* to 40 °C
- Shotcrete mix temperature limits 10 °C to 35 °C

  *(providing that immediate protection from freezing is available)

As some surface cracking can be tolerated, protection from high winds and related drying is not critical. The evaporation limit can be as high as 2.0 kg/m/hr.

Curing is generally not required for temporary facings.

Construction facing shotcrete has been successfully placed below 0 °C with immediate cover. The key is planning protection. The recent ACI 506R [46] Guide to Shotcrete states:

> *When it is known or anticipated that shotcrete will be placed under cold weather conditions, a plan should be developed, submitted and approved to outline the*

procedures for subgrade preparation, shotcrete application, curing and protection of the shotcrete. Shotcrete can be placed successfully under very adverse conditions with proper planning and procedures designed to the specific application.

Hot weather protection is less important for construction facings where the short-term use can tolerate some cracking. However, in cold weather, there must be sufficient protection to achieve a minimum of 4 to 5 MPa in-situ compressive strength before the shotcrete is frozen.

7.8 Potential Problems With Construction Facing Shotcrete In Soil Nailing Applications

7.8.1 General Durability

Unless the duration of construction facing exposure is longer than 36 months, long-term durability of the construction facing is not generally a concern and air entrainment is not required. Factors affecting the long-term durability of shotcrete are generally the same as those for concrete. Details are discussed in 7.9.3.

7.8.2 Potential Problems With the Shotcrete

Table 7.3 describes potential problems for construction facing shotcrete, together with advice on how to avoid such problems.

TABLE 7.3
POTENTIAL CONSTRUCTION FACING SHOTCRETE DEFECTS

Defect	Cause	Results In	How To Avoid
Lack of bond to soil	Presence of overspray. Loose soil. Excess groundwater seepage.	Cannot develop integral action between soil and shotcrete.	Remove loose soil. Place flashcoat. Control groundwater
Rebound trapped	Rebound not allowed to escape. Poor nozzle technique.	Localized weak and porous zone.	Improve nozzle technique.
Slough/Sag	Slump too high. Lift too thick.	Horizontal cracks. Voids under rebar.	Improve Nozzle technique. Add silica fume. Add accelerator. Reduce water content at batching. Reduce lift thickness.
Blow-off	Water build-up behind shotcrete.	Destruction of shotcrete.	Proper surface and sub-surface drainage.
Slough	Loose wire mesh or loose drain strip.	Voids beneath mesh. Shotcrete cracking.	Provide additional support for mesh. Pin drain strip to face.
Deficient shotcrete thickness	Inadequate quality control during shotcrete placement.	Weakened construction facing.	Improve shotcrete placement quality control.

7.9 Additional Aspects Of Shotcreting For Permanent Shotcrete Facing

7.9.1 General

The majority of the previous text has focused on shotcrete for construction facing because it is in this construction that the majority of soil nailing shotcrete is used. However, permanent shotcrete facings are gaining use in some parts of the country as an alternate to cast-in-place or precast cladding.

The fundamentals of materials, shotcreting equipment and practices, and quality assurance are common to both permanent and temporary construction. Because of the need for long term structural integrity and service life, permanent shotcrete has some additional requirements described here.

7.9.2 Comparison of Shotcrete and Conventional CIP Concrete

Differences between shotcrete and cast concrete arise from two features of shotcrete:

- In shotcrete mixture, proportions are heavy to the mortar phase due to high sand and cement contents (i.e. low coarse aggregate); these mortar rich proportions are necessary to minimize rebound and accommodate transport through the small diameter shotcrete hoses.

- The method of shotcrete placement uses compressed air driven velocity to achieve compaction with no supplementary vibration; therefore, variable localized porosity and higher entrapped air result.

With suitable shotcrete mix designs and well-qualified structural shotcreters, it is now possible to produce high performance shotcrete equivalent to high quality concrete. Table 7.4 compares the properties of shotcrete and cast-in-place concrete.

7.9.3 Durability and Defects

For a permanent soil nail wall requiring long service life, permanent shotcrete durability aspects to be considered are:

- Freezing and thawing. ACI Committee 362 "Guide for the Design of Durable Parking Structures" [67] has defined a range of exposure conditions for the various areas and elevations of the United States. Basically all areas north of the Utah-New Mexico border have demanding exposure. Normal protection consists of air entrainment and a good quality paste (water:cement ratio <0.50).

TABLE 7.4
COMPARISON OF SHOTCRETE AND CIP CONCRETE

Property	Comparison
Shrinkage (1)	Shotcrete is higher, sometimes much higher, due to low coarse aggregate content and higher cement factor.
Propensity to crack	Higher in shotcrete due to (1) and thin sections with higher surface area.
Creep	Higher in shotcrete due to (1). This can be advantageous because it relieves and redistributes local stresses.
Bond	Similar but can be higher in shotcrete due to impact of shotcrete on surface. There is also high rebound of the coarse aggregate sizes that initially strike a surface leaving behind a paste-rich bond layer. In both cases, bonds depends on preparation of the sub-strata surface.
Compressive strength	A range of strengths to 70 MPa are reported for shotcrete. The high cement content (and low water:cement ratio) of shotcrete results in generally high strengths.
Flexural strength	Higher in shotcrete.
Modulus of elasticity	Generally related to the square root of compressive strength, similar to cast concrete.
Porosity and permeability	Higher in shotcrete, particularly on localized basis due to compaction voids. Typical absorption values-see table 7.1-of good quality concrete, with a water:cement ratio of 0.45, will be in the 5 to 6% range; for a comparable shotcrete mix, they might be 6 to 8%. Permeability values for good quality shotcrete in the 10^{-12} m/sec range have been obtained. The permeability is markedly decreased when silica fume is added.
In-place density	Lower in shotcrete due to both lower coarse aggregate content and higher porosity. For a given source of aggregate, shotcrete density will be typically 25 to 50 kg/m^3 lower than cast concrete.

- Leaching of water-soluble portions of the hydrated cement paste (mainly lime) in areas of porosity or cracking. This is primarily an aesthetic consideration that only affects durability in advanced stages.

- Attack of the cement paste by aggressive chemicals (typically sulfates) in groundwater. Protection can be provided by selection of proper cements (Type II or V).

Of the above, freezing and thawing is the most significant. Distress manifests itself as progressive surface scaling. For distress to occur, the shotcrete must be saturated with water and therefore problems will occur only where there is free water on the shotcrete surface or groundwater behind the facing. Chlorides from deicing salts will greatly accelerate the process by increasing the number of freezing cycles and the pressure within the paste pores. As discussed previously, air can be entrained into wet-mix shotcrete to provide long-term durability against freeze-thaw.

Soil nailing shotcrete seldom suffers from freezing and thawing; because of vertical orientation, it is rarely saturated. In addition, construction facing shotcrete is partially protected by the permanent facing.

Table 7.5 presents a description of potential shotcrete defects for permanent shotcrete facings, in addition to those listed in table 7.3 for construction facings. Permanent shotcreting requires the use of a blowpipe in concert with the nozzle to remove rebound and overspray.

TABLE 7.5
POTENTIAL PERMANENT SHOTCRETE FACING DEFECTS

Defect	Cause	Results In	How To Avoid
Overspray and rebound	Shotcrete stream impacts on surface and fines travel to adjacent surfaces.	Localized weak, porous zones. Poor bond to rebar or surface.	Clean adjacent surfaces. Use blow pipe to remove.
Lack of bond to previous shotcrete or concrete.	Foreign material on surface - dirt, laitance or overspray.	Delamination. No integral action between two shotcrete layers.	Mechanically roughen fresh shotcrete surface. Sandblasting may be required to remove laitance. Hydromilling at pressures >30 MPa will normally remove foreign material. As-shot or rough screed finish is normally sufficiently sound and rough to provide excellent bond to subsequent shotcrete or cast concrete.
Rebar shadowing	Improper nozzle techniques. Rebar too large or too congested.	Void or porous zone behind rebar.	Improve nozzle technique, normally move in closer. Use blow pipe to remove rebound.

7.9.4 Materials

Permanent facing shotcrete might include these additional materials:

Silica Fume

This finely ground silica is becoming more commonly used in shotcrete where its benefits include reduced rebound, increased strength, decreased permeability, and greater lift thicknesses without sloughing. It is commonly used at dosages of 8 to 12 percent by weight of cement. It markedly increases the cohesiveness of the mix. Silica fume reduces the plastic viscosity but can greatly increase the flow resistance (i.e. shootability) at high dosages. For this reason, it is sometimes proposed by shotcrete Contractors to improve one-pass production, particularly for thicker applications. Other advantages include reduced wash-out of surface fines for shotcreting in the rain and bond to wet ground.

Bonding Compounds

Bonding compounds are not normally required between layers of shotcrete. The bond of new shotcrete to properly prepared soil, rock, old concrete, or shotcrete surfaces is normally excellent. The initial mix that contacts a hard surface has high aggregate rebound leaving a paste-rich layer against the surface, which enhances bond.

7.9.5 Shotcrete Application-Permanent Shotcrete Facings

7.9.5.1 Protection and Curing

Protection practices for concrete are generally applicable to permanent shotcrete. However, the relatively thin shotcrete section and associated large surface area makes shotcrete more sensitive than CIP concrete to cooling and surface drying. In addition, shotcrete does not have the benefit of in-form curing.

In all weather, but particularly in hot weather, protection from high winds is essential to good workmanship. In hot weather, the rate of surface evaporation should not be allowed to exceed 1.0 kg/m/hr (see ACI 305R [68] for method of calculation). Protection from surface drying is even more critical for shotcrete containing silica fume because of its susceptibility to plastic shrinkage.

Ideally, the mix should be heated for cold weather shotcreting and cooled for hot. Heating of aggregates and mix water is practical but cooling is normally not. Basic temperature placement limits are:

- ambient temperature limits 5 to 35°C
- shotcrete mix temperature limits 10 to 30°C

In some locations, this greatly limits the time of year for shotcreting. Such limits can be extended by prudent construction such as immediate application of protection covers to retain heat or prevent drying, and/or immediate application of the curing system.
Curing systems can include the usual curing compounds, polyethylene sheet drapes (with sealed edges) and water spray or flooding. Curing compound should not be used on surfaces to receive subsequent layers. If the surface is left as-shot, the rate of compound coverage must be increased to perhaps double the normal value because of the increased surface area. Ideally, curing compound should be applied in two stages-immediately after shooting and the next day.

Fog spray in the area of nozzling has been found to be beneficial in hot weather.

Shotcrete can be placed in drizzling rain up to the stage where the surface paste is washed off. Additions of accelerators or silica fume will greatly reduce susceptibility to wash-off.

It is generally desirable to shoot with a mix as fresh as possible, particularly in hot weather. The usual 90-minute limit from batching to shooting should be reduced to about:

- 60 minutes for ambient temperature above 30°C,
- 45 minutes for ambient temperature above 35°C.

Alternatively, retarders can be added. The converse is true for cold weather where longer times may be allowable.

As a principle, batch to batch delivery of the shotcrete at the desired slump and uniformity is essential to shotcrete quality and productivity. A mix that becomes difficult to pump should be discarded.

Inspection should assure that curing and protection are in place. Inspection after shotcreting by wetting the face and allowing partial drying will locate cracks and localized porosity.

7.9.5.2 Shooting Reinforcement in Permanent Facings

In addition to the information presented in section 7.5.3, the following applies to permanent shotcrete facings.

The ACI 506R [46] Guide suggests that it is difficult to avoid voids or shadowing behind rebar if bar(s) are larger than No. 16, if there are two layers, or if bar spacing is closer than about 100 mm. It would be difficult to use shotcrete in soil nailing with these restraints. Quality shotcrete has been demonstrated with bars up to No. 25, at 75 mm spacing, with laps and with two layers in a total 300 mm wall thickness, using an appropriate mix design and a highly experienced nozzleman. The 506R [46] suggestions are conservative for a competent contractor.

When shooting through and encasing reinforcing bars, the nozzle should be held closer than normal and at a slight upward angle to permit better encasement of horizontal steel and minimize the accumulation of rebound. Also, the mix should be a little wetter than normal, but not so wet that material will slough behind the bar. This procedure forces the plastic shotcrete behind the bar while preventing build-up on the front face of the bar.

Guidance on in-place evaluation of permanent shotcrete is given in ACI 506.4R [69]. If the inspector wishes to check rebar encapsulation in situ, sacrificial bars can be installed to avoid cutting the primary rebar.

7.9.5.3 Surface Preparation

Proper surface preparation is essential to achieve bond to the surface receiving the shotcrete. The basic parameters desired are lack of absorption of water from the mix, cleanliness, and sufficient roughness to achieve a mechanical as well as cement hydration (chemical) bond. To maximize bond, the receiving surface should be damp with no free water.

7.9.5.4 Finishing

For aesthetic reasons, permanent facings may require other than an as-shot finish. Finishing must commence immediately after shooting and consists of cutting and screening to the alignment wires or forms followed by wood, steel or rubber floating as specified. If a high quality permanent face finish is required, the contractor may shoot a thin (flash) coat of sand cement over previously placed as-shot shotcrete to facilitate finishing. Flash coat application should occur before the base coat has set. Finished shotcrete surfaces can also be colored with colored pigmented sealers.

7.9.5.5 Tolerances

Section 7.7.2 notes the problem with overly restrictive shotcrete tolerances.

ACI 506.2 [49] now states: "Unless specified elsewhere, tolerances of shotcrete shall comply with ACI 117 [70] (for normal cast concrete) as modified herein. Increase tolerance for concrete dimensions for gun finished shotcrete by factor of 2. Increase tolerance for concrete dimensions for all other shotcrete by factor of 1.5, which recognizes the need for more relaxed tolerances. Suggested tolerances for both temporary and permanent shotcrete facings, largely drawn from ACI 506 Committee discussions, are presented in table 7.6.

The two key issues for permanent shotcrete are planeness for aesthetic reasons and cover on rebar and nails for corrosion protection. Finished shotcrete surfaces will always have elements of waviness. Rebar cover can readily be checked in the field by probing.

7.9.5.6 Joints

Square construction joints are not desirable in shotcrete construction because they form a trap for rebound and a route for water infiltration. The entire joint should be thoroughly cleaned and wetted prior to the application of additional shotcrete. Where a section of shotcrete is left incomplete at the end of a shift, some provision must be made to ensure the joint will not develop a plane of weakness at this point. The joint should be tapered to an edge or shoulder, usually about one-half the thickness of the shotcrete or a minimum of 25 mm.

The spacing of contraction joints in permanent shotcrete facings depends on the application and should be designated on the drawings. In practice, the spacing usually varies from 5 to 10 m. Contraction joints are intended to prevent random cracking. For reasons explained in table 7.4, shotcrete is susceptible to shrinkage, which can best be mitigated by curing procedures.

TABLE 7.6
SUGGESTED CONSTRUCTION TOLERANCES FOR SOIL NAIL SHOTCRETE FACING

Item or Element Tolerance from Specified Dimension	Type of Soil Nail Wall	
	Permanent Facing	Construction Facing
Horizontal Location of Wire Mesh, Rebar and Studs on Plates	±10	±15
Thickness of shotcrete		
if trowelled, or screeded	-15	-10
if left as-shot	-30	-10
Planeness of surface, gap under 3 m straightedge, any direction		
if trowelled, or screeded	15	N/A
if left as-shot	30	N/A
Spacing between bars	25	25

Notes:

- All dimensions in mm
- "+" indicates increase in dimension, "-" indicates decrease
- N/A = not applicable or controlled by other tolerance requirements
- Permanent facing refers to full-thickness permanent shotcrete facing
- Construction facing refers to shotcrete construction facing which will be covered by a CIP final facing
- Tolerances for a temporary wall with construction facing only, i.e. will not be covered by a CIP facing, can have less restrictive tolerances. These may vary from project to project depending on application, site geometry and other constraints and thus are left to discretion of individual Owners/Designers to establish.
- Tolerances and finishing requirements for CIP facings refer to highway agency standard specifications or project special provisions.

Expansion joints similar to those used in conventional CIP facings may be required in long permanent shotcrete facings, normally those over 30 m. Such joints are normally formed with asphalt impregnated fiberboard or similar compressible material and are approximately 20 mm in width.

7.9.5.7 In Situ Density and Permeability

For some shotcrete applications, high in-situ density or low permeability may be important. Such properties reduce through-face weeping and improve corrosion protection of the nail head steel. A simple method of specifying such performance is the absorption test. Table 7.1 contains recommendations.

7.10 Steel Fiber Reinforced Shotcrete (SFRSC)

Steel fibers been used as reinforcement in shotcrete used for temporary construction facings for the purpose of:

- Replacing welded wire mesh where mesh installation is difficult, e.g., irregular or rough excavation face surfaces; or where soil stand-up time is limited and a quick shotcrete application ("flashcoat) is required.

- Providing post-cracking residual strength when the shotcrete cracks due to shrinkage or localized over stressing.

- Increasing the flexural strength of the mix. Increases in compressive strength are nominal.

Fibers for shotcrete are steel or polypropylene. At this time, steel is preferred for better post-crack residual strength and resistance to shrinkage cracking. Polypropylene may perform adequately at higher dosages, say 4 or 5 kg/m^3, but these dosages can create mix workability, mixing and pumping problems. Shotcrete contractors also state that steel fibers in shotcrete produce increased wear on equipment, particularly on hoses. The swing tube in wet-mix pumps may also suffer increased wear.

It should be noted that the shotcrete placing process tends to force the fibers to an in-plane orientation rather than the random orientation achieved in normal concrete. This improves their effectiveness in flexure. Fibers are not generally used in combination with mesh because they have a tendency to drape over the mesh wires during shooting and become poorly distributed. The function of the fibers is to replace the wire mesh. As a reinforcement, their primary function is crack control, specifically forcing and controlling crack distribution and preventing large crack openings.

Steel fibers are more efficient at longer lengths and higher aspect ratios. The aspect ratio of fibers is the length of fiber divided by the fiber's effective diameter. Research shows an ideal length to be about 40 mm to develop maximum fiber-paste bond, but such long lengths are often impractical for pumping and shooting in wet-mix shotcrete. Lengths as low as 20 mm can be used; 25 mm is common. The delivery hose diameter should be a minimum of twice the fiber length. The aspect ratio should be greater than 60 and ideally in the 100 range. This means that a typical fiber width of 0.5 mm diameter would have to be a minimum of 30 mm long. For aspect ratios over 100, problems with workability can be experienced; under 50 provides little improvement.

Fiber dosages should be a minimum of 50 kg/m^3 and ideally in the 70 kg/m^3 range. A steel fiber dosage of about 60 kg/m^3 is approximately equal to 0.75% by volume of reinforcement in the mix. In a flexural test, typical failures are by pull-out of the fibers in bond. Fibers with hooked ends or deformed surfaces generally are more effective due to improved bond.

ASTM C1018 [71] provides a test procedure for evaluating the effectiveness of fibers. It determines the maximum flexural strength and the residual (post cracking) strength by quantifying the load deflection curve. The Appendix to C1018 provides guidance in interpreting results of these tests.

ASTM C1116 [50] presents a "Standard Specification for Fiber-Reinforced Concrete and Shotcrete". A common specification would require:

- Fibers shall comply with ASTM A820 [72] Types I, II or III with a minimum length of 25 mm and a minimum aspect ratio of 60.

- The minimum fiber dosage shall be 60 kg/m^3.

- Beams cut from test panels shall be conditioned in the same manner as compressive strength cores, tested in accordance with ASTM C1018 [71], and have the following minimum strength properties:

- 7-day and 28- day first crack flexural strength of 4 MPa and 5 MPa, respectively.

- Residual post cracking strength, $R_{10, 30} = 60$ at 7 and 28 days.

ACI 544.4R [73], "Design Considerations for Steel Fiber Reinforced Concrete", contains guidelines on design procedures with steel fibers. At the time of preparation of this manual, structural design procedures for designing with steel fibers are not as well developed as for conventional concrete reinforced with welded wire mesh or deformed reinforcing bars. Use of steel fiber reinforced shotcrete is increasing for applications such as tunnel linings and slope stabilization/protection and there has been some limited usage to date as temporary soil nail wall facings.

CHAPTER 8. CONTRACTING METHODS, PLANS, AND SPECIFICATIONS

Soil nailing is still regarded by some as an emerging technology even though extensive test programs have been conducted on full-scale soil nail walls and soil nailing has been successfully demonstrated in both Europe and the United States to provide adequate temporary and permanent in-situ reinforcement. Economy, reliable design methodologies, appropriate construction techniques, and confidence in performance of the structures have all contributed to the popularity of soil nailing in Europe and in localized regions of the United States. When introducing soil nailing projects, public agencies worldwide have ensured the quality of the final wall by structuring the contract documents to build trust and a positive contractual relationship by prequalifying contractors and equitably distributing the risk.

This section of the manual is intended to assist public agencies with little or no experience in procuring contracts where soil nail walls are the preferred construction method. Although the contents of this manual reflect the collective experience of the FHWA, State, and Industry peer reviewers, soil nailing contracting methods should be formulated to take advantage of new practices that prove to be both economical and durable. Since contractors often introduce innovative, cost-competitive solutions, it is recommended that the contract documents for soil nail projects be structured to allow specialty contractors to make use of the latest available design methodologies and construction techniques. Thus, contract documents that are performance based rather than procedural are preferred. Current contracting procedures and guideline information for owners and agencies to successfully procure soil nail wall contracts are summarized in this chapter.

8.1 Contractor Prequalification

The likelihood of obtaining a satisfactory installation improves when a qualified contractor experienced with the construction sequence and procedure is selected to build the wall. As with any new technology, inexperienced contractors unfamiliar with the proven construction methods for soil nail walls can jeopardize the future use of the technology. This aspect becomes critical if serious construction related problems develop, particularly if such problems occur on an agency's initial application. In such cases, there will be a tendency for administrators, designers of alternate support systems, and representatives of private owners to unduly question established design methodologies on future projects where soil nailing is the preferred economical or environmentally correct solution. Therefore, the responsibility for safeguarding soil nailing from an unwarranted reputation is incumbent upon all future users and contracting parties. The news of one poorly executed project reaches farther than ten successful ones.

For agencies or owners with no previous experience with soil nailing, the quality assurance issue is further complicated since the knowledge of the in-house design/inspection staff is limited, at least initially, due to the lack of experience or familiarity with the new technology. A proven method of helping to ensure success of the technology is to clearly define the contractor prequalification requirements in the contract documents followed by enforcement of these requirements. Such prequalification requirements are commonly being used by many transportation agencies for other specialty construction techniques, such as permanent tieback walls, drilled shaft foundations, shotcreting, etc. There may be a tendency to accept bids from contractors experienced with other ground reinforcement techniques such as rock bolting and tieback walls. For instance, it should be recognized that soil nail wall construction differs significantly from tieback wall construction since no vertical ground support members are installed. Soil nailing has a uniquely defined set of construction procedures which, when not observed by inexperienced contractors, can produce highly undesirable construction conditions that could jeopardize the structure and be hazardous to workers. To ensure a safe working environment, minimize construction problems, and provide a quality product, it is recommended that only bids from contractors experienced with soil nail wall construction be allowed. Typical prequalification requirements are included in the appendix B, Guide Specification.

8.2 Contracting Methods and Definitions

The general types of contracting methods currently being used for soil nail wall design and construction may be generally classified as Owner Design and Contractor Design/Build. Owner Design Contracts may be further structured using an Owner Design - Performance or an Owner Design - Procedural/Prescriptive based specification. Design/build contracting methods are performance based and place the responsibility of both design and construction on the contractor Each of these contracting methods assigns different requirements and responsibilities to the owner and the contractor. The owner's ability to select the wall type, define the work required to construct the wall, available in-house design expertise, as well as the human resource requirements demanded by the project will often dictate the contracting method selected.

The procedural/prescriptive based contracting method is generally not advantageous since the owner is fully responsible for the design and performance of the soil nail system and is responsible for directing the contractor's work if changes are required (i.e., the owner assumes all the risk).

For purpose of the discussions which follow related to the alternate contracting methods, the following definitions will be used for the various types of contract specifications that can be used:

Owner Design - Performance Specification

Nail final drillhole diameter and installation method required to provide the design nail pullout resistance is the contractor's responsibility.

Owner Design - Procedural/Prescriptive Specification

Nail drillhole diameter and installation method is specified by owner.

Design/Build - Performance Specification

Implicit - Owner determines that a soil nailing wall is feasible and specifies a soil nailing wall. The contractor prepares the design calculations and detailed plans and constructs the wall.

Open - Owner specifies that a wall be built. The contractor selects the wall type, prepares the design calculations and detailed plans, and constructs the wall.

8.2.1 Contract Procurement with Owner-Design

Designs prepared by the owner afford more control over the final product and allow the owner to gain valuable in-house expertise. Once this expertise is gained the owner can more comfortably meet demanding project schedules that otherwise would have required engaging a consultant or letting a design/build contract. In-house expertise also provides a more reliable basis for reviewing designs prepared by contractors or their consultants. Where project schedules burden the resources of the owner but the agency stills prefers more direct involvement in the design, the agency may engage consultants familiar with soil nail wall design and construction or use a pre-bid specialty contractor design contract.

8.3 Plan Preparation

The level of effort required by the owner to prepare plans will vary substantially depending on the chosen contract procurement method and the associated owner/contractor division of responsibilities. The most fundamental effort in plan preparation for an agency would be to prepare "conceptual plans" for a design/build contract. Increasingly higher levels of owner effort are required for performance and procedural plans/specifications prepared for owner designs.

8.3.1 Conceptual Plans for Design/Build Contracting

Conceptual plans furnished by the owner for design/build contracts are recommended to contain, at a minimum, the following geometric, structural, geotechnical, and design information.

A. Geometric Requirements

- Wall plan and profile.
- Beginning and end of wall stations.
- Wall alignment topographic survey.
- Existing and finish grade profiles both behind and in front of the wall.
- Cross sections showing the limits of construction at the retaining wall location intervals of 15 m or closer.
- Horizontal and vertical curve data and wall control points.
- Required wall appurtenances such as barriers, coping, drainage, etc.
- Right-of-way and permanent or temporary construction easement limits, location of all active and abandoned existing utilities, adjacent structures or other potential interferences.
- Staged excavation sequencing.
- Quantity tables showing estimated square footage of wall areas and other pay items.

B. Structural Requirements

- Conceptual details for wall reinforcement and load transfer devices between nail and facing wall or cast-in-place wall.
- Level of corrosion protection required (none, fusion bonded epoxy or full encapsulation).

excavation lifts.

- Specifications requirements for materials, material certifications, and construction quality control tests.
- Corrosion protection details.
- Type of finish facing required, including dimensions and reinforcing steel requirements.
- Maximum time duration of finish cut face exposure prior to nail installation and closure with structural shotcrete.
- Minimum required nail grout and shotcrete strengths prior to allowing excavation to proceed to the next lift.
- Wall drainage requirements.
- Nail testing procedure(s) and acceptance criteria.
- Wall construction monitoring responsibilities and requirements.
- Methods of measurement and payment.

The contractor's responsibilities include:

- Fulfilling the contract submittal requirements.
- Selecting the soil nail installation methods and equipment and final drillhole diameter.
- Complying with material specifications, construction tolerances, and minimum/maximum dimensions.
- Obtaining and verifying the soil nail load carrying capacity and pullout resistance values used in design.
- Completing construction excavations in accordance with the specifications.
- Installing wall finish facings in accordance with the contract documents.
- Performing the required soil nail testing.

The owner's responsibilities during construction include:

- Verifying that construction tolerances, construction sequencing, and minimum soil nail requirements have been satisfied.
- Verifying that drilling procedures are not causing excessive ground loss or subsidence.
- Verifying compliance with the specified material properties and requirements.
- Verifying that corrosion protection requirements have been satisfied.
- Verifying that construction excavations are staged in accordance with the specifications.
- Verifying that finish facings are constructed in accordance with the contract documents.
- Observing, verifying, and recording the results of all soil nail and construction quality control testing.

Provided below is a general listing of the owners advantages and disadvantages when contracting soil nail wall construction using an owner prepared design and a performance based construction specification.

OWNER DESIGN - PERFORMANCE SPECIFICATION

Advantages	Disadvantages
Valuable in-house expertise is obtained	Requires owner staff and expertise
Owner has control over final product	Requires sufficient staff to support project
Equitable risk is shared between the owner and the contractor	Requires assumptions regarding the contractor's construction procedures and equipment
Contractor's experience, equipment, and expertise is utilized	Less economical if the design does not optimize the contractor's procedures and equipment

8.2.1.2 Contracts with Owner Design and Procedural/Prescriptive Specifications

When procedural ("prescriptive") construction specifications are used, the agency or owner is fully responsible for not only the design but also the construction performance of the soil nail wall system, provided the contractor has installed the nails in accordance with the prescriptive specification requirements. A procedural contract specifies all details of design and construction, including the required soil nail drillhole diameter. The agency must be confident in its ability to predict the performance of the contractor's procedures and equipment to preclude claims or disputes from arising during construction.

Procedural contracts typically do not ensure higher quality work but do open the bidding field to contractors inexperienced with soil nailing. Inherently, the owner is responsible for directing the contractors activities if changes are required and, therefore, is exposed to a higher likelihood of contractor requests for additional compensation. For example, if the soil nail load carrying capacity (i.e., pullout resistance values used in the design) are not achieved and the owner has specified the drillhole diameter and installation methods, it then becomes the owner's responsibility to direct the contractor with respect to what changes must be made. All resulting extra costs due to the changes and project delay/impact are borne by the owner. Moreover, the owner is responsible for the construction performance that can result in extensive contractor claims if the owner's inspection staff is unfamiliar with the contractor's construction procedures and/or equipment. The responsibility of the contractor is to submit material certifications and build the soil nail system in strict accordance with the plans and specifications.

Provided below is a general listing of the owner's advantages and disadvantages when contracting soil nail wall construction using an owner prepared design and a procedural/prescriptive based construction specification.

OWNER DESIGN - PROCEDURAL /PRESCRIPTIVE SPECIFICATION

Advantages	Disadvantages
Widens the bidding field	Owner assumes all risks
	Owner is fully responsible for the design and performance of the system
	Owner directs the contractor's work when changes are required
	Owner must be highly confident in predicting the contractors performance
	The owner must have highly qualified and experienced design and inspection staff
	Unqualified contractors may be awarded the contract
	Potential for claims and cost overruns is high

The procedural ("prescriptive") contracting method is not generally advantageous for soil nail wall construction and is not recommended.

8.2.2 Contractor Design/Build ("Turnkey") Contracting Methods

With the Owner-Design contracting method employing soil nailing on a project, the owner's engineer defines the scope of work and the owner shares responsibility in the design and installation of the soil nail system. With the Design/Build method, the owner outlines the projects ultimate needs and the specialty contractor is responsible for the detailed soil nail system design and installation. Ideally, the owner's engineer defines the performance criteria and objectives. Based upon specified requirements and limitations, a design/build proposal is submitted, either before the bid advertisement (pre-bid), or after contract award (post-bid). Measurement and payment is often performed on a lump sum basis.

Design/build specifications may be advantageous when:

- The wall will be constructed in ground where the scope of the various work items is difficult to establish;
- difficult ground conditions may require modifications in the construction methods; or
- the agency has adequate hands-on-experience with soil nail design and construction.

In bouldery soils, residual soils, and weathered rock, soil nailing is often less expensive than a ground anchor wall. However, it is difficult to use conventional contracting methods in these grounds because it is difficult to prepare specifications that adequately define the scope of work or assure adequate coordination of the excavation, shotcrete, and finish concrete work required to build a soil nail wall. Using a design/build specification in these instances, enables qualified contractors to define and coordinate the work so a satisfactory wall can be built. With a design/build contract, field modifications, which occur regularly when working in difficult ground, can be made without seriously impacting the progress of the work. When an owner with limited soil nailing experience wants to construct a wall, they can solicit pre-bid designs from prequalified contractors. Then the contractor and the owner will agree on the details of the design prior to bid. The owner gains the benefit of the contractor's experience while maintaining control over the design.

Design/build contracts require the contractor to both design and build the soil nail system to meet the performance requirements specified by the owner. The owner usually selects the wall type and is generally responsible for obtaining the subsurface information, characterizing the ground engineering properties, providing the bidders with a design/build specification, and providing construction inspection. The contractor is responsible for preparing a detailed design that meets the design requirements set forth in the contract documents and installing the soil nail wall to satisfy the performance requirements. In Europe, for example, soil nailing has grown fastest in the countries (e.g., France and Germany) where contractors often select where soil nailing will be used and prepare the detailed working drawings and construct the wall. While not the contracting method currently in predominant use by highway agencies in the United States, its use is increasing as more emphasis is being placed on use of innovative contracting methods. This arrangement may be beneficial in difficult ground or where agencies or owners lack or cannot develop in-house expertise in a timely manner to support the project schedule. The contractor's design responsibilities are met either through in-house expertise or through engaging a design consulting engineer. Design/build contracts provide incentive for the contractor to innovate as well as allow the contractor to tailor the design to his equipment.

The owner can obtain a quality product at a competitive price provided that the contractor prequalification requirements are clearly defined in the contract documents and are enforced during procurement and adequate construction inspection is provided by the owner. However, owners should be willing to forfeit a certain degree of control in the design process since this has been made the contractor's responsibility. This could result in design features with which owners are unfamiliar or uncomfortable. It is customary in some countries to require a design/build project to be warranted for a specified period after construction. In the United States, however, insurance and bonding companies are reluctant to provide extended warrantees. The owner can prequalify contractors or contractor/engineer teams and maintain control over the design by requiring presubmission of detailed designs prior to advertisement.

"Design/Build" is in effect a generic term. There are several variations of Design/Build alternative contracting approaches that are being used to procure alternate retaining wall systems and which can be used to procure soil nail walls. These can be summarized briefly as follows:

1. Alternate Wall, Pre-bid Design

   The owner contacts qualified specialty wall contractors and informs them that they are planning to build a wall (e.g. soil nail wall) at a particular site. They request that the specialty contractors, interested in bidding on the walls, prepare a complete set of working drawings for the walls during the design phase of the work. The owner furnishes the geotechnical information and establishes the design requirements. Performance criteria and necessary project design information in this option are usually made available at least 90 to 120 days prior to contract advertisement date. The specialty contractors prepared detailed design calculations and the working drawings and the state's consultant or the state review them. The accepted designs are included in the bid documents and the owner agrees to require the prime contractor to use the specialty contractor that prepared the preapproved design to construct the wall. The prime contractor is free to select the best design. The prime contractor lists the selected specialty contractor in the bid submission. The owner and the contractor agreed on the details of the work before bid. Claims are reduced since the contractor knows that his design is acceptable. The wall work can start immediately since the working drawings have been pre-approved.

2. Alternate Wall, Post-bid Design

Contract documents for the post-bid method are prepared to allow for various prequalified contractor designed wall alternates. In the bid documents each wall is identified and acceptable alternative wall types are identified. The design requirements for the alternative wall types are established in the project special provisions or standard agency specifications. The prime contractor decides which type of wall he will build and he selects a specialty wall contractor or preapproved wall system vendor to prepare working drawings for the alternate walls he selects. The detailed design calculations and working drawings are then submitted to the owner for review and approval. After the working drawings are approved, the prime contractor and/or the specialty wall contractor construct the walls in accordance with the approved working drawings.

3. Alternate Wall, Typical Section Design

The owner selects which wall(s) they will build as a particular wall type and notifies the specialty wall contractors that they are preparing to contract for work that includes that wall type. Contractors interested in the wall work, are required to prepare typical working drawing details for a selected section(s) of the wall. The contractors and the owner agree on the design and working drawing details during the design phase of the work. The contract documents include the typical working drawing details, and identify the specialty contractors approved to construct the walls. At bid time, the prime contractor selects a specialty wall contractor and bids the work. When the work is awarded, the specialty contractor completes the final detailed working drawings using the preapproved designs and details. This method allows the specialty contractors' pre-bid design costs to be low. Only the successful bidder is required to prepare a complete set of final working drawings.

4. Alternate Wall, Contractor Selects Wall Type

On selected projects, owners have contacted specialty contractors and identified wall locations where they or their consultants want the specialty contractor to select the type of wall to be built and prepare the detail working drawings for the walls during the design phase. The owner establishes the design requirements and the finish for the wall, and establishes the location of the wall. The contractor selects the type of wall, i.e., soil nailing, tieback, cantilevered, etc., and prepares the detailed working drawings for review by the state or their consultant. The accepted designs are included in the bid documents and the prime contractor selects the design and the specialty contractor. This method allows the most economical wall alternate to be selected for the project and enables the contractor to know that his wall type and details are acceptable before bid. The owner or his consultant maintains control over the design while allowing the specialty contractors to use their experience, proprietary techniques, and patents.

5. Value Engineering

   Most U.S. transportation agencies now use Value Engineering clauses in their contract documents. After project award, the contractor can submit proposed alternate designs to substitute for all or elements of the wall system design which was included in the plans and was bid. Value Engineering is in effect a version of Design/Build. Some agencies refer to value engineering submittals as cost reduction incentive proposals. Typically, if the contractor's proposed value engineering alternate design is accepted, the accrued cost savings are shared equally between contractor and owner.

When the specification requires the contractor to prepare a detailed design and construct the wall the contract documents should include:

- Results of the geotechnical investigation.
- Site Limitations (i.e., right-of-way and easement limits).
- Existing utility drawings.
- Ancillary structures design and details.
- Design criteria, safety factors, and material specification requirements.
- Level of corrosion protection required.
- Finished wall face requirements.
- Construction tolerances for wall alignment and drillhole location/inclination.
- Percentage of nails to be tested, testing procedures, and acceptance criteria.
- Wall construction monitoring requirements.
- Methods of measurement and payment.

The contractor's responsibilities include:

- Designing the wall and provide detailed design plans and design calculations.
- Constructing the wall in accord with the approved final plans and construction specifications.
- Complying with material specifications and wall tolerances.
- Obtaining and verifying the soil nail load carrying capacity and pullout resistance values used in design.
- Installing the soil nails and wall facing in accordance with the finished wall face requirements and construction tolerances.
- Performing production soil nail testing.
- Performing any redesign, propose alternative methods and install additional test nails if the design criteria is not met.

The owner's responsibilities during construction include:

- Reviewing the contractor prepared detailed plans and designing calculations to verify that the design satisfies the project requirements, design criteria, and minimum agency design standards.

- Reviewing required contractor submittals.
- Verifying compliance with the specified material requirements.
- Verifying that corrosion protection requirements have been satisfied.
- Verifying that the wall finish face requirements are met.
- Verifying that wall construction tolerance requirements have been satisfied.
- Observing, verifying, and recording the results of all soil nail tests.

Provided below is a general listing of the owners advantages and disadvantages when contracting soil nail wall construction using contractor design/build contracting methods.

CONTRACTOR DESIGN/BUILD

Advantages	Disadvantages
Cost effective	Owner assumes maintenance responsibilities if the structure is not warranted
May be advantageous when very difficult ground is expected	Owner has less control over the design unless pre-bid design is used.
Does not require large owner staff	Potential for undesirable or unfamiliar design features to be incorporated into the design
Requires less in-house expertise than required for owner design	Owner must still provide inspection to assure construction quality is acceptable.
Design is tailored to the contractors construction procedures and equipment	Requires adequate in-house expertise to review design, submissions and monitor construction operations
Provides incentive for contractor innovation	
Allows contractor to use proprietary knowledge and methods.	

8.3 Plan Preparation

The level of effort required by the owner to prepare plans will vary substantially depending on the chosen contract procurement method and the associated owner/contractor division of responsibilities. The most fundamental effort in plan preparation for an agency would be to prepare "conceptual plans" for a design/build contract. Increasingly higher levels of owner effort are required for performance and procedural plans/specifications prepared for owner designs.

8.3.1 Conceptual Plans for Design/Build Contracting

Conceptual plans furnished by the owner for design/build contracts are recommended to contain, at a minimum, the following geometric, structural, geotechnical, and design information.

A. Geometric Requirements

- Wall plan and profile.
- Beginning and end of wall stations.
- Wall alignment topographic survey.
- Existing and finish grade profiles both behind and in front of the wall.
- Cross sections showing the limits of construction at the retaining wall location intervals of 15 m or closer.
- Horizontal and vertical curve data and wall control points.
- Required wall appurtenances such as barriers, coping, drainage, etc.
- Right-of-way and permanent or temporary construction easement limits, location of all active and abandoned existing utilities, adjacent structures or other potential interferences.
- Staged excavation sequencing.
- Quantity tables showing estimated square footage of wall areas and other pay items.

B. Structural Requirements

- Conceptual details for wall reinforcement and load transfer devices between nail and facing wall or cast-in-place wall.
- Level of corrosion protection required (none, fusion bonded epoxy or full encapsulation).

- Limits and requirements for drainage features beneath, behind, or through the structure.

- Facing finishes, color, and requirements for wall facing elements.

- Required nail and facing reinforcement steel grades and strengths as well as, shotcrete, concrete, and nail grout strengths.

C. Geotechnical Requirements

- 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 wall.

- Subsurface data sheet containing graphic logs of borings and test pits and a generalized description of each deposit (fills should be clearly identified), including soil/rock classification, color, density, moisture, plasticity, rock RQD, groundwater levels, SPT tests and logs of CPT soundings (if performed).

- Refer to availability of subsurface investigation report.

- Subsurface cross sections adequate to define representative conditions in front of and behind the wall along the full wall length.

- Notes to describe anticipated difficult installation conditions or to warn the contractor of latent subsurface conditions such as existing foundations, active and abandoned utilities, etc.

D. Design Requirements

- Applicable code requirements.

- Magnitude, location, and direction of external loads, surcharges, piezometric levels, etc.

- Reference to design methods to be utilized or other provisions such as minimum nail lengths/diameters and minimum shotcrete facing wall thickness.

- References for design of facing and connection to the nails.

- Design criteria including seismic coefficients, soil/rock shear strengths (friction angle and cohesion), unit weights, and design pullout resistances for each strata.

- Minimum partial safety factors (for Service Load Design) or load and resistance factors (for Load and Resistance Factor Design) to be used in the design on the soil/rock shear strength and pullout resistance; surcharges; unit weights; and steel, shotcrete, and concrete materials.

Subsequent to contract award, the contractor must submit detailed design calculations as well as construction-ready **plans and specifications** for the structure based on these requirements. When pre-bid final designs are required these must be submitted for review prior to contract award.

8.3.2 Final Plans

Final or construction-ready plans may either be prepared by the owner for owner-design contracts or the contractor for design/build contracts. In general, all elements required for the preparation of conceptual plans, presented in the previous section, are required for the preparation of final plans with the following additional information typically provided on performance-based final plans:

- General notes specifying construction sequencing or other special construction requirements.
- Design parameters and applicable codes.
- Special notes required to indicate cross sections or specific nails that may require special treatment.
- Special notes advising contractor of known subsurface utilities, obstructions, or interferences.
- Nail wall typical sections.
- Nail details including spacing, size, inclination, corrosion protection details, and capacity used in design.
- Details, dimensions, and schedules for all nails, reinforcing steel, wire mesh, bearing plates, etc., and/or attachment devices for cast-in-place or prefabricated facings.
- A typical cross section of production and test nails defining the nail length, minimum drillhole diameter, inclination, and test nail bonded and unbonded test lengths.
- Wall elevation view showing nail locations and elevations; vertical and horizontal spacing; and the location of wall drainage elements.
- A reference baseline and elevation datum should be shown to fix all nail locations.

Procedural final plans would include the following requirement in addition to those mentioned above:

- Final required drillhole diameter.

8.4 Example Plan Details

An example set of Soil Nail Wall final plans are included in Appendix A, Example Plan Details.

8.5 Guide Construction Specifications

8.5.1 Guide Specifications for Owner-Design

Soliciting bids for construction of a soil nail wall system that has been designed by the owner will require that the owner provide a performance based construction specification that, as a minimum, provides the information presented in Section 8.2.1.1. Example performance based construction specifications for a design prepared by the owner is contained in Appendix B, Permanent Soil Nails and Wall Excavation Guide Specification (Owner-Design) and Appendix C, Shotcrete Facing and Wall Drainage Guide Specifications (Owner-Design).

8.5.2 Guide Specification for Contractor Design/Build Solicitation

Soliciting a contractor design/build bid for construction of a soil nail wall system will require that the owner provide the bidders with a design/build specification that provides sufficient information for the bidders to prepare their bid packages. As a minimum, it is recommended the specification provide the information presented in Section 8.2.2. In addition, the specification should require a contractor submittal that includes design drawings, specifications (if not prepared by the owner), and design calculations. An example specification prepared by the owner for solicitation of a contractor design/build bid is included in appendix B, Soil Nail Retaining Wall Guide Specification (Design/Build Solicitation).

(Note - The Example Plans and Guide Construction Specifications in this manual are in metric (SI) units. A separate set are also available in English units from FHWA).

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APPENDIX A

FEDERAL HIGHWAY ADMINISTRATION
EXAMPLE PLAN DETAILS
SOIL NAIL RETAINING WALL

(SI UNITS)

(An electronic version of this section is available at www.fhwa.dot.gov/bridge**)**

FEDERAL HIGHWAY ADMINISTRATION

EXAMPLE PLAN DETAILS

SOIL NAIL RETAINING WALL
(SI UNITS)

DRAWING INDEX:

SHEET NO.	DRAWING NO.	DRAWING TITLE
SHEET 1	49720	COVER SHEET
SHEET 2	49721	CONSTRUCTION NOTES
SHEET 3	49722	SITE PLAN
SHEET 4	49723	SOIL NAIL WALL ELEVATION - SEGMENTS A AND B
SHEET 5	49724	SOIL NAIL WALL ELEVATION - SEGMENTS C, D AND E
SHEET 6	49725	TYPICAL SECTION AND NAIL DETAILS
SHEET 7	49726	SOIL NAIL CORROSION PROTECTION DETAILS
SHEET 8	49727	WALL FACING DETAILS
SHEET 9	49728	WALL DRAINAGE DETAILS
SHEET 10	49729	SUBSURFACE DATA SHEET

DATE	REVISION	BY

DESIGNED PE NO. ____
DRAWN
CHECKD
REVIEWED
REVIEWED

SOIL NAIL WALL DESIGN MANUAL
EXAMPLE PLANS

SOIL NAIL RETAINING WALL

COVER SHEET

DATE: SEPT. 23, 1996 CALC. BOOK: ____ SHEET: 1 OF: 10
STRUCTURE NO: ____ DRAWING NO.: 49720

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GENERAL NOTES AND SOIL NAIL WALL DESIGN PARAMETERS

1. All Material and workmanship shall be in accordance with the requirements of the State of ____________, Department of Transportation, Standard Specifications for Road, Bridge, and Municipal Construction, dated ____________.

2. The soil nail wall structures have been designed in accordance with the SLD (Service Load Design) procedures contained in the FHWA "Manual for Design and Construction Monitoring of Soil Nail Walls", Report No. FHWA-SA-96-069. Structural design of any individual wall elements not covered in the FHWA manual have been designed in accordance with the requirements of the 1994 AASHTO Standard Specifications for Highway Bridges and latest interim specifications including the Guide Specifications for Seismic Design of Highway Bridges. A pseudo-static seismic acceleration coefficient of 0.2g was used in the design.

3. Reinforced concrete and shotcrete: f_y = 420 MPa
   f_c = 28 MPa
   Nail grout: f_c = 21 MPa

4. Structural steel: AASHTO M183/ASTM A36

5. Unless otherwise noted on the plans, minimum concrete/shotcrete cover measured from the face of concrete/shotcrete to the face of any reinforcing bar shall be as follows:

   Front side of permanent facings exposed to weather: 50 mm
   Permanent shotcrete facings exposed to soil: 75 mm
   Permanent CIP facings cast against temporary shotcrete: 38 mm

6. Unless otherwise shown on the plans, all exterior corners and edges shall have a 20 mm chamfer and all interior corners shall have a 20 mm fillet.

7. Design Soil Parameters:

Soil/Rock Type	Ultimate Friction Angle (degrees)	Ultimate Cohesion (kPa)	Unit Weight (kN/m^3)	Allowable Pullout Resistance (kN/m)
Silty Sand	34	7.2	19.6	22.0
Clayey Silt	30	12.0	18.8	14.5
Silty Sandy Gravel	38	4.8	20.4	44.0

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 5 m in front of the soil nail walls without approval of the Engineer.

12. Where grade slopes away from the back of the wall, the drainage gutter shall not be required.

13. Unless specified otherwise, Engineer will provide survey control points for top of wall alignment. Contractor responsible for survey control as excavation is brought down. See special provisions.

14. At the contractors option, nails may be drilled and installed through a temporary stabilizing berm. See Dwg. 49725.

15. Excavation in the vicinity of the wall face requires special care and effort compared to general roadway excavation. See special provisions and Dwg. 49725.

SOIL NAIL WALL QUANTITIES

ITEM		SEGMENT A	SEGMENT B	SEGMENT C	SEGMENT D	SEGMENT E	SUB TOTAL
NAIL LENGTH (m)	25 M	310	442	356	652	287	2,047
	29 M	0	188	112	448	865.5	1,613.5
	32 M	0	0	0	0	569.5	569.5
PLATES (200 x 200 x 20 mm)	NO.	62	107	66	172	217	624
	WEIGHT (kg)	383	660	407	1,061	1,339	3,850
SHOTCRETE AREA - 100 mm THICK	(Sm)	125.5	242.0	170.0	406.5	495.5	1,439.5
CIP AREA - 200 mm THICK	(Sm)	132.0	262.0	186.5	447.0	520.0	1,547.5
TOTAL SHOTCRETE CONSTRUCTION FACING REINFORCING STEEL	(kg)	551	1,021	719	1,706	1,950	5,947
TOTAL CIP WALL REINFORCING STEEL	(kg)	2,017	3,936	2,784	6,650	7,955	23,350

TYPICAL CONSTRUCTION SEQUENCE

1. Walls shall be built from the top down in accordance with the staged excavation lifts shown on Dwg. 49725 and the 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 Engineer.

   2.1 Install pre-production verification test nails. See table below.

   2.2 Excavate to stage I rough grade.

   2.3 Trim to final wall face excavation line or to stabilizing berm (if used).

   2.4 Drill, install and grout nails. Trim stabilization berm (if used) to final wall face excavation line.

   2.5 Install geocomposite drainage strip.

   2.6 Place reinforcing and apply shotcrete. No excavation which has exposed wall face shall be left unstabilized by shotcrete at the end of the work day unless Engineer approves otherwise.

   2.7 Perform nail pullout tests per specifications after shotcrete and nail grout have attained their specified strengths.

3. 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 Dwg. 49728.

4. Install CIP final facing.

VERIFICATION TEST NAIL LOCATIONS:

SOIL TYPE	WALL SEGMENT	WALL STATION (m)	WALL ELEVATION (m)
SILTY SAND	B	39.3	113.5
	E	197.5	123.5
CLAYEY SILT	B	78.8	114.0
	E	186.0	120.0
SILTY SANDY GRAVEL	D	153.5	118.0
	E	213.0	117.5

SEE SPECIFICATIONS FOR TESTING REQUIREMENTS.
NOTE: SET BONDED LENGTH COMPLETELY INTO ONE SOIL OR ROCK UNIT FOR VERIFICATION TEST NAILS.

DATE	REVISION	BY

DESIGNED ______ PE NO. ______
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REVIEWED ______
REVIEWED ______

SOIL NAIL WALL DESIGN MANUAL
EXAMPLE PLANS

SOIL NAIL RETAINING WALL

CONSTRUCTION NOTES

DATE: SEPT. 23, 1998 CALC. BOOK: ____ SHEET: 2 OF: 10
STRUCTURE NO: ____________ DRAWING NO.: 49721

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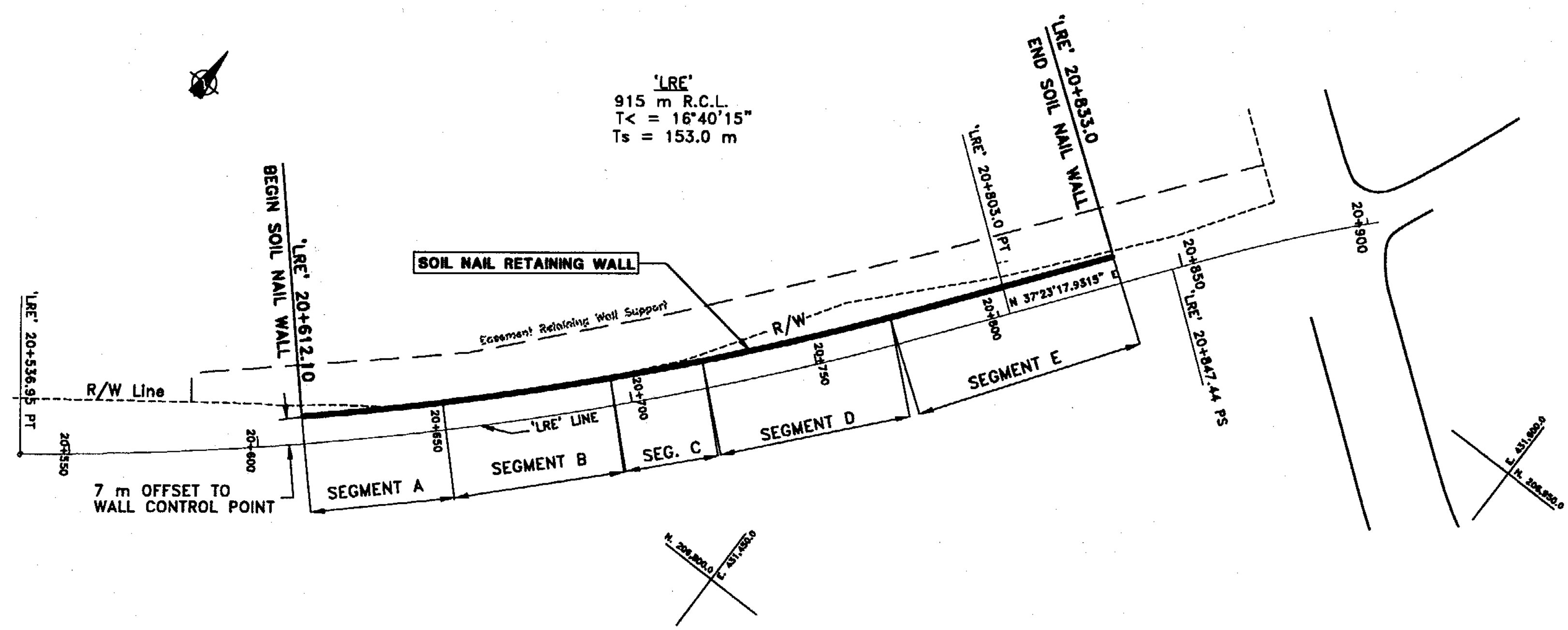

SITE PLAN

SCALE: 1: 1200

LEGEND:

- — — Retaining Wall Support Easement
- - - - Right-of-Way (R/W) Line

DATE	REVISION	BY

DESIGNED ______ PE NO. ______
DRAWN ______
CHECKD ______
REVIEWED ______
REVIEWED ______

SOIL NAIL WALL DESIGN MANUAL
EXAMPLE PLANS

SOIL NAIL RETAINING WALL

SITE PLAN

DATE: SEPT. 23, 1998 CALC. BOOK: ____ SHEET: 3 OF: 10
STRUCTURE NO: __________ DRAWING NO.: 49722

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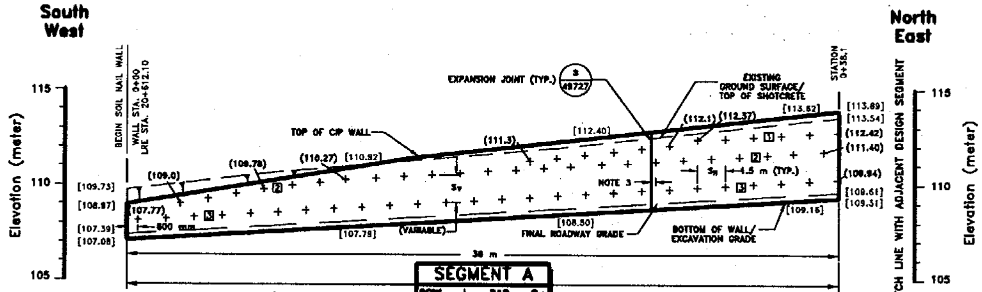
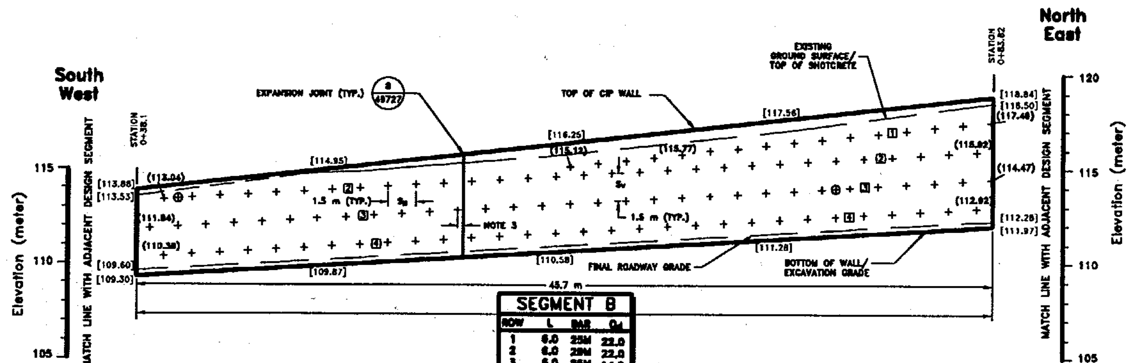


SEGMENT A			
ROW	L	BAR	Q_d
1	5.0	25M	22.0
2	5.0	25M	22.0
3	5.0	25M	14.5

WEST ELEVATION

SCALE 1:250



SEGMENT B			
ROW	L	BAR	Q_d
1	6.0	25M	22.0
2	6.0	25M	22.0
3	6.0	25M	14.5
4	6.0	25M	14.5

WEST ELEVATION

SCALE 1:250

LEGEND:

+	Nail	L	Drilled Length of Nail (meter)
⊕	Verification Test Nail	BAR	Steel Bar Size
2	Nail Row	Q_d	Allowable Pullout Resistance (kN/m)
(370.7)	Nail Elevation	S_H	Horizontal Nail Spacing (meter)
[373.6]	Grade Elevation	S_V	Vertical Nail Spacing (meter)

NOTES:

1. Elevations shown are based on the National Geodetic Vertical Datum of 1929 (M.S.L.=0.0)
2. Nail elevations not shown shall be linearly interpolated between those shown.
3. The expansion joint locations shall be located prior to the construction of soil nails and at least 300 mm clear distance shall be provided between the joint and the nails.
4. If reinforcing bars with cut threads are used for nails, the next larger bar size above that shown shall be provided at no additional cost.

DATE	REVISION	BY

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CHECKD ____
REVIEWED ____
REVIEWED ____

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

SOIL NAIL RETAINING WALL

WALL ELEVATION - SEGMENTS A AND B

DATE: SEPT. 23, 1999 CALC. BOOK: ___ SHEET: 4 OF: 10
STRUCTURE NO: ________ DRAWING NO.: 49723

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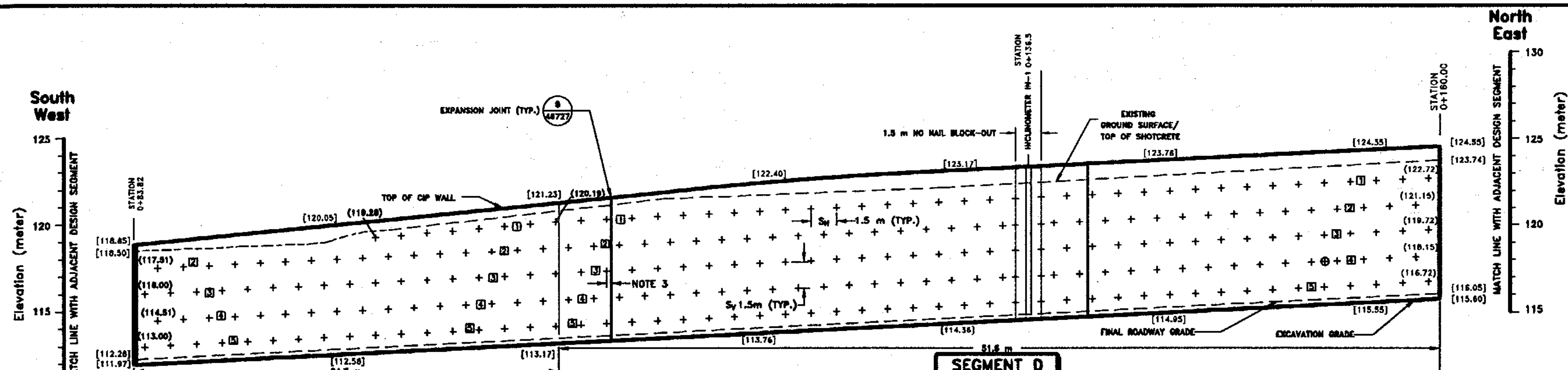


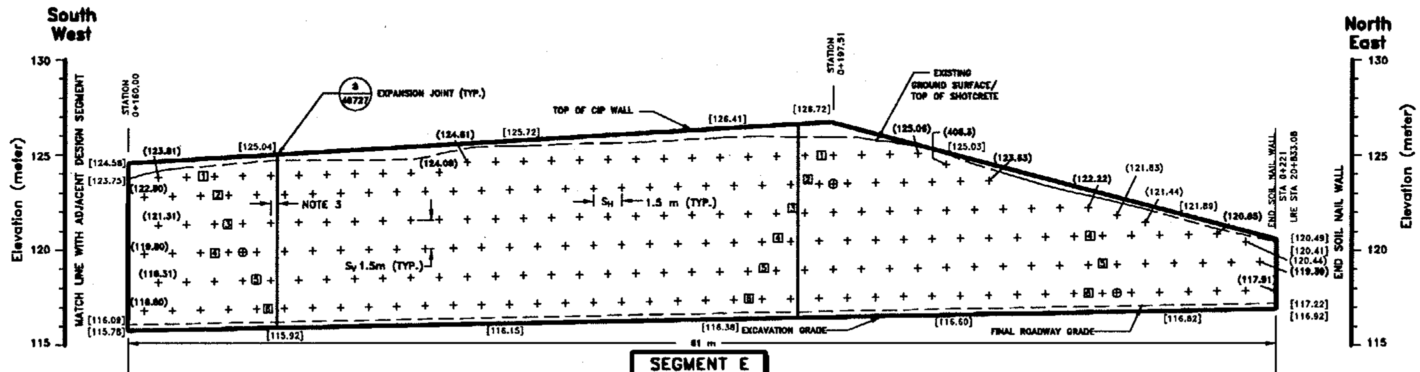
SEGMENT C

ROW	L	BAR	Q_d
1	7.0	25M	22.0
2	7.0	29M	22.0
3	6.0	25M	14.5
4	6.0	25M	14.5
5	6.0	25M	14.5

SEGMENT D

ROW	L	BAR	Q_d
1	7.0	29M	22.0
2	7.0	29M	22.0
3	6.0	29M	14.5
4	6.0	25M	44.0
5	6.0	25M	44.0

WEST ELEVATION
0 5 meter SCALE 1:250



SEGMENT E

ROW	L	BAR	Q_d
1	8.5	29M	22.0
2	8.5	32M	22.0
3	8.5	32M	14.5
4	8.5	29M	14.5
5	7.0	29M	44.0
6	7.0	25M	44.0

WEST ELEVATION
0 5 meter SCALE 1:250

LEGEND:

+	Nail	L	Drilled Length of Nail (meter)
⊕	Verification Test Nail	BAR	Steel Bar Size
[2]	Nail Row	Q_d	Allowable Pullout Resistance (kN/m)
(370.7)	Nail Elevation	S_H	Horizontal Nail Spacing (meter)
[373.6]	Grade Elevation	S_V	Vertical Nail Spacing (meter)

NOTES:

1. Elevations shown are based on the National Geodetic Vertical Datum of 1929 (M.S.L.=0.0)
2. Nail elevations not shown shall be linearly interpolated between those shown.
3. The expansion joint locations shall be located prior to the construction of soil nails and at least 300 mm clear distance shall be provided between the joint and the nails.
4. If reinforcing bars with cut threads are used for nails, the next larger bar size above that shown shall be provided at no additional cost.

DATE	REVISION	BY

DESIGNED PE NO.
DRAWN
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REVIEWED

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

WALL ELEVATION - SEGMENTS C, D & E

SOIL NAIL RETAINING WALL

DATE: SEPT. 23, 1996 CALC. BOOK: SHEET: 6 OF: 10
STRUCTURE NO: DRAWING NO.: 49724

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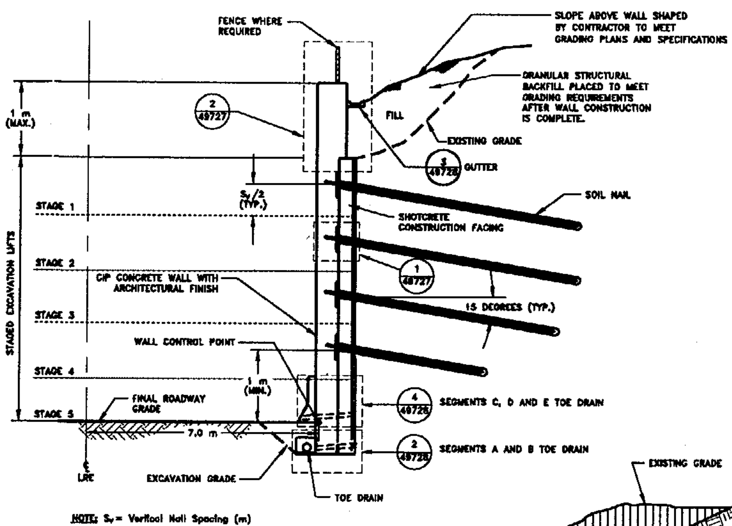

1 TYPICAL SECTION 49725
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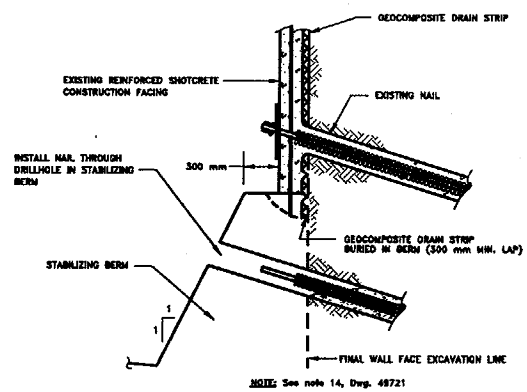
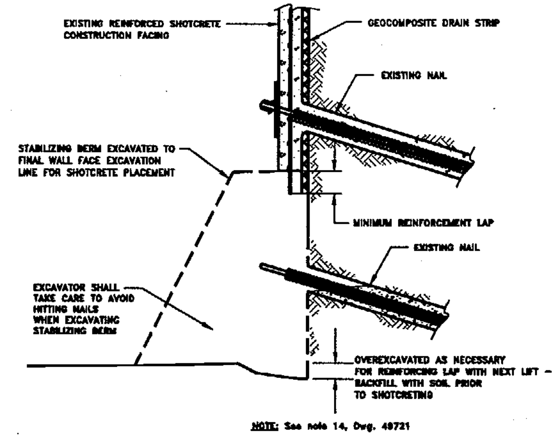
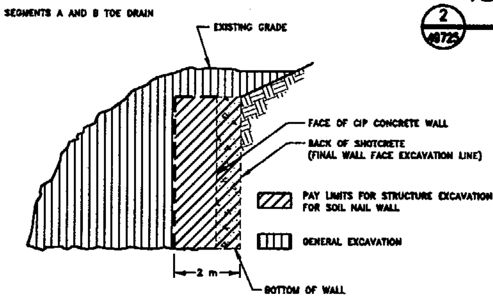


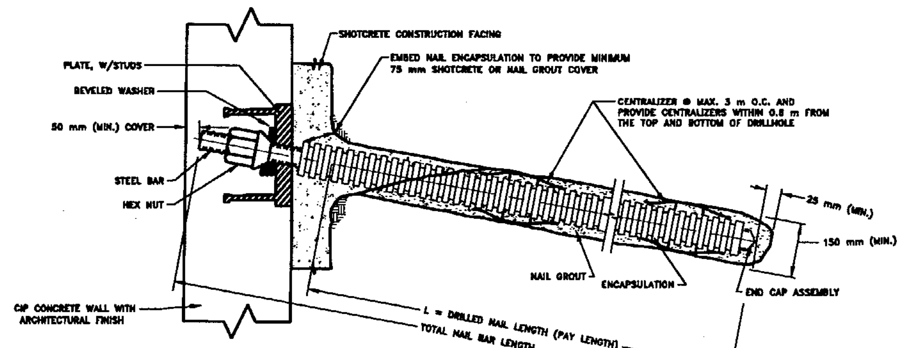
2 NAIL INSTALLATION THROUGH TEMPORARY STABILIZING BERM (CONTRACTOR OPTION) 49725
NOT TO SCALE



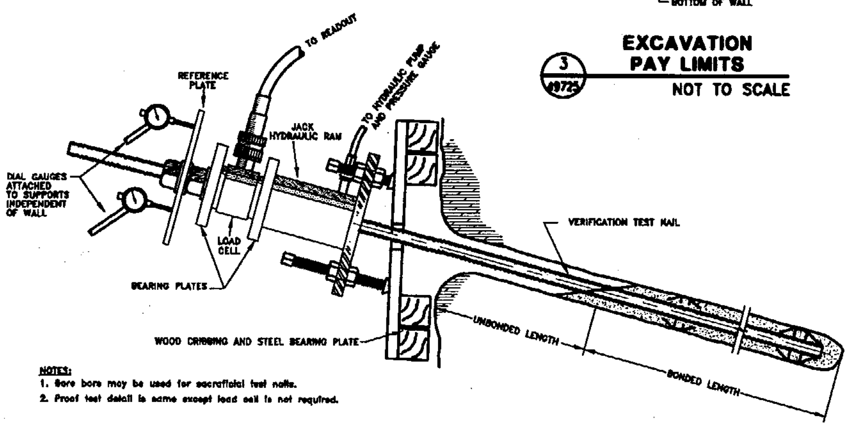
2A EXCAVATION OF TEMPORARY STABILIZING BERM FOR SHOTCRETE PLACEMENT (CONTRACTOR OPTION) 49725
NOT TO SCALE



3 EXCAVATION PAY LIMITS 49725
NOT TO SCALE



4 ENCAPSULATED PRODUCTION SOIL NAIL DETAIL 49725
NOT TO SCALE



NOTES:
1. Bare bars may be used for sacrificial test nails.
2. Proof test detail is same except load cell is not required.

5 VERIFICATION TEST SOIL NAIL DETAIL 49725
NOT TO SCALE

DATE	REVISION	BY

DESIGNED ____ PE NO. ____
DRAWN ____
CHECKD ____
REVIEWED ____
REVIEWED ____

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

SOIL NAIL RETAINING WALL

TYPICAL SECTION AND NAIL DETAILS

DATE: SEPT. 23, 1998 CALC. BOOK: ____ SHEET: 6 OF: 10
STRUCTURE NO: ____________ DRAWING NO. 49725

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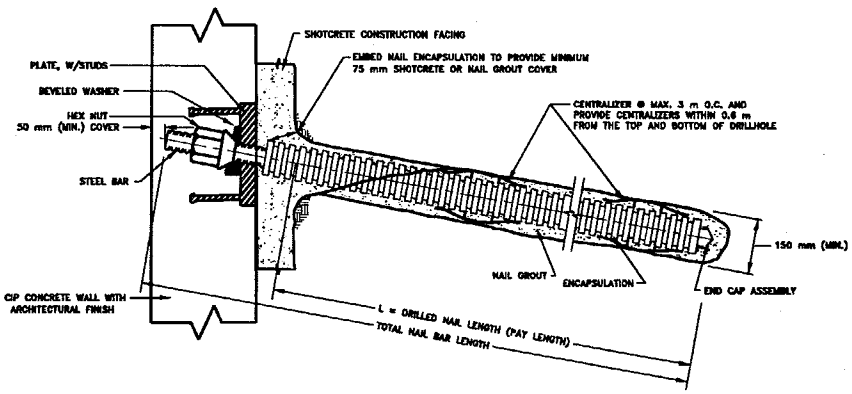
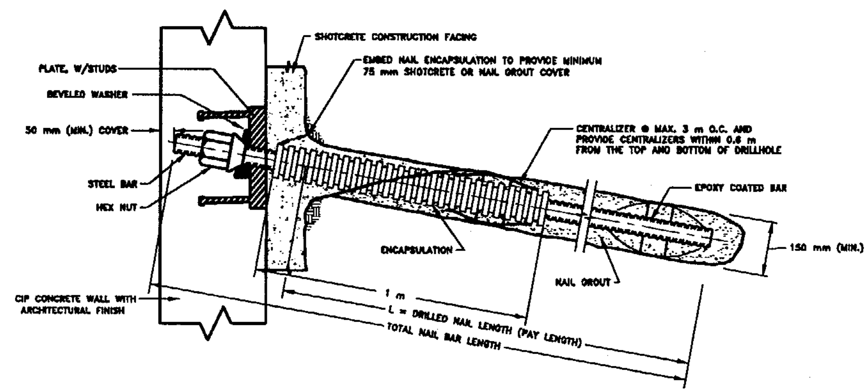


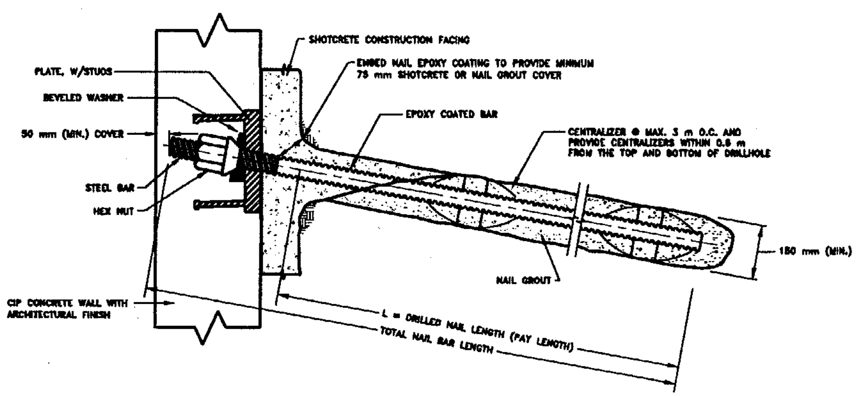
1 / 49726 **ENCAPSULATED SOIL NAIL WITH BARE STEEL AT THE TOP DETAIL**

NOT TO SCALE



3 / 49726 **UPPER 1 METER ENCAPSULATED EPOXY COATED SOIL NAIL DETAIL (PER CALTRANS)**

NOT TO SCALE



2 / 49726 **EPOXY COATED NAILS WITH MACHINE THREADS DETAIL**

NOT TO SCALE

NOTE TO DESIGNER:
The three alternate corrosion protection details shown on this sheet are intended to illustrate alternate protection systems in use. Typically, only one of the details would be specified on an individual project.

DATE	REVISION	BY		
			DESIGNED	PE NO.
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			REVIEWED	

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

SOIL NAIL RETAINING WALL

ALTERNATIVE SOIL NAIL CORROSION PROTECTION DETAILS

DATE: SEPT. 23, 1996 CALC. BOOK: ____ SHEET: 7 OF: 10
STRUCTURE NO: ____ DRAWING NO.: 49726

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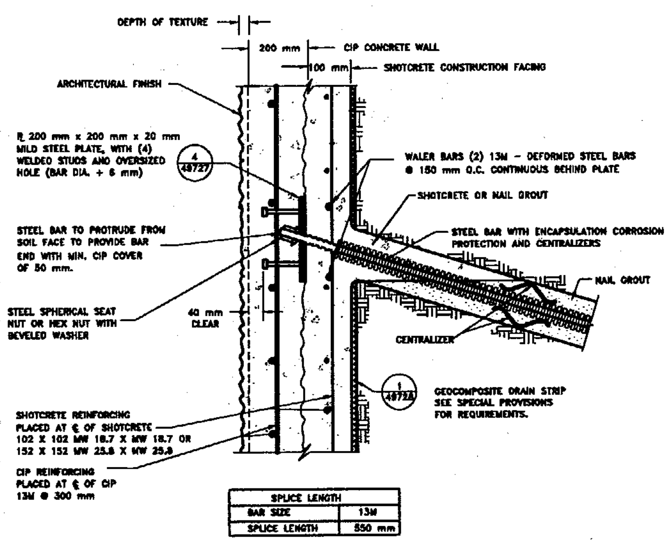
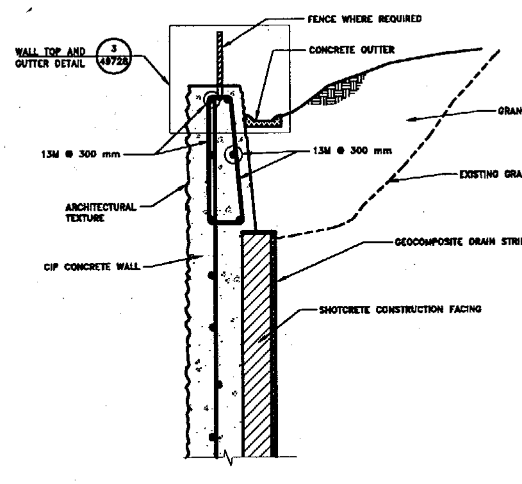
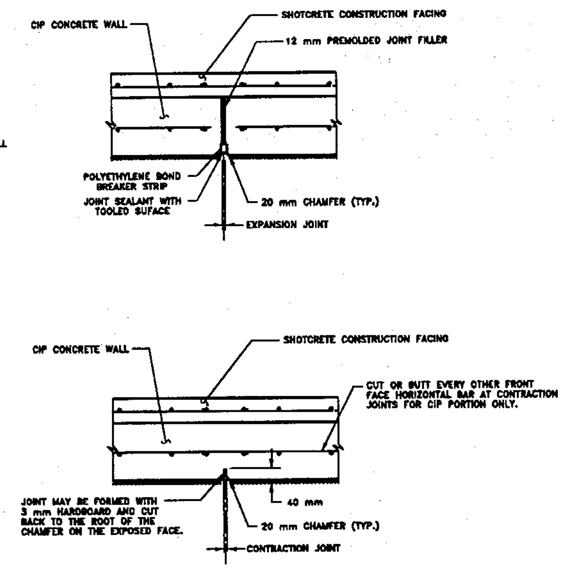


SPLICE LENGTH	
BAR SIZE	13M
SPLICE LENGTH	550 mm

1 / 49727 **FINISHED WALL SECTION**
NOT TO SCALE

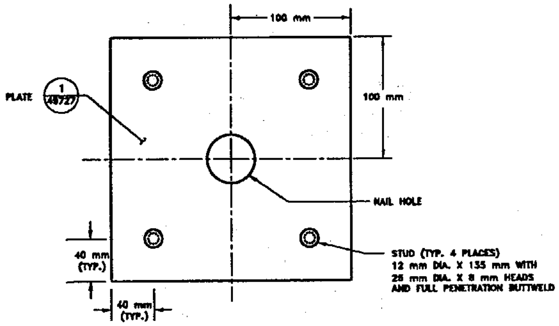


2 / 49727 **CANTILEVER DETAIL**
NOT TO SCALE

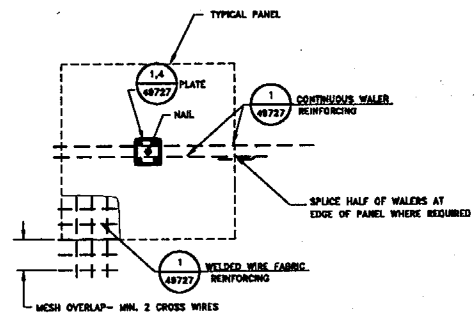


3 / 49727 **TYPICAL EXPANSION AND CONTRACTION JOINTS**
NOT TO SCALE

NOTE:
EXPANSION JOINTS IN CIP WALL TO BE LOCATED AT A MAXIMUM SPACING OF 28 m CENTER TO CENTER, CENTERED BETWEEN NAILS, EXCEPT IF THE JOINT IS WITHIN .5 m OF A STEP AT THE TOP OF WALL, THE JOINT IS TO BE LOCATED AT THAT STEP. CONTRACTION JOINTS IN CIP WALL TO BE LOCATED AT MAXIMUM SPACING OF 9 m CENTER TO CENTER. EXPANSION AND CONTRACTION JOINTS NOT REQUIRED THROUGH SHOTCRETE CONSTRUCTION FACING.



4 / 49727 **CONNECTOR PLATE WITH STUD DETAIL**
NOT TO SCALE



5 / 49727 **TYPICAL SHOTCRETE PANEL STEEL**
NOT TO SCALE

DATE	REVISION	BY

DESIGNED ______ PE NO. ______
DRAWN ______
CHECKD ______
REVIEWED ______
REVIEWED ______

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

SOIL NAIL RETAINING WALL

WALL FACING DETAILS

DATE: SEPT. 23, 1996 CALC. BOOK: ______ SHEET: 8 OF: 10
STRUCTURE NO: ______ DRAWING NO.: 49727

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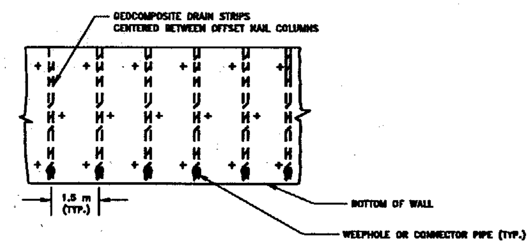
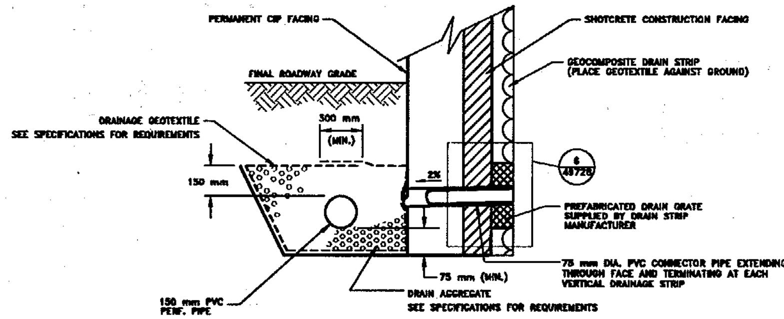


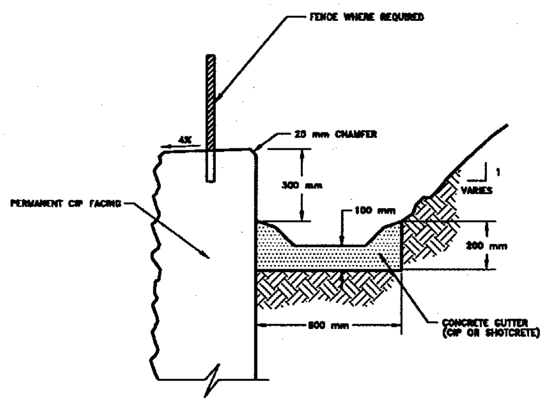
GEOCOMPOSITE DRAINAGE STRIP DETAIL (1 / 49728)

NOT TO SCALE



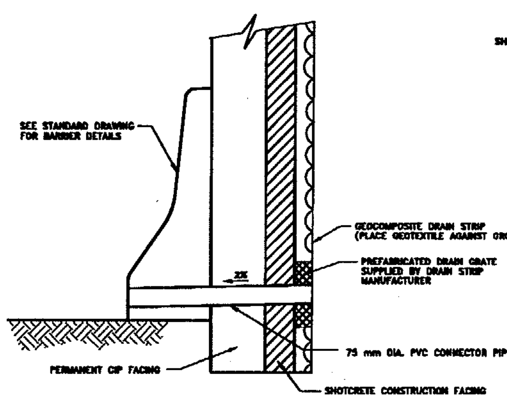
SEGMENTS A & B TYPICAL WALL TOE DRAIN (2 / 49728)

NOT TO SCALE



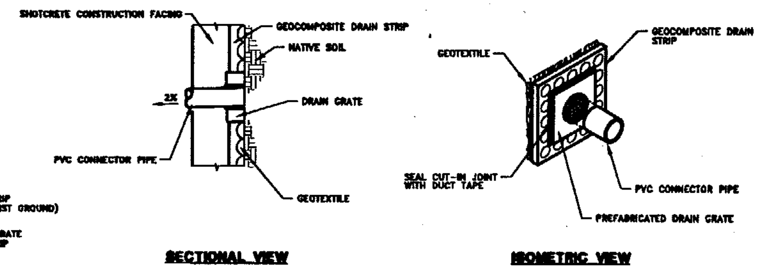
WALL TOP AND CONCRETE GUTTER DETAIL (3 / 49728)

NOT TO SCALE



SEGMENTS C, D & E TYPICAL WALL BASE DRAIN, TRAFFIC BARRIER AND PVC CONNECTOR PIPE DETAIL (4 / 49728)

NOT TO SCALE



NOTE: DRAIN GRATE INSTALLATION SHALL NOT DISRUPT GEOTEXTILE

DRAIN GRATE DETAILS (6 / 49728)

NOT TO SCALE

DATE	REVISION	BY

DESIGNED ____ PE NO. ____
DRAWN
CHECKD
REVIEWED
REVIEWED

SOIL NAIL WALL DESIGN MANUAL EXAMPLE PLANS

SOIL NAIL RETAINING WALL

WALL DRAINAGE DETAILS

DATE: SEPT. 23, 1996 CALC. BOOK: ____ SHEET: 9 OF: 10
STRUCTURE NO: ____ DRAWING NO.: 49728

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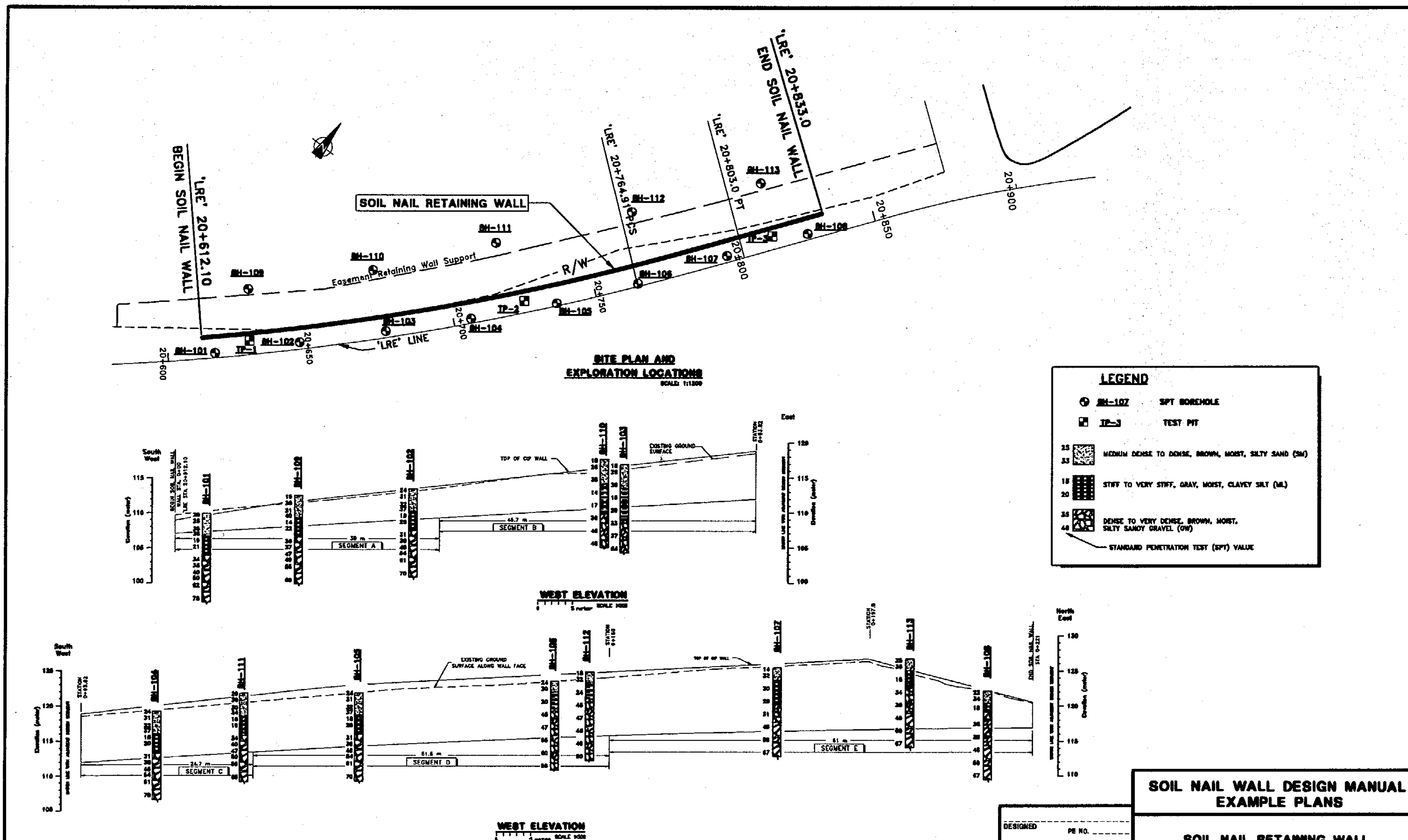

SOIL NAIL WALL DESIGN MANUAL
EXAMPLE PLANS

SOIL NAIL RETAINING WALL

SUBSURFACE DATA SHEET

DESIGNED	PE NO.
DRAWN	
CHECKD	
REVIEWED	
REVIEWED	

DATE	REVISION	BY

DATE: SEPT. 23, 1996 CALC. BOOK:____ SHEET: 10 OF: 10
STRUCTURE NO:________ DRAWING NO.: 49729

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APPENDIX B1

FHWA GUIDE SPECIFICATION
FOR
PERMANENT SOIL NAILS AND WALL EXCAVATION
(OWNER-DESIGN)

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDB1.SI
Last Revision 10/31/98

(An electronic version of this section is available at www.fhwa.dot.gov/bridge**)**

FHWA GUIDE SPECIFICATION FOR PERMANENT SOIL NAIL AND WALL EXCAVATION (OWNER-DESIGN)

1.0 DESCRIPTION. The Work shall consist of constructing permanent soil nail retaining walls as specified herein and shown on the Plans. The Contractor shall furnish all labor, materials and equipment required for completing the Work. The Contractor shall select the method of excavation, drilling method and equipment, final drillhole diameter(s), and grouting procedures to meet the performance requirements specified herein. ***(See Commentary 1.0)***

Soil nailing work shall include excavating in accordance with the staged lifts shown in the Plans; drilling soil nail drillholes to the specified minimum length and orientation indicated on the Plans; providing, placing and grouting the encapsulated or epoxy coated nail bar tendons into the drillholes; placing drainage elements; placing shotcrete reinforcement; applying shotcrete facing over the reinforcement; attaching bearing plates and nuts; performing nail testing; and installing instrumentation (if required). Shotcrete facing and wall drainage construction is covered by the Shotcrete Facing and Wall Drainage Specification. CIP concrete facing construction (if required) is covered by the Standard Specifications and/or CIP Facing Special Provisions. Soil nail wall instrumentation (if required) is covered by the Soil Nail Wall Instrumentation Specification.

The term "Soil Nail" as used in these specifications is intended as a generic term and refers to a reinforcing bar grouted into a drilled hole installed in any type of ground. Soil nail walls are built from the top down in existing ground.

Soil and rock properties, strength parameters, partial safety factors or load and resistance factors, design requirements and other criteria are shown on the Plans. In addition to the subsurface information presented in the Plans, Geotechnical Report(s) titled ________ are also available to bidders and can be obtained from ________.

Where the imperative mood is used within this Specification for conciseness, "the Contractor shall" is implied.

1.1 Soil Nail Contractor's Experience Requirements and Submittal. The soil nailing Contractor shall submit a project reference list verifying the successful construction completion of at least 3 permanent soil nail retaining wall projects during the past 3 years totaling at least 1000 square meters of wall face area and at least 500 permanent soil nails. A brief description of each project with the Owner's name and current phone number shall be included.

A Registered Professional Engineer employed by the soil nailing Contractor and having experience in the construction of permanent soil nail retaining walls on at least 3 completed projects over the past 3 years shall supervise the Work. The on-site supervisor and drill rig operators shall have experience installing permanent soil nails on at least 3 projects over the past 3 years. The Contractor shall not use consultants or manufacturer's representatives to satisfy the requirements of this section.

At least 30 calendar days before starting the wall, the soil nail Contractor shall submit 5 copies of the completed project reference list and a list identifying the supervising Engineer, drill rig operators, and on site supervisors assigned to the project. The personnel list shall contain a summary of each individual's experience and be complete enough for the Engineer to determine whether each individual satisfies the required qualifications. The Engineer will approve or reject the Contractor's qualifications within 15 calendar days after receipt of a complete submission. Work shall not be started nor materials ordered until the Engineer's written approval of the Contractor's qualifications is given.

The Engineer may suspend the Work if the Contractor uses non-approved personnel. If work is suspended, the Contractor shall be fully liable for all resulting costs and no adjustment in contract time will result from the suspension.

1.2 Construction Site Survey. Before bidding the Work, the Contractor shall review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, and location of existing structures and above ground facilities.

The Contractor is responsible for field locating and verifying the location of all utilities shown on the Plans prior to starting the Work. Maintain uninterrupted service for those utilities designated to remain in service throughout the Work. Notify the Engineer of any utility locations different from shown on the Plans that may require nail relocations or wall design modification. Subject to the Engineer's approval, additional cost to the Contractor due to nail relocations and/or wall design modification resulting from utility locations different from shown on the Plans, will be paid as Extra Work. ***(See Commentary 1.2)***

Prior to start of any wall construction activity, the Contractor and Engineer shall jointly inspect the site to observe and document the pre-construction condition of the site, existing structures and facilities. During construction, the Contractor shall observe the conditions above the soil nail wall on a daily basis for signs of ground movement in the vicinity of the wall. Immediately notify the Engineer if signs of movements such as new cracks in structures, increased size of old cracks or separation of joints in structures, foundations, streets or paved and unpaved surfaces are observed. If the Engineer determines that the movements exceed those anticipated for typical soil nail wall construction and require corrective action, the Contractor shall take corrective actions necessary to stop the movement or perform repairs. When due to the Contractor's methods or operations or failure to follow the specified/approved construction sequence, as determined by the Engineer, the costs of providing corrective actions will be borne by the Contractor. When due to differing site conditions, as determined by the Engineer, the costs of providing corrective actions will be paid as Extra Work.

1.3 Construction Submittals.

Upon approval of the soil nailing Contractor's qualifications submittal set forth in Section 1.1, submit 5 copies of the following information, in writing, to the Engineer for review and approval. Provide submittal item numbers 1 through 1D at least 15 calendar days prior to initiating the nail wall construction and submittal items 2 through 6 at least 15 calendar days prior to start of nail the job site at no additional cost. Materials for soil nail structures shall consist of the following:

Solid Bar Nail Tendons: AASHTO M31/ASTM A615, Grade 420 or 520, ASTM A722 for Grade 1035. Deformed bar, continuous without splices or welds, new, straight, undamaged, bare or epoxy coated or encapsulated as shown on the Plans. Threaded a minimum of 150 mm on the wall anchorage end to allow proper attachment of bearing plate and nut. Threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g. continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, provide the next larger bar number designation from that shown on the Plans, at no additional cost.

Fusion Bonded Epoxy Coating: ASTM A775. Minimum 0.3 mm thickness electrostatically applied. Bend test requirements are waived. Coating at the wall anchorage end of epoxy coated bars may be omitted over the length provided for threading the nut against the bearing plate.

Encapsulation: Minimum 1 mm thick corrugated HDPE tube conforming to AASHTO M252 or corrugated PVC tube conforming to ASTM D1784, Class 13464-B. Encapsulation shall provide at least 5 mm of grout cover over the nail bar and be resistant to ultra violet light degradation, normal handling stresses, and grouting pressures. Factory fabrication of the encapsulation is preferred. Upon the Engineers approval, the encapsulation may be field fabricated if done in strict accordance with the manufacturer's recommendations. ***(See Commentary 2a and 2b)***

Centralizers: Manufactured from Schedule 40 PVC pipe or tube, steel or other material not detrimental to the nail steel (wood shall not be used); securely attached to the nail bar; sized to position the nail bar within 25 mm of the center of the drillhole; sized to allow tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole.

Nail Grout: Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 10.5 Mpa and a minimum 28-day compressive strength of 21 Mpa per AASHTO T106/ASTM C109. ***(See Commentary 2c)***

Admixtures: AASHTO M194/ASTM C494. Admixtures which control bleed, improve flowability, reduce water content and retard set may be used in the grout subject to review and acceptance by the Engineer. Accelerators are not permitted. Expansive admixtures may only be used in grout used for filling sealed encapsulations. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations.

Cement: AASHTO M85/ASTM C150, Type I, II, III or V.

Fine Aggregate: AASHTO M6/ASTM C33.

Film Protection: Polyethylene film per AASHTO M171.

Bar Couplers: Bar couplers shall develop the full ultimate tensile strength of the bar as certified by the manufacturer.

2.1 Materials Handling And Storage. Store cement to prevent moisture degradation and partial hydration. Do not use cement that has become caked or lumpy. Store aggregates so that segregation and inclusion of foreign materials are prevented. Do not use the bottom 150 mm of aggregate piles in contact with the ground.

Store steel reinforcement on supports to keep the steel from contacting the ground. Damage to the nail steel as a result of abrasion, cuts, nicks, welds, and weld splatter shall be cause for rejection. Do not ground welding leads to nail bars. Protect nail steel from dirt, rust, and other deleterious substances prior to installation. Heavy corrosion or pitting of nails shall be cause for rejection. Light rust that has not resulted in pitting is acceptable. Place protective wrap over anchorage end of nail bar to which bearing plate and nut will be attached to protect during handling, installation, grouting and shotcreting.

Do not move or transport encapsulated nails until the encapsulation grout has reached sufficient strength to resist damage during handling. Handle encapsulated nails in a manner that will prevent large deflections, distortions or damage. Repair encapsulated nails that are damaged or defective in accordance with the manufacturer's recommendations or remove them from the site.

Handle and store epoxy coated bars in a way that will prevent them from being damaged beyond what is permitted by ASTM 3963. Repair damaged epoxy coating in accordance with ASTM A775 and the coater's recommendations using an epoxy field repair kit approved by the epoxy manufacturer. Repaired areas shall have a minimum 0.3 mm coating thickness.

3.0 CONSTRUCTION REQUIREMENTS

3.1 Site Drainage Control. Provide positive control and discharge of all surface water that will affect construction of the soil nail retaining wall. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water at no additional cost. Upon substantial completion of the wall, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss. ***(See Commentary 3.1a)***

The regional groundwater table is anticipated to be below the level of the wall excavation based on the results of the geotechnical site investigation. Localized areas of perched water or seepage may be encountered during excavation at the interface of geologic units or from localized groundwater seepage areas. ***(See Commentary 3.1b)***

Immediately contact the Engineer if unanticipated existing subsurface drainage structures are discovered during excavation. Suspend work in these areas until remedial measures meeting the Engineer's approval are implemented. Capture surface water runoff flows and flows from existing subsurface drainage structures independently of the wall drainage network and convey them to an outfall structure or storm sewer, as approved by the Engineer. Cost of remedial measures required to capture and dispose of water resulting from encountering unanticipated subsurface drainage structures will be paid for as Extra Work.

3.2 Excavation. Coordinate the work and the excavation so the soil nail wall is safely constructed. Perform the wall construction and excavation sequence in accordance with the Plans and approved submittals. No excavations steeper than those specified herein or shown on the Plans will be made above or below the soil nail wall without written approval of the Engineer.

3.2.1 Excavation and Wall Alignment Survey Control

Unless specified otherwise, the Engineer will provide survey reference and control points at or offset along the top of wall alignment at approximate 10 meter intervals prior to starting wall excavation. The Contractor will then be responsible for providing the necessary survey and alignment control during excavation of each lift, locating and drilling each drillhole within the allowable tolerances and for performing the wall excavation and nail installation in a manner which will allow for constructing the shotcrete construction facing to the specified minimum thickness and such that the finish CIP structural facing can be constructed to the specified minimum thickness and to the line and grade indicated in the Plans. Where the as-built location of the front face of the shotcrete exceeds the allowable tolerance from the wall control line shown on the Plans, the Contractor will be responsible for determining and bearing the cost of remedial measures necessary to provide proper attachment of nail head bearing plate connections and satisfactory placement of the final facing, as called for on the Plans.

3.2.2 General Roadway Excavation. Complete clearing, grubbing, grading and excavation above and behind the wall before commencing wall excavation. Do not overexcavate the original ground behind the wall or at the ends of the wall, beyond the limits shown on the Plans. Do not perform general roadway excavation that will affect the soil nail wall until wall construction starts. Roadway excavation shall be coordinated with the soil nailing work and the excavation shall proceed from the top down in a horizontal staged excavation lift sequence with the ground level for each lift excavated no more than mid-height between adjacent nail rows, as illustrated on the Plans. Do not excavate the full wall height to the final wall alignment as shown on the Plans but maintain a working bench of native material to serve as a platform for the drilling equipment. The bench shall be wide enough to provide a safe working area for the drill equipment and workers. ***(See Commentary 3.2.2)***

Perform rock blasting within 60 meters of the soil nail wall using controlled blasting techniques designed by a qualified blasting consultant or a Professional Engineer registered in the State of ____. Blasting shall not damage completed soil nail work or disrupt the remaining ground to be soil nailed or shotcreted. Repair damaged areas at no additional cost.

3.2.3 Soil Nail Wall Structure Excavation. Structure excavation in the vicinity of the wall face will require special care and effort compared to general earthwork excavation. The excavation Contractor should take this into account during bidding. Due to the close coordination required between the soil nail Contractor and the excavation Contractor, the excavation Contractor shall perform the structure excavation for the soil nail wall under the direction of the soil nail specialty Contractor. The structure excavation pay limits are shown on the Plans.

Excavate to the final wall face using procedures that: (1) prevent over excavation; (2) prevent ground loss, swelling, air slaking, or loosening; (3) prevent loss of support for completed portions of the wall; (4) prevent loss of soil moisture at the face; and (5) and prevent ground freezing. Costs associated with additional thickness of shotcrete or concrete or other remedial measures required due to irregularities in the cut face, excavation overbreak or inadvertent over excavation, shall be borne by the Contractor.

The exposed unsupported final excavation face cut height shall not exceed the vertical nail spacing plus the required reinforcing lap or the short-term stand-up height of the ground, whichever is less. Complete excavation to the final wall excavation line and application of the shotcrete in the same work shift unless otherwise approved by the Engineer. Application of the shotcrete may be delayed up to 24 hours if the Contractor can show that the delay will not adversely affect the excavation face stability. A polyethylene film over the face of the excavation may reduce degradation of the cut face caused by changes in moisture. Damage to existing structures or structures included in the Work shall be repaired and paid by the Contractor where approval is granted for the extended face exposure period.

At the Contractor's option, during each excavation lift, nails may be drilled and installed through a temporary stabilizing berm, as illustrated on the Plans. Purpose of the stabilizing berm is to prevent or minimize instability or sloughing of the final excavation face due to ground conditions and/or drilling action. The stabilizing berm geometry illustrated on the plans shows the top of berm extending horizontally out from the bottom front face of the overlying shotcrete a distance of 0.3 meters and cut down from that point to the base grade for that excavation lift at a slope not steeper than 1H:1V. The Contractor may use a different berm geometry than illustrated on the Plans, upon satisfactory demonstration that the different geometry provides satisfactory performance. Following the installation of nails in that lift, excavate the temporary stabilizing berm to the final wall face excavation line and clean the final excavation face of all loose materials, mud, rebound and other foreign matter which could prevent or reduce shotcrete bond. Ensure that installed nails and corrosion protection are not damaged during excavation of the stabilizing berm. Repair or replace nails or corrosion protection damaged or disturbed during excavation of the stabilizing berm, to the Engineer's satisfaction, at no additional cost. Do not excavate the stabilizing berm until the nail grout has aged for at least 24 hours. Remove hardened nail grout protruding from the final wall excavation line more than 50 mm in a manner that prevents fracturing the grout at the nail head. Sledge hammer removal of the grout is not allowed. The use of hand held rock chippers is acceptable provided their use does not damage or disturb the remaining grout at the nail head, the nail bar or corrosion protection. Alternative excavation and soil nail installation methods that meet these objectives may be submitted to the Engineer for review in accordance with the Submittals section. ***(See Commentary 3.2.3)***

Excavation to the next lift shall not proceed until nail installation, reinforced shotcrete placement, attachment of bearing plates and nuts and nail testing has been completed and accepted in the current lift. Nail grout and shotcrete shall have cured for at least 72 hours or attained at least their specified 3-day compressive strength before excavating the next underlying lift. Excavating the next lift in less than 72 hours will only be allowed if the Contractor submits compressive strength test results, for tests performed by a qualified independent testing lab, verifying that the nail grout and shotcrete mixes being used will provide the specified 3-day compressive strengths in the lesser time.

Notify the Engineer immediately if raveling or local instability of the final wall face excavation occurs. Unstable areas shall be temporarily stabilized by means of buttressing the exposed face with an earth berm or other methods. Suspend work in unstable areas until remedial measures are developed.

3.2.4 Wall Discontinuities. Where the Contractor's excavation and installation methods result in a discontinuous wall along any nail row, the ends of the constructed wall section shall extend beyond the ends of the next lower excavation lift by at least 3 meters. Slopes at these discontinuities shall be constructed to prevent sloughing or failure of the temporary slopes. If sections of the wall are to be constructed at different times, prevent sloughing or failure of the temporary slopes at the end of each wall section.

3.2.5 Excavation Face Protrusions, Voids or Obstructions. Remove all or portions of cobbles, boulders, rubble or other subsurface obstructions encountered at the wall final excavation face which will protrude into the design shotcrete facing. Determine method of removal of face protrusions, including method to safely secure remnant pieces left behind the excavation face and for promptly backfilling voids resulting from removal of protrusions extending behind the excavation face. Notify the Engineer of the proposed method(s) for removal of face protrusions at least 24 hours prior to beginning removal. Voids overbreak or over-excavation beyond the plan wall excavation line resulting from the removal of face protrusions or excavation operations shall be backfilled with shotcrete or concrete, as approved by the Engineer. Removal of face protrusions and backfilling of voids or over-excavation is considered incidental to the work. Cost due to removal of unanticipated man-made obstructions will be paid as Extra Work.

3.3 Nail Installation. Determine the required drillhole diameter(s), drilling method, grout composition and installation method necessary to achieve the nail pullout resistance(s) specified herein or on the Plans, in accordance with the nail testing acceptance criteria in the Nail Testing section.

No drilling or installation of production nails will be permitted in any soil/rock unit until successful pre-production verification testing of nails is completed in that unit and approved by the Engineer. Install verification test nails using the same equipment, methods, nail inclination and drillhole diameter as planned for the production nails. Perform pre-production verification tests in accordance with the Verification Testing Section prior to starting wall excavation and prior to installation of production nails in the specific lift in which the designated verification test nails are located. The number and location of the verification tests will be as indicated on the Plans or specified herein. Verification test nails may be installed through either the existing slope face prior to start of wall excavation, drill platform work bench, stabilization berm or into slot cuts made for the particular lift in which the verification test nails are located. Slot cuts will only be large enough to safely accommodate the drill and test nail reaction setup. Subject to the Engineer's approval, verification test nails may also be installed at angle orientations other than perpendicular to the wall face or at different locations than specified, as long as the Contractor can demonstrate that the test nails will be bonded into ground which is representative of the ground at the verification test nail locations designated on the Plans or herein. Install the production soil nails before the application of the reinforced shotcrete facing. At the Contractor's request and subject to the Engineer's written approval, the shotcrete facing may be placed before drilling and installing the nails. Provide a blockout through the shotcrete facing at drillhole locations using PVC pipe or other suitable material, to prevent damage to the facing during drilling. As part of the required construction submittals, provide the Engineer with acceptable structural design calculations demonstrating that the facing structural capacity will not be reduced and that the bearing plates are adequate to span the nail drillhole blockout through the construction facing. If this requires larger size bearing plates and/or additional reinforcement beyond that detailed on the Plans, the extra cost will be incidental.

Where necessary for stability of the excavation face, the Contractor shall have the option of placing a sealing layer (flashcoat) of unreinforced shotcrete or steel fiber reinforced shotcrete or of drilling and grouting of nails through a temporary stabilizing berm of native soil to protect and stabilize the face of the excavation per Section 3.2.3 Wall Structure Excavation. Cost shall be incidental to the Work.

The Engineer may add, eliminate, or relocate nails to accommodate actual field conditions. Cost adjustments associated with these modifications shall be made in accordance with the General Provisions of the Contract. The cost of any redesign, additional material, or installation modifications resulting from actions of the Contractor shall be borne by the Contractor.

3.3.1 Drilling. The drill holes for the soil nails shall be made at the locations, orientations, and lengths shown on the Plans or as directed by the Engineer. Select drilling equipment and methods suitable for the ground conditions described in the geotechnical report and shown in the boring logs. Select drillhole diameter(s) required to develop the specified pullout resistance and to also provide a minimum 25 mm grout cover over bare or epoxy coated bars or minimum 12 mm grout cover over the encapsulation of encapsulated nails. A minimum required drillhole diameter is shown on the plans. ***(See Commentary 3.3.1.a).*** It is the Contractor's responsibility to determine the final drillhole diameter(s) required to provide the specified pullout resistance. Use of drilling muds such as bentonite slurry to assist in drill cutting removal is not allowed but air may be used. With the Engineer's approval, the Contractor may be allowed to use water or foam flushing upon successful demonstration, at the Contractor's cost, that the installation method still provides adequate nail pullout resistance. ***(See Commentary 3.3.1.b).*** If caving ground is encountered, use cased drilling methods to support the sides of the drillholes. Where hard drilling conditions such as rock, cobbles, boulders, or obstructions are described elsewhere in the contract documents or project Geotechnical Report, percussion or other suitable drilling equipment capable of drilling and maintaining stable drillholes through such materials, will be used.

Immediately suspend or modify drilling operations if ground subsidence is observed, if the soil nail wall is adversely affected, or if adjacent structures are damaged from the drilling operation. Immediately stabilize the adverse conditions at no additional cost.

3.3.2 Nail Bar Installation. Provide nail bars in accordance with the schedules included in the Plans. Provide centralizers sized to position the bar within 25 mm of the center of the drillhole. Position centralizers as shown on the Plans so their maximum center-to-center spacing does not exceed 3 meters. Also locate centralizers within 0.6 meters from the top and bottom of the drillhole. Securely attach centralizers to the bar so they will not shift during handling or insertion into the drill hole yet will still allow grout tremie pipe insertion to the bottom of drillhole and allow grout to flow freely up the hole.

Inspect each nail bar before installation and repair or replace damaged bars or corrosion protection. Check uncased drillholes for cleanliness prior to insertion of the soil nail bar. Insert nail bars with centralizers into the drill hole to the required length without difficulty and in a way that prevents damage to the drill hole, bar, or corrosion protection. Do not drive or force partially inserted soil nails into the hole. Remove nails which cannot be fully inserted to the design depth and clean the drill hole to allow unobstructed installation.

When using cased or hollow stem auger drilling equipment which does not allow for the centralizers to pass through the casing or auger stem, the Contractor may delete the centralizers if the neat cement grout pumped through the casing is placed using grout pressures greater than 1 MPa or if the sand-cement grout placed through the stem of the auger has a slump of 225 mm or less. ***(See Commentary 3.3.2)***

3.3.3 Nail Installation Tolerances. Nails shall not extend beyond the right-of-way or easement limits shown on the Plans. Nail location and orientation tolerances are:

Nail head location, deviation from plan design location; 150 mm any direction.
Nail inclination, deviation from plan; + or - 3 degrees.
Location tolerances are applicable to only one nail and not accumulative over large wall areas. Center nail bars within 25 mm of the center of the drillhole. **(See Commentary 3.3.3)**

Soil nails which do not satisfy the specified tolerances, due to the Contractor's installation methods, will be replaced at no additional cost. Backfill abandoned nail drill holes with tremied grout. Nails which encounter unanticipated obstructions during drilling shall be relocated, as approved by the Engineer. Cost of drilling and backfilling drillholes abandoned due to unanticipated obstructions will be paid as Extra Work

3.4 Grouting

3.4.1 Grout Mix Design. Use a neat cement grout or a sand-cement grout. Submit the proposed nail grout mix design to the Engineer for review and approval in accordance with the submittal section.

The design mix submittal shall include compressive strength test results verifying that the proposed mix will have a minimum 3-day compressive strength of 10.5 Mpa and minimum 28-day compressive strength of 21 MPa.

3.4.2 Grout Testing. Previous test results for the proposed grout mix completed within one year of the start of work may be submitted for initial verification of the required compressive strengths for installation of pre-production verification test nails and initial production nails. During production, nail grout shall be tested by the Contractor in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one test for every 40 cubic meters of grout placed. Provide grout cube test results to the Engineer within 24 hours of testing.

3.4.3 Grouting Equipment. Grout equipment shall produce a uniformly mixed grout free of lumps and undispersed cement, and be capable of continuously agitating the mix. Use a positive displacement grout pump equipped with a pressure gauge which can measure at least twice but no more than three times the intended grout pressure. Size the grouting equipment to enable the entire nail to be grouted in one continuous operation. ***(See Commentary 3.4.3).*** Place the grout within 60 minutes after mixing or within the time recommended by the admixture manufacturer, if admixtures are used. Grout not placed in the allowed time limit will be rejected.

3.4.4 Grouting Methods. Grout the drillhole after installation of the nail bar. Each drillhole will be grouted within 2 hours of completion of drilling, unless otherwise approved by the Engineer. Inject the grout at the lowest point of each drill hole through a grout tube, casing, hollow-stem auger, or drill rods. Keep the outlet end of the conduit delivering the grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. Completely fill the drillhole in one continuous operation. Cold joints in the grout column are not allowed except at the top of the test bond length of proof tested production nails. ***(See Commentary 3.4.4).*** At the Contractor's option, the grout tube may remain in the hole provided it is filled with grout. Grouting before insertion of the nail is allowed provided the nail bar is immediately inserted through the grout to the specified length without difficulty.

During casing removal for drillholes advanced by either cased or hollow-stem auger methods, maintain sufficient grout level within the casing to offset the external groundwater/soil pressure and prevent hole caving. Maintain grout head or grout pressures sufficient to ensure that the drillhole will be completely filled with grout and to prevent unstable soil or groundwater from contaminating or diluting the grout. Record the grout pressures for soil nails installed using pressure grouting techniques. Control grout pressures to prevent excessive ground heave or fracturing.

Remove the grout and nail if grouting is suspended for more than 30 minutes or does not satisfy the requirements of this specification or the Plans, and replace with fresh grout and undamaged nail bar at no additional cost.

3.5 Nail Testing. ***(See Commentary 3.5).*** Perform both verification and proof testing of designated test nails. Perform pre-production verification tests on sacrificial test nails at locations shown on the

Plans or listed herein. Perform proof tests on production nails at locations selected by the Engineer. Required nail test data shall be recorded by the Engineer. Do not perform nail testing until the nail grout and shotcrete facing have cured for at least 72 hours and attained at least their specified 3-day compressive strength. Testing in less than 72 hours will only be allowed if the Contractor submits compressive strength test results, for tests performed by a qualified independent testing lab, verifying that the nail grout and shotcrete mixes being used will provide the specified 3-day compressive strengths in the lesser time.

3.5.1 Proof Test Nail Unbonded Length. Provide temporary unbonded lengths for each test nail. Isolate the test nail bar from the shotcrete facing and/or the reaction frame used during testing. Isolation of a test nail through the shotcrete facing shall not affect the location of the reinforcing steel under the bearing plate. Accepted proof test nails may be incorporated as production nails provided the temporary test unbonded length is fully grouted subsequent to testing. Submit the proposed test nail isolation methods, methods for providing an unbonded test length and methods for grouting the unbonded length subsequent to testing to the Engineer for review and approval in accordance with the Submittals section. Where temporary casing of the unbonded length of test nails is provided, install the casing in a way that prevents any reaction between the casing and the grouted bond length of the nail and/or the stressing apparatus.

3.5.2 Testing Equipment. Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The load cell is required only for the creep test portion of the verification test. Provide description of test setup and jack, pressure guage and load cell calibration curves in accordance with Submittals section.

Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. If the reaction frame will bear directly on the shotcrete facing, design it to prevent cracking of the shotcrete. Independently support and center the jack over the nail bar so that the bar does not carry the weight of the testing equipment. Align the jack, bearing plates, and stressing anchorage with the bar such that unloading and repositioning of the equipment will not be required during the test.

Apply and measure the test load with a hydraulic jack and pressure guage. The pressure guage shall be graduated in 500 kPa increments or less. The jack and pressure guage shall have a pressure range not exceeding twice the anticipated maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the nail load during verification tests with both the pressure gauge and the load cell. Use the load cell to maintain constant load hold during the creep test load hold increment of the verification test.

Measure the nail head movement with a dial gauge capable of measuring to 0.025 mm. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the nail and support the gauge independently from the jack, wall or reaction frame. Use two dial gauges when the test setup requires reaction against a soil cut face. ***(See Commentary 3.5.2)***

3.5.3 Pre-production Verification Testing of Sacrificial Test Nails. Pre-production verification testing shall be performed prior to installation of production nails to verify the Contractor's installation methods and nail pullout resistance. Perform pre-production verification tests at the locations and elevations shown on the Plans or herein and per Nail Installation Section 3.3, unless otherwise approved by the Engineer. Perform a minimum of 2 verification tests in each different soil/rock unit and for each different drilling/grouting method proposed to be used, at each wall location. Verification test nails will be sacrificial and not incorporated as production nails. Bare bars can be used for the sacrificial verification test nails.

Develop and submit the details of the verification testing arrangement including the method of distributing test load pressures to the excavation surface (reaction frame), test nail bar size, grouted drillhole diameter and reaction frame dimensioning to the Engineer for approval in accordance with Submittals section. Construct verification test nails using the same equipment, installation methods, nail inclination, and drillhole diameter as planned for the production nails. Changes in the drilling or installation method may require additional verification testing as determined by the Engineer and shall be provided at no additional cost. Payment for additional verification tests required due to differing site conditions, if determined by the Engineer, shall be per the contract unit price.

Test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 1 meter. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 3 meters. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 420 and Grade 520 bars, or 80 percent of the ultimate strength for Grade 1035 bars.The Contractor shall provide larger verification test bar sizes, if required to safely accommodate the 3 meter minimum test bond length and testing to 2 times the allowable pullout resistance requirements, at no additional cost.

The verification test bonded length L_{BV} shall not exceed the test allowable bar structural load divided by 2 times the allowable pullout resistance value. The following equation shall be used for determining the verification test nail maximum bonded length to be used to avoid structurally overstressing the verification test nail bar size:

$L_{BV} = C\, f_Y\, A_S / 2\, Q_d$, or 3 meters, whichever is greater.

L_{BV} = Maximum Verification Test Nail Bonded Length (m)
C = 0.9 for Grade 420 and 520 bars and 0.8 for Grade 1035 bars
f_Y = Bar Yield or Ultimate Stress (kN/m^2)
(Note: f_Y = 420,000 kN/m^2, 520,000 kN/m^2 and 1,035,000 kN/m^2 respectively for Grade 420, 520 and 1035 bars)
A_S = Bar Steel Area (m^2)
2 = Pullout resistance safety factor
Q_d = Allowable pullout resistance (kN/m, kilonewtons per lineal meter of grouted nail length, specified herein or on the Plans)

The Design Test Load (DTL) during verification testing shall be determined by the following equation:

DTL = Design Test Load (kN) = L_{BV} x Q_d
L_{BV} = As-built bonded test length (m)
Q_d = Allowable pullout resistance (kN/m, kilonewtons per lineal meter of grouted nail length, specified herein or on the Plans)
MTL = 2.0 x DTL = Maximum Test Load (kN)

Verification test nails shall be incrementally loaded to a maximum test load of 200 percent of the Design Test Load (DTL) in accordance with the following loading schedule. The soil nail movements shall be recorded at each load increment.

VERIFICATION TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	1 minute
0.25 DTL	10 minutes
0.50 DTL	10 minutes
0.75 DTL	10 minutes
1.00 DTL	10 minutes
1.25 DTL	10 minutes
1.50 DTL (Creep Test)	60 minutes
1.75 DTL	10 minutes
2.00 DTL(Max.Test Load)	10 minutes

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

Each load increment shall be held for at least 10 minutes. The verification test nail shall be monitored for creep at the 1.50 DTL load increment. Nail movements during the creep portion of the test shall be measured and recorded at 1 minute, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minutes. The load during the creep test shall be maintained within 2 percent of the intended load by use of the load cell.

3.5.4 Proof Testing of Production Nails. Perform proof testing on 5 percent (1 in 20) of the production nails in each nail row or minimum of 1 per row. The locations shall be designated by the Engineer. A verification test nail successfully completed during production work shall be considered equivalent to a proof test nail and shall be accounted for in determining the number of proof tests required in that particular row.

Production proof test nails shall have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail shall be at least 1 meter. The bonded length of the test nail shall be determined based on the production nail bar grade and size such that the allowable bar structural load is not exceeded during testing, but shall not be less than 3 meters. Production proof test nails shorter than 4 meters in length may be constructed with less than the minimum 3 meter bond length with the unbonded length limited to 1 meter. The allowable bar structural load during testing shall not be greater than 90 percent of the yield strength for Grade 420 and Grade 520 bars, or 80 percent of the ultimate strength for Grade 1035 bars.

The proof test bonded length L_{BP} shall not exceed the test allowable bar load divided by 1.5 times the allowable pullout resistance value, or above minimum lengths, whichever is greater. The following equation shall be used for sizing the proof test nail bonded length to avoid overstressing the production nail bar size:

$L_{BP} = C\, f_Y\, A_S / 1.5\, Q_d$, or above minimum lengths, whichever is greater.

L_{BP} = Maximum Proof Test Nail Bonded Length (m)
C = 0.9 for Grade 420 and 520 bars and 0.8 for Grade 1035 bars
f_Y = Bar Yield or Ultimate Stress (kN/m^2)
(Note: f_Y = 420,000 kN/m^2, 520,000 kN/m^2 and 1,035,000 kN/m^2 respectively for Grade 420, 520 and 1035 bars)
A_S = Bar Steel Area (m^2)
1.5 = Pullout resistance safety factor
Q_d = Allowable pullout resistance (kN/m, kilonewtons per lineal meter of grouted nail length, specified herein or on the Plans)

The Design Test Load (DTL) during proof testing shall be determined by the following equation:

DTL = Design Test Load (kN) = $L_{BP} \times Q_d$
L_{BP} = As-built bonded test length (m)
Q_d = Allowable pullout resistance (kN/m, kilonewtons per lineal meter of grouted nail length, specified herein or on the Plans)
MTL = 1.5 x DTL = Maximum Test Load (kN)

Proof tests shall be performed by incrementally loading the proof test nail to a maximum test load of 150 percent of the Design Test Load (DTL). The nail movement at each load shall be measured and recorded by the Engineer in the same manner as for verification tests. The test load shall be monitored by a jack pressure gauge with a sensitivity and range meeting the requirements of pressure gauges used for verification test nails. At load increments other than maximum test load, the load shall be held long enough to obtain a stable reading. Incremental loading for proof tests shall be in accordance with the following loading schedule. The soil nail movements shall be recorded at each load increment.

PROOF TEST LOADING SCHEDULE

LOAD	HOLD TIME
AL (.05 DTL max.)	Until Stable
0.25 DTL	Until Stable
0.50 DTL	Until Stable
0.75 DTL	Until Stable
1.00 DTL	Until Stable
1.25 DTL	Until Stable
1.50 DTL (Max. Test Load)	See Below

The alignment load (AL) should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the Design Test Load (DTL). Dial gauges should be set to "zero" after the alignment load has been applied.

All load increments shall be maintained within 5 percent of the intended load. Depending on performance, either 10 minute or 60 minute creep tests shall be performed at the maximum test load (1.50 DTL). The creep period shall start as soon as the maximum test load is applied and the nail movement shall be measured and recorded at 1 minutes, 2, 3, 5, 6, and 10 minutes. Where the nail movement between 1 minute and 10 minutes exceeds 1 mm, the maximum test load shall be maintained an additional 50 minutes and movements shall be recorded at 20 minutes, 30, 50, and 60 minutes.
(See Commentary 3.5.4)

3.5.5 Test Nail Acceptance Criteria. A test nail shall be considered acceptable when:

1. For verification tests, a total creep movement of less than 2 mm per log cycle of time between the 6 and 60 minute readings is measured during creep testing and the creep rate is linear or decreasing throughout the creep test load hold period.

2. For proof tests, a total creep movement of less than 1 mm is measured between the 1 and 10 minute readings or a total creep movement of less than 2 mm is measured between the 6 and 60 minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

3. The total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length. ***(See Commentary 3.5.5)***

4. A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

Successful proof tested nails meeting the above test acceptance criteria may be incorporated as the test nail. The pullout failure load shall be recorded as part of the test data.

Successful proof tested nails meeting the above test acceptance criteria may be incorporated as production nails, provided that (1) the unbonded length of the test nail drillhole has not collapsed during testing, (2) the minimum required drillhole diameter has been maintained, (3) the specified corrosion protection is provided, and (4) the test nail length is equal to or greater than the scheduled production nail length. Test nails meeting these requirements shall be completed by satisfactorily grouting up the unbonded test length. Maintaining the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails cannot be satisfactorily grouted subsequent to testing, the proof test nail shall become sacrificial and shall be replaced with an additional production nail installed at no additional cost.

3.6 Test Nail Rejection

If a test nail does not satisfy the acceptance criterion, the Contractor shall determine the cause.

3.6.1 Verification Test Nails. The Engineer will evaluate the results of each verification test. Installation methods which do not satisfy the nail testing requirements shall be rejected. The Contractor shall propose alternative methods and install replacement verification test nails. Replacement test nails shall be installed and tested at no additional cost.

3.6.2 Proof Test Nails. The Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test nail and the adjacent passing proof test nail. Alternatively, the Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient load carrying capacity. Contractor modifications may include, but are not limited to; the installation of additional proof test nails; increasing the drillhole diameter to provide increased capacity; modifying the installation or grouting methods; reducing the production nail spacing from that shown on the Plans and installing more production nails at a reduced capacity; or installing longer production nails if sufficient right-of way is available and the pullout capacity behind the failure surface controls the allowable nail design capacity. The nails may not be lengthened beyond the temporary construction easements or the permanent right-of-way shown on the Plans. Installation and testing of additional proof test nails or installation of additional or modified nails as a result of proof test nail failure(s) will be at no additional cost.

3.7 Nail Installation Records. Records documenting the soil nail wall construction will be maintained by the Engineer, unless specified otherwise. ***(See Commentary 3.7)***. The Contractor shall provide the Engineer with as-built drawings showing as-built nail locations and as-built shotcrete facing line and grade within 5 days after completion of the shotcrete facing and as-built CIP facing line and grade within 5 days after completion of the CIP facing.

4.0 METHOD OF MEASUREMENT. The unit of measurement for production soil nails will be per lineal meter. The length to be paid will be the length measured along the bar centerline from the back face of shotcrete to the bottom tip end of nail bar as shown on the Plans. No separate measurement will be made for proof test nails, which shall be considered incidental to production nail installation. Specified verification test nails will be measured on a unit basis for each verification test successfully completed. Failed verification test nails or additional verification test nails installed to verify alternative nail installation methods proposed by the Contractor will not be measured.

Structure Excavation for Soil Nail Wall will be measured as the theoretical plan volume in cubic meters within the structure excavation pay limits shown on the Plans. This will be the excavation volume within the zone measured from top to bottom of shotcrete wall facing and extending out 2 meters horizontally in front of the plan wall final excavation line. Additional excavation beyond the plan wall final excavation line resulting from irregularities in the cut face, excavation overbreak or inadvertent excavation, will not be measured. ***(See Commentary 4.0)***

General roadway excavation will not be a separate wall pay item but will be measured and paid as part of the general roadway excavation including haul pay item.

The final pay quantities will be the design quantity increased or decreased by any changes authorized by the Engineer.

5.0 BASIS OF PAYMENT. The accepted quantities of soil nails and soil nail wall structure excavation will be paid for at the contract unit prices. Payment will be full compensation for all labor, equipment, materials, material tests, field tests and incidentals necessary to acceptably fabricate and construct the soil nails and perform the structure excavation, including the excavation and wall alignment survey control, for the soil nail wall(s) in accordance with all requirements of the contract. Payment will be made for each of the following bid items included in the bid form:

Pay Item	Measurement Unit
Permanent Soil Nails. No._ Bar (Grade _)	Lineal Meters
Verification Test Nails	Each
Structure Excavation-Soil Nail Wall	Cubic Meters

END OF SECTION

APPENDB1.SI

COMMENTARY TO GUIDE SPECIFICATION FOR PERMANENT SOIL NAILS AND WALL EXCAVATION (OWNER-DESIGN)

1.0 DESCRIPTION

1.0 Specifier. The most common source of construction errors arise because the excavation subcontractor is unfamiliar with the staged excavation requirements necessary during soil nail wall construction. Excavators unfamiliar with soil nailing may overexcavate the first or subsequent lifts. This may result in hauling berm material back to the site for the nail installation subcontractor's working berm or even in failure of the wall. To reduce the likelihood of these problems the Specifier should add the following paragraph to the earthworks or roadway excavation section of the Special Provisions to direct the excavation contractor's attention to the soil nail wall specification. Preferably, the excavators movements in the vicinity of the wall should be controlled by the soil nail installation subcontractor:

> Soil nail wall construction requires excavation in staged lifts and the wall excavation Contractor should consult the Soil Nail and Shotcrete Facing Specifications for details. A pre-construction meeting shall be held prior to the start of wall construction. The Engineer, prime Contractor, excavation Contractor, and soil nail specialty Contractor shall attend the meeting. The pre-construction meeting will be conducted to clarify the construction requirements for the work, to coordinate the construction schedule and activities, and to identify contractual relationships and delineation of responsibilities - particularly those pertaining to wall excavation, nail installation and testing, excavation and wall alignment survey control, and shotcrete and CIP facing construction. Soil nail wall construction requires excavation in staged lifts and excavation in the vicinity of the wall face requires special care and effort compared to general earthwork excavation. The excavation Contractor should take this into account during bidding and should consult the Wall Excavation and Measurement/Payment Sections of the Soil Nail and Shotcrete Facing Specification prior to bidding.

1.2 Specifier. The location of both active and abandoned buried utilities within the ground mass to receive soil nail reinforcement can have a profound impact on the design and construction of soil nail walls. Careful consideration of the presence and location of all utilities is required for successful design and installation of soil nails.

1.3 Specifier. The appropriate portion of the following paragraph can be included as an additional submittal requirement where utilities are present:

> 9. Detailed preliminary action plan for precluding disruption of buried utilities where identified on the Plans or in the geotechnical report.

2.0 MATERIALS

2 a. Specifier. Corrosion protection requirements vary between Transportation Agencies. The most common and simplest tests utilized to measure the aggressiveness of the soil environment include electrical resistivity, pH, chloride, and sulfate. Per FHWA-RD-89-198, the ground is considered aggressive if any one of these indicators show critical values as detailed below:

PROPERTY	TEST DESIGNATION*	CRITICAL VALUES*
Resistivity	AASHTO T-288 ASTM G 57	below 2,000 ohm-cm
pH	AASHTO T-289 ASTM G 51	below 5
Sulfate	AASHTO T-290 ASTM D516M, ASTM D4327	above 200ppm
Chloride	AASHTO T-291 ASTM D512, ASTM D4327	above 100 ppm

* Specifier should check test standards for latest updates and individual transportation agencies may have limits on critical values different than tabulated above.

2b. Specifier. Standard specifications or test methods for any of the above items which are common to your agency can be referenced in lieu of the above listed AASHTO/ASTM references.

2 c. Specifier. A 3 day strength criterion has been selected in addition to the traditional 28 days because soil nailing grout and shotcrete are normally required to accept loads at an early age and therefore it is the early strength that is critical.

3.0 CONSTRUCTION REQUIREMENTS

3.1a Specifier. Existing drainage features or future permanent drainage features should not be integrated with the wall drainage network indicated on the Plans nor should the wall drainage network be expressly relied upon to service flows from such drainage features, without written authorization from the Engineer.

3.1b Specifier. Review the subsurface investigation data to confirm that the reference to the groundwater table being below the base of the excavation is correct. If not, discuss with the wall designer/geotechnical engineer and revise the wording and/or the design as deemed necessary. Soil *nail wall construction can be very difficult and costly below the water table and is generally considered a nonapplication unless the site can be effectively dewatered prior to excavation.*

3.2.2 Specifier. Typically, a minimum 5 meter wide working bench is required for adequate drill rig access. For sliver type road cuts, a 5 meter wide working bench may not be available in the upper excavation rows using the excavated material. In these instances, it may be necessary to import additional fill material for construction of the initial working benches. Alternatively, some projects have been successfully completed using crane mounted drill equipment for the installation of the upper nail rows. It is the Contractor's responsibility to assess equipment and access needs and account for such in the bid.

3.2.3 Specifier. Experience has shown that the use of a temporary stabilizing berm can be beneficial to protect and buffer the final excavation face, particularly in the upper rows of nails, since this is the area where loose, sloughing soils are most often encountered. A "stabilizing berm" is simply a wedge of soil which is temporarily left in front of the final wall excavation face to provide a buffer for the soils at the final cut face. Nail holes are drilled and nails installed and grouted through the berm. Following nail installation, the stabilizing berm is excavated to expose the soils at the final wall excavation face. The purpose of the berm is to reduce the potential for face sloughing and raveling, and ground loss and loosening of the bearing soil below the nails caused by drilling vibrations and drill cuttings removal. Secondarily, the stabilizing berm will reduce loss of natural moisture from the soil exposed at the final excavation face. For example, in predominantly granular soils, natural moisture will provide some "apparent" cohesion and will help provide soil standup. In dense competent ground and in more cohesive soils, a stabilizing berm is usually not needed.

3.3.1.a Specifier. For nails installed in soil, a 150 mm drillhole diameter is commonly being used. In stronger, competent ground such as bedrock, weathered rock and stiff residual soils, smaller 100 mm diameter drillholes have been successfully used. In these types of ground, the designer can specify or the Contractor can be allowed to reduce the minimum drillhole diameter to 100 mm, with the performance requirement that the required pullout resistance and specified minimum grout cover must still be provided.

3.3.1.b Specifier. The use of drilling muds, water or drilling foams to assist in drill cuttings removal is not generally recommended due to the potential for decreasing the ground to grout bond along the drillhole walls, resulting in lower soil nail pullout resistance. In some ground, the use of water or drilling foams may improve drillability and/or hole cleaning. Therefore, the Contractor may be allowed to use water or foam flushing upon successful demonstration, at the Contractor's cost, that the installation method still provides adequate nail pullout resistance. Use of bentonite drilling slurries are not recommended for drilling soil nail drillholes in any type of ground.

3.3.2 Specifier. Where the deletion of centralizers is allowed due to use of cased or hollow stem auger installations which do not allow for the centralizers to pass through the casing, the Contractor must still ensure a centralized nail at the wall excavation face. This is required since the nail head and surrounding grout column help provide bearing support for some of the shotcrete *facing weight during construction. Grouting techniques utilizing grout pressures greater than 1 MPa or stiff sand-cement grout (slump not greater than 225 mm) are necessary to ensure that the nail bar is supported in the position that it exited the drill casing or the auger stem and does not settle to the bottom of the drillhole.*

3.3.3 *Specifier. The nail head and surrounding grout column help provide bearing support for some of the shotcrete facing weight during construction. Therefore, it is important for the nail bar to be reasonably centered in the nail grout column and not to lay on the bottom of the drillhole.*

3.4.3 Specifier. Where grout takes are anticipated to be high due to fractured rock or gravel, cobbles, boulders or other types of open voided coarse material, the following sentence can be tailored to suit the anticipated site condition and included:

> Neat cement grout takes may be high due to the potential for open voided coarse material. Alternative grouting methods including low slump/high viscosity sand-cement grout mixtures or neat cement grout contained in a geotextile sock encapsulation of the nail may be used, provided the specified nail pullout resistance is still successfully provided. Alternative proposed grouting methods shall be submitted to the Engineer for approval.

3.4.4 Specifier. Nail grout cold joints, other than for proof test nails, are not desirable because of the potential for groundwater migration along the cold joint and subsequent potential corrosion of the nail bar. This will not be as critical for epoxy coated or encapsulated bars.

3.5 Specifier. Soil nails are field tested to verify that the nail loads can be carried without excessive movements and with an adequate safety factor for the service life of the structure. In addition, testing is used to verify the adequacy of the Contractor's drilling, installation and grouting methods. Soil nail field pullout testing is critically important and should be viewed as an extension of design. Current Industry standard is to perform proof tests on 5 percent of total number of production nails and minimum of 1 each nail row. There currently is no Industry standard for number of pre-production verification tests. Recommended minimum guideline is at least 2 verification tests per different soil/rock unit or per different drilling/grouting method for each wall location.

3.5.2 Specifier. Experience with testing nails reacting against a soil cut face has resulted in racking and misalignment of the system on some projects. Two dial guages are recommended for this test setup to determine if racking is occurring and to provide a more accurate average nail head movement measurement.

3.5.4 Field Pullout Testing to Verify Pullout Resistance Values Used in Design

3.5.4 Specifier. This guide specification presents field pullout testing procedures compatible with a SLD design approach. Irrespective of the method of design employed (SLD or LRFD), field pullout testing should be performed using the same maximum test loads. For SLD, the allowable *pullout resistance is taken as one half of the estimated ultimate pullout resistance for Group 1 loading. Field testing consists of verification testing to 200 percent of the allowable pullout resistance (i.e., 100 percent of the ultimate pullout resistance used for design) and of proof testing to 150 percent of the allowable pullout resistance (i.e., 75 percent of the ultimate pullout resistance used for design).*

For LRFD, the design pullout resistance is taken as 70 percent of the ultimate pullout resistance for Strength Limit State loading. The design pullout resistance of LRFD is therefore higher than the allowable pullout resistance of SLD, and reflects the load factors applied with the LRFD approach. Nevertheless, for similar reliability of the soil nail wall design using the two design approaches, similar maximum pullout resistance values should be achieved in the field. Therefore, irrespective of the design approach used, i.e. SLD or LRFD, it is recommended that field verification testing be performed to 100 percent of the ultimate pullout resistance value used in the design (conventionally specified as 200 percent of the allowable pullout resistance where the allowable pullout resistance is one half of the ultimate pullout resistance), and field proof testing be performed to 75 percent of the ultimate pullout resistance value used in the design (conventionally specified as 150 percent of allowable).

3.5.5 Specifier. The minimum acceptable elastic movement is computed as follows:

measured movement, DL, acceptable if $> 0.8(P)(UL)(10^6)/(A_S)(E)$

where measured movement DL is in mm and:

P = *Maximum applied test load (kN)*
UL = *Length from the back of the nail to jack connection to the top of the bond*
A_S = *Cross-sectional area of the soil nail bar steel* (mm^2)
E = *Young's modulus of steel (typically 200,000 MPa)*

3.7 Specifier. Typical records maintained by the Engineer should include the following:

- *a. Contractor's name*
- *b. Drill rig operator's name*
- *c. Date and time of start and finish of drilling*
- *d. Drilling difficulties*
- *e. Caving or sloughing of excavation or drillhole*
- *f. Groundwater conditions*
- *g. Drill casing requirements*
- *h. Installed nail drillhole and bar diameter*
- *i. Design nail length*
- *j. Installed nail length*
- *k. As-built nail location and deviation from specified tolerances*
- *l. Date, time and method grout was placed including grout pressure(if applicable)*
- *m. Design changes*

(See FHWA "Soil Nailing Field Inspector's Manual" for example forms for recording above information)

4.0 Specifier. The methods of excavation , e.g. dozer vs. backhoe, and amount of care and effort taken by the excavation Contractor in the vicinity of the wall face are beyond the Owner's control. It is better to indicate that overbreak is a possibility and place the responsibility for estimating and controlling overbreak on the Contractor.

END OF COMMENTARY

APPENDB1.SI

APPENDIX B2

FHWA GUIDE SPECIFICATION FOR SOIL NAIL RETAINING WALL (DESIGN-BUILD SOLICITATION)

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDB2.SI
Last Revision 10/31/98

(An electronic version of this section is available at www.fhwa.dot.gov/bridge**)**

FHWA GUIDE SPECIFICATION FOR SOIL NAIL RETAINING WALL (DESIGN-BUILD SOLICITATION)

(Commentary: This guide specification is set up for a post-bid design solicitation to solicit soil nail wall designs where the Owner has selected a soil nail wall as the desired wall type for the given wall location(s). It can be modified as appropriate to also serve as a pre-bid design solicitation and /or for a solicitation where alternate wall types are allowed by the Owner, with the Contractor allowed to select and submit a design for the wall type which the Contractor feels is most cost-effective.)

1.0 DESCRIPTION. This work consists of designing and constructing permanent soil nail retaining wall(s) at the locations shown on the "Layout Drawings". The Contractor shall furnish all labor, plans, drawings, design calculations, or other materials and equipment required to design and construct the soil nail wall(s) in accordance with this Specification and the ______ Standard Specifications. *(Commentary: If walls are temporary rather than permanent, revise above wording.)*

Where the imperative mood is used within this Specification for conciseness, "the Contractor shall" is implied.

1.1 Contractor's Experience Requirements. The Contractor shall be experienced in the construction of permanent soil nail retaining walls and have successfully constructed at least 3 projects in the last 3 years involving construction of permanent soil nail retaining walls totaling at least 1000 square meters of wall face area and at least 500 permanent soil nails.

A Professional Engineer employed by the soil nailing Contractor and having experience in the construction of at least 3 completed permanent soil nail retaining wall projects over the past 3 years, shall supervise the work. The Contractor shall not use consultants or manufacturers' representatives to satisfy the supervising Engineer requirements of this section.

The soil nail wall shall be designed by a Registered Professional Engineer with experience in the design of at least 3 successfully completed permanent soil nail retaining wall projects over the past 3 years. The wall designer may be either a employee of the Contractor or a separate Consultant designer meeting the stated experience requirements.

At least 45 calendar days before the planned start of wall excavation, the Contractor shall submit the experience qualifications and details for the referenced design and construction projects, including a brief project description with the owner's name and current phone number. Upon receipt of the experience qualifications submittal, the Engineer will have 15 calendar days to approve or reject the proposed soil nailing Contractor and Designer.

1.2 Pre-Approval List

The following soil nailing design-build specialty Contractors are pre-approved:

1. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

2. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

3. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

1.3 Available Information. Available information developed by the Department or by the Department's duly authorized representative include the following items:

1. "*Layout Drawings*" prepared by _______ , dated ______. The "*Layout Drawings*" include the approved preliminary plans, profile and typical cross sections for the proposed soil nail wall locations. *(Commentary- Specifier: Refer to chapter 8 of the FHWA "Manual for Design and Construction Monitoring of Soil Nail Walls", Report No. FHWA- SA-96-069 for detailed guidance on conceptual plan information to provide on the Design-Build "Layout Drawings")*

2. Geotechnical Report No. _____ Titled ______, dated______ , included in the bid documents, contains the results of test pits, exploratory borings and other site investigation data obtained vicinity the proposed wall locations.

1.4 Soil Nail Wall Design Requirements. Design the soil nail walls using the Service Load Design (SLD) procedures contained in the FHWA "Manual for Design and Construction Monitoring of Soil Nail Walls", Report No. FHWA- SA-96-069. The required partial safety factors, allowable strength factors and minimum global stability soil factors of safety shall be in accord with the FHWA manual, unless specified otherwise. Estimated soil/rock design shear strength parameters, slope and external surcharge loads, type of wall facing and facing architectural requirements, soil nail corrosion protection requirements, known utility locations, easements, and right-of-ways will be as shown on the "*Layout Drawings*" or specified herein. Structural design of any individual wall elements not covered in the FHWA manual shall be by the service load or load factor design methods in conformance with Article 3.22 and other appropriate articles of the 15th Edition of the AASHTO

Standard Specifications for Highway Bridges including current interim specifications. The seismic design acceleration coefficient is ______ g.

(Commentary: The FHWA manual also presents LRFD design procedures for soil nail walls. Revise specification if LRFD design is required. The FHWA manual also sets forth different recommended global stability soil partial safety factors/soil resistance factors, for SLD/LRFD respectively, for non-critical and critical (e.g. bridge end slope removal application in front of an existing bridge abutment) structures and temporary during construction conditions. Specify which to use.)

1.5 Soil Nail Wall Design Submittals. At least 30 calendar days before the planned start of wall excavation, submit complete design calculations and working drawings to the Engineer for review and approval. Include all details, dimensions, quantities, ground profiles, and cross-sections necessary to construct the wall. Verify the limits of the wall and ground survey data before preparing drawings.

1.5.1 Design Calculations. Design calculations shall include, but not be limited to, the following items:

(1) A written summary report which describes the overall soil nail wall design.

(2) Applicable code requirements and design references.

(3) Nail wall critical design cross-section(s) geometry including soil/rock strata and location, magnitude, and direction of design slope or external surcharge loads and piezometric levels.

(4) Design criteria including, soil/rock shear strengths (friction angle and cohesion), unit weights, and ground-grout pullout resistances and nail drillhole diameter assumptions for each soil/rock strata.

(5) Partial safety factors/strength factors (for Service Load Design) or load and resistance factors (for Load and Resistance Factor Design) used in the design on the pullout resistance, surcharges, soil/rock unit weights, nail head strengths, and steel, shotcrete, and concrete materials. Minimum required global stability soil factor of safety for SLD design or minimum required global stability soil resistance/load ratio for LRFD design.

(6) Seismic design acceleration coefficient.

(7) Design calculation sheets with the project number, wall 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 any symbols and computer programs used in the design.

(9) Nail wall final design cross-section(s) geometry including soil/rock strata and location, magnitude, and direction of slope or external surcharge loads and piezometric levels with critical slip

surface shown along with minimum calculated Global stability soil factor of safety for SLD design or for minimum Global stability soil resistance/load ratio for LRFD design and required nail lengths and strengths (nail bar sizes and grades) for each nail row.

(10) Structural design calculations for wall facing(s) and nail head/facing connections including consideration of facing flexural and punching shear strength, headed studs tensile strength, upper cantilever, minimum reinforcement ratio, cover and splice requirements.

(11) Other design calculations.

1.5.2 Working Drawings. Working drawings shall include, but not be limited to, the following items:

(1) A plan view of the wall(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 of wall stations.

(d) 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.

(e) Limit of longest nails.

(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 wall.

(2) An elevation view of the wall(s) identifying:

(a) The elevation at the top of the wall, at all horizontal and vertical break points, and at least every 10 meters along the wall.

(b) Elevations at the wall base and the top of leveling pads for casting CIP facing (if applicable).

(c) Beginning and end of wall stations.

(d) The distance along the face of the wall to all steps in the wall base.

(e) Wall elevation view showing nail locations and elevations; vertical and horizontal nail spacing; and the location of wall drainage elements and permanent facing expansion/contraction joints (if applicaple) along the wall length.

(f) Existing and finish grade profiles both behind and in front of the wall.

(3) Design parameters and applicable codes.

(4) General notes for constructing the wall including construction sequencing or other special construction requirements.

(5) Horizontal and vertical curve data affecting the wall and wall control points. Match lines or other details to relate wall stationing to centerline stationing.

(6) A listing of the summary of quantities on the elevation drawing of each wall showing estimated square meters of wall face areas and other pay items.

(7) Nail wall typical sections including staged excavation lift elevations, wall and excavation face batter, nail spacing and inclination, nail bar sizes, and corrosion protection details.

(8) A typical detail of production and test nails defining the nail length, minimum drillhole diameter, inclination, and test nail bonded and unbonded test lengths.

(9) 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.

(10) Dimensions and schedules of all reinforcing steel including reinforcing bar bending details.

(11) Details and dimensions for wall appurtenances such as barriers, coping, drainage gutters, fences, etc.

(12) Details for constructing walls around drainage facilities.

(13) Details for terminating walls and adjacent slope construction.

(14) Facing finishes, color and architectural treatment requirements (if applicable) for permanent wall facing elements.

(Commentary: An example set of Soil Nail Wall Final Plans are included in Appendix A, Example Plan Details, in FHWA-SA-96-069).

The drawings and calculations shall be signed and sealed by the Contractor's Professional Engineer and by the Consultant designer's Professional Engineer (if applicable), previously approved by the

Owner's Engineer. If the soil nail Contractor uses a Consultant designer subcontractor to prepare the design, the soil nail Contractor shall still have overall contract responsibility for both the design and the construction.

Submit 3 sets of the wall drawings with the initial submission. One set will be returned with any indicated corrections. The Engineer will approve or reject the Contractor's submittals within 15 calendar days after receipt of a complete submission. If revisions are necessary, make the necessary corrections and resubmit 3 revised sets. When the drawings are approved, furnish 5 sets and a Mylar sepia set of the drawings. The Contractor will not be allowed to begin wall construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time will be allowed due to incomplete submittals.

Revise the drawings when plan dimensions are revised due to field conditions or for other reasons. Within 30 days after completion of the work, submit as-built drawings to the Engineer. Provide revised design calculations signed by the approved Registered Professional Engineer for all design changes made during the construction of the wall.

2.0 MATERIALS AND 3.0 CONSTRUCTION REQUIREMENTS. Construct the wall according to the approved drawings and the appropriate Sections in the Standard Specifications as applicable. Materials and construction requirements will be as set forth in the Permanent Soil Nail and Wall Excavation and Temporary Shotcrete Facing and Wall Drainage Specifications. All portions of these Specifications will apply except for Measurement and Payment, which will be as set forth below.

(Commentary: This solicitation is written assuming the Owner Agency will still provide construction inspection of the design-build construction. If not the case, and more responsibility for construction inspection and testing is to be placed on the Contractor, then the referenced construction specifications should be modified accordingly).

4.0 METHOD OF MEASUREMENT. The unit of measurement for soil nailed walls will be lump sum for each wall listed on the bid schedule. When plan dimension changes are authorized during construction to account for field conditions, the lump sum price of the wall will be adjusted by applying a calculated per square meter cost adjustment factor to the added or decreased wall front face area resulting from the change. The adjustment factor will be determined by dividing the lump sum price bid for each wall by its original shotcrete facing front face area shown on the original approved working drawings. If the actual quantity increases or decreases by more than ______ percent from the original plan quantity, as authorized by the Engineer, the contract price will be adjusted per subsection _______ of the Standard Specifications.

5.0 BASIS OF PAYMENT. Payment will be full compensation for all labor, equipment, materials, tests, and incidentals necessary to acceptably design and construct the soil nail wall including the wall drainage network and the temporary shotcrete construction facing or permanent shotcrete facing (if applicable).

Pay Item	Measurement Unit
Soil Nail Retaining Wall No. 1	Lump Sum
Soil Nail Retaining Wall No. 2	Lump Sum

If required, permanent CIP facings or CIP drainage gutters will be measured and paid for separately per Lump Sum, in accordance with Section _______ of the Standard Specifications or CIP Facing special provision.

APPENDB2.SI

APPENDIX C1

FHWA GUIDE SPECIFICATION
FOR
TEMPORARY SHOTCRETE CONSTRUCTION FACING AND WALL DRAINAGE
(OWNER-DESIGN)

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDC1.SI
Last Revision 10/31/98

(An electronic version of this section is available at www.fhwa.dot.gov/bridge**)**

FHWA GUIDE SPECIFICATION
FOR
TEMPORARY SHOTCRETE CONSTRUCTION FACING AND WALL DRAINAGE
(OWNER-DESIGN)

1.0 DESCRIPTION. Shotcrete facing and wall drainage work consists of furnishing all materials and labor required for placing and securing geocomposite drainage material, connection pipes, footing drains, weepholes and horizontal drains (if required), drainage gutter, reinforcing steel and shotcrete for the temporary shotcrete construction facing and nail head bearing plates and nuts for the soil nail walls shown on the Plans. The Work shall include any preparatory trimming and cleaning of soil/rock surfaces and shotcrete cold joints to receive new shotcrete.

Shotcrete shall comply with the requirements of ACI 506.2, "Specifications for Materials, Proportioning and Application of Shotcrete", except as otherwise specified. Shotcrete shall consist of an application of one or more layers of concrete conveyed through a hose and pneumatically projected at a high velocity against a prepared surface.

Shotcrete may be produced by either a wet-mix or dry-mix process. The wet-mix process consists of thoroughly mixing all the ingredients except accelerating admixtures, but including the mixing water, introducing the mixture into the delivery equipment and delivering it, by positive displacement, to the nozzle. The wet-mix shotcrete shall then be air jetted from the nozzle at high velocity onto the surface. The dry-mix process consists of shotcrete without mixing water which is conveyed through the hose pneumatically with the mixing water introduced at the nozzle. For additional descriptive information, the Contractor's attention is directed to the American Concrete Institute ACI 506R "Guide to Shotcrete."

CIP concrete facing construction (if required) is covered by the Standard Specifications and/or CIP Facing Special Provisions. Soil nails and wall excavation are covered by the Permanent Soil Nails and Wall Excavation Specification. Soil nail wall instrumentation (if required) is covered by the Soil Nail Wall Instrumentation Specification.

Where the imperative mood is used within this Specification for conciseness, "the Contractor shall" is implied.

1.1 Contractor's Experience Requirements. Workers, including foremen, nozzlemen, and delivery equipment operators, shall be fully experienced to perform the work. All shotcrete nozzlemen on this project shall have experience on at least 3 projects in the past 3 years in similar shotcrete application work and shall demonstrate ability to satisfactorily place the shotcrete.

Initial qualification of nozzlemen will be based either on previous ACI certification or satisfactory completion of preconstruction test panels. The requirement for nozzlemen to shoot preconstruction qualification test panels will be waived for nozzlemen who can submit documented proof they have been certified in accordance with the ACI 506.3R Guide to Certification of Shotcrete Nozzlemen. The Certification shall have been done by a ACI recognized shotcrete testing lab and/or recognized shotcreting consultant and have covered the type of shotcrete to be used (plain wet-mix, plain dry-mix or steel fiber reinforced). (***See Commentary 1.1).*** All nozzlemen will be required to periodically shoot production test panels during the course of the Work at the frequency specified herein.

Notify the Engineer not less than 2 days prior to the shooting of preconstruction test panels to be used to qualify nozzlemen without previous ACI certification. Use the same shotcrete mix and equipment to make qualification test panels as those to be used for the soil nail wall shotcrete facing. Initial qualification of the nozzlemen will be based on a visual inspection of the shotcrete density and void structure and on achieving the specified 3-day and 28-day compressive strength requirements determined from test specimens extracted from the preconstruction test panels. Preconstruction and production test panels, core extraction and compressive strength testing shall be conducted in accordance with ACI 506.2 and AASHTO T24/ASTM C42, unless otherwise specified herein. Nozzlemen without ACI Certification will be allowed to begin production shooting based on satisfactory completion of the preconstruction test panels and passing 3-day strength test requirements. Continued qualification will be subject to passing the 28-day strength tests and shooting satisfactory during production test panels.

1.2 Construction Submittals. At least 15 calendar days before the planned start of shotcrete placement or CIP facing placement (if applicable), submit 5 copies of the following information, in writing, to the Engineer for review:

a. Written documentation of the nozzlemen's qualifications including proof of ACI certification (if applicable).

b. Proposed methods of shotcrete placement and of controlling and maintaining facing alignment and location and shotcrete thickness.

c. Shotcrete mix design including:
Type of Portland cement.
Aggregate source and gradation.
Proportions of mix by weight and water-cement ratio.
Proposed admixtures, manufacturer, dosage, technical literature.
Previous strength test results for the proposed shotcrete mix completed within one year of the start of shotcreting may be submitted for initial verification of the required compressive strengths at start of production work.

d. Certificates of Compliance, manufacturers' engineering data and installation instructions for the drainage geotextile, geocomposite drain strip, drain grate and accessories.

e. Certificates of Compliance for bearing plates, nuts, drainage aggregate and PVC drain piping.

f. Formwork dimensions and details for casting the CIP facing over the shotcrete construction facing. Include details for formwork connections to the shotcrete facing and/or nails (if applicable), proposed concrete placement method and placement rates, and accompanying structural calculations verifying the structural adequacy of the formwork, connections, and shotcrete facing and/or nails to support the loading induced by the fliud CIP concrete. When anchors embedded into the shotcrete facing will be used to support the 1-sided CIP face form, include calculations illustrating the anchor design load (calculated as the design concrete fluid pressure times the anchor tributary area). The structural calculations shall be prepared and sealed by a Registered Professional Engineer proficient in structural design and licensed in the State of ______.

The Engineer will approve or reject the Contractor's submittals within 10 calendar days after receipt of a complete submission. The Contractor will not be allowed to begin wall construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time will be allowed due to incomplete submittals.

Upon delivery to the project site, provide Certified mill test results for all reinforcing steel specifying the minimum ultimate strength, yield strength, elongation and chemical composition.

1.3 Pre-Construction Meeting. A pre-construction meeting scheduled by the Engineer will be held prior to the start of wall construction. Attendance is mandatory. The shotcrete Contractor, if different than the soil nail specialty Contractor, shall attend. See Section 1.4 of the Permanent Soil Nail and Wall Excavation Specification.

2.0 MATERIALS. All materials for shotcrete shall conform to the following requirements:

Cement	AASHTO M85/ ASTM C150, Type I, II, III or V.
Fine Aggregate	AASHTO M6/ASTM C33 clean, natural.
Coarse Aggregate	AASHTO M80, Class B for quality
Water	Clean and Potable. AASHTO M157/ASTM C94
Chemical Admixtures	
Accelerator	Fluid type, applied at nozzle, meeting requirements of AASHTO M194/ASTM C494/ASTM C1141.
Water-reducer and	AASHTO M194/ASTM C494 Type A, C, D, E, F, or G

Superplastisizer	
Retarders	AASHTO M194/ ASTM C494 Type B or D.
Mineral Admixtures	
Fly Ash	AASHTO M295/ASTM C618 Type F or C, cement
Silica Fume	ASTM C1240, 90 percent minimum silicon dioxide solids content, not to exceed 12 percent by weight of cement.
Welded Wire Fabric	AASHTO M55/ASTM A185 or A497.

(See Commentary for Steel Fiber Reinforced Shotcrete)

Reinforcing Bars for Shotcrete Facing	AASHTO M31/ASTM A615, Grade 420, deformed.
Bearing Plates	AASHTO M183/ASTM A36.
Nuts	AASHTO M291, grade B, hexagonal, fitted with beveled washer or spherical seat to provide uniform bearing.
Prepackaged Shotcrete	ASTM C928.
Drainage Geotextile	
For Wall Footing Drain	AASHTO M288 Class 2, Permittivity min. 0.2 per second; AOS 0.25 mm max.
For Drain Strip	AASHTO M288 Class 3, Permittivity min. 0.2 per second; AOS 0.25 mm max.
Drainage Aggregate	AASHTO M43/ASTM C33 No. 67 with no more than two percent passing the 0.075 mm sieve.
Geocomposite Drain Strip	Miradrain 6000, Amerdrain 500 or approved equal.
Film Protection	Polyethylene films per AASHTO M-171.
PVC Connector and Drain Pipes:	
Pipe	ASTM 1785 Schedule 40 PVC, solid and perforated wall, cell classification 12454-B or 12354-C, wall thickness SDR 35,

	with solvent weld or elastomeric gasket joints.
Fittings	ASTM D3034, cell classification 12454-B or 12454-C, wall thickness SDR35, with solvent weld or elastomeric gasket joints.
Solvent Cement	ASTM D2564
Primer	ASTM F656

Materials shall be delivered, stored and handled to prevent contamination, segregation, corrosion or damage. Store liquid admixtures to prevent evaporation and freezing.

Drainage geotextile and geocomposite drain strips shall be provided in rolls wrapped with a protective covering and stored in a manner which protects the fabric from mud, dirt, dust, debris, and shotcrete rebound. Protective wrapping shall not be removed until immediately before the geotextile or drain strip is installed. Extended exposure to ultra-violet light shall be avoided. Each roll of geotextile or drain strip in the shipment shall be labelled to identify the production run.

2.1 Shotcrete Mix Design. The Contractor must receive notification from the Engineer that the proposed mix design and method of placement are acceptable before shotcrete placement can begin.

2.1.1 Aggregate. Aggregate for shotcrete shall meet the strength and durability requirements of AASHTO M6/M80 and the following gradation requirements: ***(See Commentary 2.1.1)***

Sieve Size	Percent Passing by Weight
12.5 mm	100
9.50 mm	90-100
4.75 mm	70-85
2.36 mm	50-70
1.18 mm	35-55
0.60 mm	20-35
0.30 mm	8-20
0.15 mm	2-10

2.1.2 Proportioning and Use of Admixtures. Proportion the shotcrete to be pumpable with the concrete pump furnished for the work, with a cementing materials content of at least 390 kilograms per cubic meter and water/cement ratio not greater than 0.50. Do not use admixtures unless approved by the Engineer. Thoroughly mix admixtures into the shotcrete at the rate specified by the manufacturer. Accelerators (if used) shall be compatible with the cement used, be non-corrosive to steel and not promote other detrimental effects such as cracking or excessive shrinkage. The maximum allowable chloride ion content of all ingredients shall not exceed 0.10% when tested to

AASHTO T260.

2.1.3 Air Entrainment. Air entrainment is not required for temporary shotcrete construction facings.

2.1.4 Strength Requirements. Provide a shotcrete mix capable of attaining 14 MPa compressive strength in 3 days and 28 MPa in 28 days. ***(See Commentary 2.1.4).*** The average compressive strength of each set of three test cores extracted from test panels or wall face must equal or exceed 85 percent of the specified compressive strength, with no individual core less than 75 percent of the specified compressive strength, in accordance with ACI 506.2.

2.1.5 Mixing and Batching. Aggregate and cement may be batched by weight or by volume in accordance with the requirements of ASTM C94 or AASHTO M241/ASTM C685. Mixing equipment shall thoroughly blend the materials in sufficient quantity to maintain placing continuity. Ready mix shotcrete shall comply with AASHTO M157. Shotcrete shall be batched, delivered, and placed within 90 minutes of mixing. The use of retarding admixtures may extend application time beyond 90 minutes if approved by the Engineer.

Premixed and packaged shotcrete mix may be provided for on-site mixing. The packages shall contain materials conforming to the Materials section of this specification. Placing time limit after mixing shall be per the manufacturers' recommendations.

2.2 Field Quality Control. Both preconstruction test panels (for nozzlemen without previous ACI certification) and production test panels or test cores from the wall facing are required. Shotcreting and coring of test panels shall be performed by qualified personnel in the presence of the Engineer. The Contractor shall provide equipment, materials, and personnel as necessary to obtain shotcrete cores for testing including construction of test panel boxes, field curing requirements and coring. Compressive strength testing will be performed by the Engineer. ***(See Commentary 2.2).*** Shotcrete final acceptance will be based on the 28-day strength.

Shotcrete production work may commence upon initial approval of the design mix and nozzlemen and continue if the specified strengths are obtained. The shotcrete work by a crew will be suspended if the test results for their work does not satisfy the strength requirements. The Contractor shall change all or some of the following: the mix, the crew, the equipment, or the procedures. Before resuming work, the crew must shoot additional test panels and demonstrate that the shotcrete in the panels satisfies the specified strength requirements. The cost of all work required to obtain satisfactory strength tests will be borne by the Contractor.

2.2.1 Preconstruction Test Panels. Each nozzleman without previous ACI certification shall furnish at least one preconstruction test panel for each proposed mixture being considered and for each shooting position to be encountered on the job. Preconstruction test panels shall be made prior to the commencement of production work using the same equipment, materials, mixture proportions and procedures proposed for the job

Make preconstruction test panels with minimum dimensions of 750 x 750 mm square and at least 100 mm thick. Slope the sides of preconstruction and production test panels at 45 degrees over the full panel thickness to release rebound. ***(See Commentary 2.2.1).***

2.2.2 Production Test Panels . Furnish at least one production test panel or, in lieu of production test panels, six 75mm diameter cores taken from the shotcrete facing, during the first production application of shotcrete and henceforth for every 500 m^2 of shotcrete placed. ***(See Commentary 2.2.2).*** Construct the production test panels simultaneously with the shotcrete facing installation at times designated by the Engineer. Make production test panels with minimum dimensions of 450x450mm square and at least 100 mm thick.

2.2.3 Test Panel Curing, Test Specimen Extraction and Testing. Immediately after shooting, field moist cures the test panels by covering and tightly wrapping with a sheet of material meeting the requirements of ASTM C171 until they are delivered to the testing lab or test specimens are extracted. Do not immerse the test panels in water. Do not further disturb test panels for the first 24 hours after shooting. Provide at least six 75 mm diameter core samples cut from each preconstruction test panel and production test panel. Contractor has the option of extracting test specimens from test panels in the field or transporting to another location for extraction. Keep panels in their forms when transported. Do not take cores from the outer 150 mm of test panels measured in from the top outside edges of the panel form. **(See Commentary 2.2.3)**. Trim the ends of the cores to provide test cylinders at least 75 mm long. If the Contractor chooses to take cores from the wall face in lieu of making production test panels, locations will be designated by the Engineer. Clearly mark the cores and container to identify the core locations and whether they are for preconstruction or production testing. If for production testing, mark the section of the wall represented by the cores on the cores and container. Immediately wrap cores in wet burlap or material meeting requirements of ASTM C171 and seal in a plastic bag. Deliver cores to the Engineer or testing lab, as directed by the Engineer, within 48 hours of shooting the panels. The remainder of the panels will become the property of the Contractor. Compressive strength testing will be performed by the Engineer. Upon delivery to the testing lab, samples will be placed in the moist room until the time of test. When the test length of a core is less than twice the diameter, the correction factors given in AASHTO T24/ASTM C42 will be applied to obtain the compressive strength of individual cores. Three cores will be tested at 3 days and three cores will be tested at 28 days in accordance with AASHTO T24/ASTM C42.

Fill core holes in the wall by dry-packing with non-shrink patching mortar after the holes are cleaned and dampened. Do not fill core holes with shotcrete.

3.0 CONSTRUCTION REQUIREMENTS

3.1 Wall Drainage Network. Install and secure all elements of the wall drainage network as shown on the Plans, specified herein, or as required by the Engineer to suit the site conditions. The drainage network shall consist of installing geocomposite drain strips, PVC connection pipes and wall footing

drains as shown on the Plans or as directed by the Engineer. Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting.
Unanticipated subsurface drainage features exposed in the excavation cut face shall be captured independently of the wall drainage network and shall be mitigated prior to shotcrete application in accordance with Section 3.1 of the Soil Nail and Wall Excavation Specification. Costs due to the required mitigation will be paid for as Extra Work.

3.1.1 Geocomposite Drain Strips. Install geocomposite drain strips centered between the columns of nails as shown on the Plans. The drain strips shall be at least 300 mm wide and placed with the geotextile side against the ground. Secure the strips to the excavation face and prevent shotcrete from contaminating the ground side of the geotextile. Drain strips will be continuous. Splices shall be made with a 300 mm minimum overlap such that the flow of water is not impeded. Repair damage to the geocomposite drain strip, which may interrupt the flow of water. ***(See Commentary 3.1.1)***

3.1.2 Footing Drains. Install footing drains at the bottom of each wall as shown on the Plans. The drainage geotextile shall envelope the footing drain aggregate and pipe and conform to the dimensions of the trench. Overlap the drainage geotextile on top of the drainage aggregate as shown on the Plans. Replace or repair damaged or defective drainage geotextile.

3.1.3 Connection Pipes and Weepholes. Install connection pipes as shown on the Plans. Connection pipes are lengths of solid PVC pipe installed to direct water from the geocomposite drain strips into a footing drain or to the exposed face of the wall. Connect the connection pipes to the drain strips using either prefabricated drain grates as shown on the Plans or using the alternate connection method described below. Install the drain grate per the manufacturer's recommendations. The joint between the drain grate and the drain strip and the discharge end of the connector pipe shall be sealed to prevent shotcrete intrusion. Connection pipes that end at the footing drain shall be extended to the edge of the drain. Do not puncture the drainage fabric around the footing drain.

The alternative acceptable method for connection of the connector pipe to the drain strip involves cutting a hole slightly larger than the diameter of the pipe into the strip plastic core but not through the geotextile. Wrap both ends of the connection pipe in geotextile in a manner that prevents migration of fines through the pipe. Tape or seal the inlet end of the pipe where it penetrates the drain strip and the discharge end of the connector pipe in a manner that prevents penetration of shotcrete into the drain strip or pipe. To assure passage of groundwater from the drain strip into the connector pipe, slot the inlet end of the connector pipe at every 45 degrees around the perimeter of the pipe to a depth of 6 mm.

Weepholes, if required, shall be provided through the construction facing to drain water from behind the facing. Install as shown on the Plans. Use PVC pipe to form the weephole through the shotcrete. Cover the end of the pipe contacting the soil with a drainage geotextile. Prevent shotcrete intrusion into the discharge end of the pipe. ***(See Commentary 3.1.3)***

3.2 Temporary Shotcrete Construction Facing

3.2.1 Shotcrete Alignment and Thickness Control. Ensure that the thickness of shotcrete satisfies the minimum requirements shown on the Plans using shooting wires, thickness control pins, or other devices acceptable to the Engineer. Install thickness control devices normal to the surface such that they protrude the required shorcrete thickness outside the surface. Ensure that the front face of the shotcrete does not extend beyond the limits shown on the Plans.

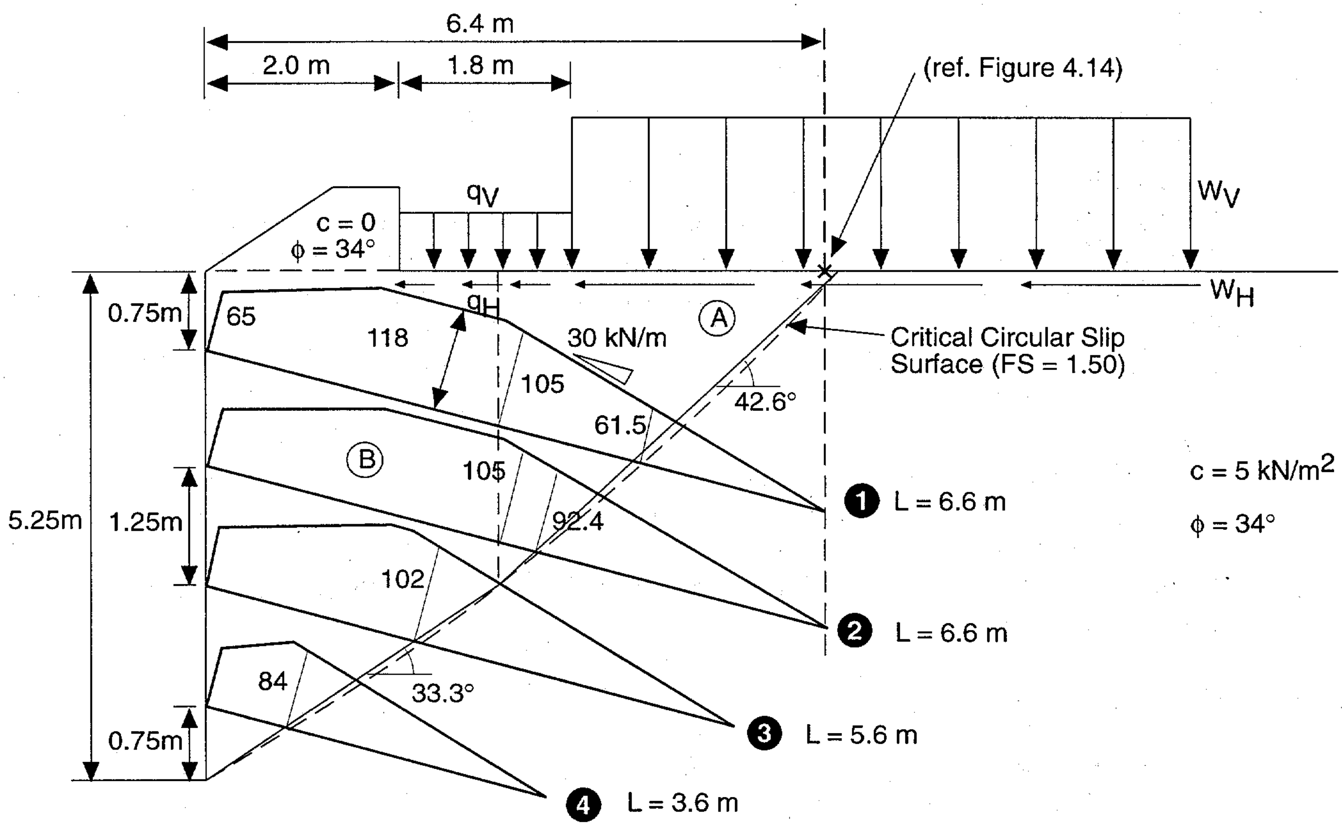
3.2.2 Surface Preparation. Clean the face of the excavation and other surfaces to be shotcreted of loose materials, mud, rebound, overspray or other foreign matter that could prevent or reduce shotcrete bond. Protect adjacent surfaces from overspray during shooting. Avoid loosening, cracking, or shattering the ground during excavation and cleaning. Remove any surface material which is so loosened or damaged, to a sufficient depth to provide a base that is suitable to receive the shotcrete. Remove material that loosens as the shotcrete is applied . Cost of additional shotcrete is incidental to the work. Divert water flow and remove standing water so that shotcrete placement will not be detrimentally affected by standing water. Do not place shotcrete on frozen surfaces.

3.2.3 Delivery and Application. Maintain a clean, dry, oil-free supply of compressed air sufficient for maintaining adequate nozzle velocity at all times. The equipment shall be capable of delivering the premixed material accurately, uniformly, and continuously through the delivery hose. Control shotcrete application thickness, nozzle technique, air pressure, and rate of shotcrete placement to prevent sagging or sloughing of freshly-applied shotcrete.

Apply the shotcrete from the lower part of the area upwards to prevent accumulation of rebound. Orient nozzle at a distance and approximately perpendicular to the working face so that rebound will be minimal and compaction will be maximized. Pay special attention to encapsulating reinforcement. Do not work rebound back into the construction. Where shotcrete is used to complete the top ungrouted zone of the nail drill hole near the face, position the nozzle into the mouth of the drillhole to completely fill the void.

A clearly defined pattern of continuous horizontal or vertical ridges or depressions at the reinforcing elements after they are covered with shotcrete will be considered an indication of insufficient reinforcement cover or poor nozzle techniques. In this case the application of shotcrete shall be immediately suspended and the Contractor shall implement corrective measures before resuming the shotcrete operations. The shotcreting procedure may be corrected by adjusting the nozzle distance and orientation, by insuring adequate cover over the reinforcement, by adjusting the water content of the shotcrete mix or other means. Adjustment in water content of wet-mix will require requalifying the shotcrete mix.

3.2.4 Defective Shotcrete. Repair shotcrete surface defects as soon as possible after placement. Remove and replace shotcrete which exhibits segregation, honeycombing, lamination, voids, or sand pockets. In-place shotcrete determined not to meet the specified strength requirement will be subject to remediation as determined by the Engineer. Possible remediation options include placement of



Surcharge:

$q_V = 33.4$ kN/m^2

$q_H = 32.8$ kN/m^2

$W_V = 86.0$ kN/m^2

$W_H = 0$ kN/m^2

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

Figure 5.16 Bridge Abutment Wall Design Example Critical Cross-Section Static Loading - Load Group I (SLD)

Headed studs location on bearing plate, from plan location: 6 mm

Spacing between reinforcing bars, from plan dimension; 25 mm

Reinforcing lap, from specified dimension: - 25mm

Thickness of shotcrete; - 10 mm

Nail head bearing plate, deviation from parallel to wall face: 10 degrees
(See Commentary 3.2.10).

3.3 Backfilling Behind Wall Facing Upper Cantilever. Compact backfill within 1 meter behind the wall facing upper cantilever using light mechanical tampers.

3.4 Safety Requirements. Nozzlemen and helpers shall be equipped with gloves, eye protection, and adequate protective clothing during the application of shotcrete. The Contractor is responsible for meeting all federal, state and local safety code requirements.

3.5 CIP Concrete Form connection to Shotcrete Facing. When mechanical, grouted, or epoxied anchors embedded into the shotcrete facing are used to support a one-sided CIP face form, perform pullout testing of the embedded anchors in accordance with ASTM C900 and as modified herein. Perform pullout testing of installed anchors prior to attachment of the face form. Select test anchor locations to be representative of the full wall surface area to be covered.

For facing areas up to 500 m^2, perform a minimum of three flexure/shear pullout tests with the anchor located approximately mid-span between two adjacent nail heads and with the nail heads or other reaction points located approximately one-half the nail spacing from the anchor. For facing areas in excess of 500 m^2, perform one additional flexure/shear pullout test for each additional 250 m^2 of face area. Test these anchors to 1.5 times their required design load (calculated as the design concrete fluid pressure times the anchor tributary area).

Perform local punching shear pullout testing on 2 percent of the installed anchors. Place the load reaction support no closer to the edge of the anchor than the embedment depth of the anchor into the construction facing. Test these anchors to 2.0 times their required design load.

Modify the anchor and/or face form support system if the tested anchors do not meet the above test acceptance criteria. Modified anchor installation will require re-testing in accordance with the above testing criteria. Cost of anchor pullout testing is incidental to the work. **(See Commentary 3.5)**

4.0 METHOD OF MEASUREMENT. The shotcrete facing will be measured in square meters of the shotcrete area completed and accepted in the final work. The net area lying in a plane of the outside front face of the structure as shown on the Plans will be measured. No measurement or payment will be made for additional shotcrete or CIP concrete needed to fill voids created by irregularities in the cut face, excavation overbreak or inadvertent excavation beyond the plan final wall face excavation line, or failure to construct the facing to the specified line and grade and tolerances. The final pay quantity shall include all structural shotcrete, admixtures, reinforcement, welded wire mesh, wire holding devices, embedded CIP face form support anchors (if applicable), wall drainage materials, bearing plates and nuts, test panels and all sampling, testing and reporting required by the Plans and this Specification. The final pay quantity shall be the design quantity increased or decreased by any changes authorized by the Engineer.

5.0 BASIS OF PAYMENT. The accepted quantity measured as provided above will be paid for at the contract unit price per square meter. Payment will be full compensation for furnishing all equipment, materials, labor, tools and incidentals necessary to complete the work as specified and as detailed on the Plans, including the work required to provide the proper shotcrete facing alignment and thickness control. All wall drainage materials including geocomposite drain strips, connection pipes, drain grates, drain aggregate and geotextile, fittings, and accessories are considered incidental to the shotcrete facing and will not be paid separately.

Payment will be made for the following bid item included in the bid form:

Pay Item	Measurement Unit
Shotcrete Construction Facing	Square Meter

END OF SECTION

APPENDC1.SI

COMMENTARY TO TEMPORARY SHOTCRETE CONSTRUCTION FACING AND WALL DRAINAGE GUIDE SPECIFICATION

1.1 EXPERIENCE REQUIREMENTS

1.1 Specifier: If shotcrete nozzlemen have not been previously certified, it may be practical for the Contractor to arrange to certify them during the preconstruction test program through local testing laboratories or other experienced ACI recognized shotcrete specialists.

2.1. SHOTCRETE MIX DESIGN

2.1.1 Specifier. The aggregate gradation presented is commonly used for dry-mix and wet-mix shotcrete and has been satisfactorily used for a number of soil nailing projects. It requires a minimum of 15% of coarse aggregate sizes, (i.e. greater than 4.75 mm). Higher coarse aggregate contents would be preferable from a quality standpoint but may present problems with regard to pumping and shooting. In some areas, a mix consisting of only fine aggregate may be proposed because the local shotcrete industry is accustomed to that material. This is particularly true for areas where dry-mix is prominent. Such mixes will have a higher propensity for shrinkage but will have reduced rebound. They are suitable for construction facings but not permanent facings.

2.1.4 Specifier. A 3 day strength criterion has been selected in addition to the traditional 28 days because soil nailing shotcrete is normally required to accept loads at an early age and therefore it is the early strength that is critical. The 28 day strength is included because of the need to compare it with the specified ultimate strength of the facing as used in the structural design.

2.2 FIELD QUALITY CONTROL

2.2 Specifier. If testing is to be the Contractor's responsibility, modify the specification accordingly.

2.2.1 Specifier. Test panels with 45 degree sloped sides are recommended to minimize entrapment of rebound when shooting the test panel and for ease of stripping the form.

2.2.2 Specifier. The 500 square meter testing area requirement may need to be adjusted depending on the area of wall to be constructed on the contract.

2.2.3 Specifier. Most Authorities require that the outside 150 mm measured in from the top outside edges of the panel form be discarded because it typically contains rebound or overspray and is not representative.

3.1 WALL DRAINAGE NETWORK

3.1.1 Specifier. At the designers option, horizontal geocomposite drain strips may be included behind horizontal shotcrete construction joints and/or where zones of localized groundwater seepage is encountered during construction.

3.1.3 Specifier. Longer horizontal drains are typically not included in the project specification unless more significant groundwater is anticipated at the excavation cut face. The project geotechnical engineer should assess the need for horizontal drains and determine the length and spacing. Most highway agencies have standard specifications and plan details covering horizontal drain material and installation requirements. Where deemed necessary, horizontal drains can be included on the Plans and the following verbiage added to the Specifications:

Slotted PVC horizontal drains, if required, shall be installed as shown on the Plans. Install horizontal drains in drill holes that slope upward at an inclination of 2 to 5 degrees. Provide PVC horizontal drain pipe meeting the materials requirements set forth in the Standard Specifications. Attach a solid inlet pipe to the slotted pipe approximately 300 mm behind the back of the construction facing. Seal the annular space between the inlet pipe and the drill hole with bentonite pellets or non shrink grout. Connect the inlet pipe to the discharge pipe that will empty directly into the footing drain, as shown on the Plans.

3.2 TEMPORARY SHOTCRETE CONSTRUCTION FACING

3.2.5 Specifier: The contractor should demonstrate that his techniques will preclude sag or joint separation between excavation lifts. This may occur in walls thicker than 100 mm due to consolidation of the loose backfill material placed to provide a reinforcing splice below the lift. It has been demonstrated that joint separation can be mitigated by building the shotcrete in each lift from the bottom while allowing adequate time for consolidation of loose splice backfill.

3.2.10 Specifier. Tolerances for a temporary wall with construction facing only, i.e. will not be covered by a CIP facing, can have less restrictive tolerances. These may vary project to project depending on application, site geometry and other constraints and thus are left to discretion of individual Owners/Designers to establish. For tolerances and finishing requirements for CIP facings, refer to highway agency standard specifications or project special provisions.

3.5 CIP Concrete Form Connection to Shotcrete Facing

3.5 Specifier. This section covering the embedded anchors can be moved to the CIP facing special provision, if desired, along with making measurement and payment of the embedded anchors and associated testing incidental to the CIP facing pay item rather than the shotcrete facing pay item.

STEEL FIBER REINFORCED SHOTCRETE (SFRSC)

Specifier. Some designers or contractors may desire to substitute steel fiber reinforcement for welded wire mesh. There are a number of research papers documenting that steel fiber test panels can produce equivalent strengths to those with wire mesh. The use of fiber reinforcing for shotcrete construction facings has been demonstrated for both above ground and underground applications such as rock slope stabilization and tunneling. However, to date it's use for soil nail wall facings has been limited. The deletion of wire mesh and the use of fiber reinforced shotcrete is particularly advantageous where the shotcrete has to be applied against a very rough, irregular excavation cut face. At this time, only steel fiber reinforcing should be considered since test data for non-metallic fibers indicate that equivalent flexural strength cannot be assured at normal dosages.

The presence of steel fibers in a shotcrete matrix provides two functions - reinforcement across a crack, similar to conventional rebar, and reinforcement at the crack tip to restrain propagation of the crack. The effectiveness of a particular fiber reinforcement is a function of the:

- *A. aspect ratio of the fibers;*
- *B. fiber dosage;*
- *C. fiber geometry with regard to bond development.*

At the time of preparation of this manual, structural design procedures for designing with steel fibers are not as well developed as for conventional concrete reinforced with welded wire mesh or deformed reinforcing bars. ACI 544.4, "Design Considerations for Steel Fiber Reinforced Concrete", contains only very limited design guidelines for designing with steel fibers. Use of steel fiber reinforced shotcrete is increasing for applications such as tunnel linings and slope stabilization/protection and there has been some limited usage to date as temporary soil nail wall facings. However, much of the usage is based on successful experience rather then rigorous structural design. In designing fiber reinforced shotcrete, the flexural strength to cracking and the post cracking residual strength are often specified. If the use of steel fiber reinforced shotcrete is allowed, the following are guidance specification clauses that can be used. Note that the SFRSC guideline specification given below calls for initial 7-day strength rather than 3-day as for plain shotcrete. The reason is that 3-days is not sufficient time to obtain and prepare the flexural beam test specimens for the more elaborate ASTM C1018 test. Past problems with the C1018 test procedure, specifically with the method of measuring deflections, has led many agencies to stop specifying the test. However, this has recently been corrected and the revised test procedure is now generally accepted by all shotcrete authorities. However, due to the elaborateness and greater expense of the ASTM C1018 test, combined with few highway agency labs equipped to perform the test, the guideline specification given below calls only for the C1018 test being required to initially test and qualify the fiber reinforced shotcrete mix design, with subesequent during production testing done using conventional compressive strength testing of cores as for plain shotcrete. Transportation agencies making initial use may wish to designate their first project as an experimental feature.

2.0 MATERIALS

Steel Fibers — ASTM A820, Type I, II, or III, Deformed, Steel Fibers, 25 to 38 mm length, minimum aspect ratio of 60.

2.1.4 Steel Fiber Reinforced Shotcrete (SFRSC)

Steel fiber reinforced shotcrete shall comply with the requirements of ASTM C1116 "Standard Specification for Fiber Reinforced Shotcrete", except as otherwise specified. Steel fiber reinforced shotcrete shall be mixed in accordance with the fiber manufacturer's recommendations with a minimum dosage of steel fibers of 60 kg/m^3. Fiber reinforced shotcrete shall have a minimum first crack flexural strength of 4 MPa at 7 days and and 5 Mpa at 28 days and a minimum Residual Strength Factor of $R_{10,30} = 60$ at 7 and 28 days as determined by testing in accordance with ASTM C1018.

2.2.1 SFRSC Preconstruction Test Panels

Preconstruction test panels shall be made prior to the commencement of production work using the same equipment, materials, mixture proportions and procedures proposed for the job Make preconstruction test panels with minimum dimensions of 750x750x150 mm. Slope the sides of preconstruction and production test panels at 45 degrees over the full panel thickness to release rebound.

2.2.2 SFRSC Production Test Panels

Furnish at least one production test panel or, in lieu of production test panels, six 75mm diameter cores taken from the shotcrete facing, during the first production application of fiber reinforced shotcrete and henceforth for every 500 square meters of shotcrete placed. Construct the production test panels simultaneously with the shotcrete facing installation at times designated by the Engineer. Make production test panels with minimum dimensions of 450x450x100 mm.

2.2.3 SFRSC Test Panel Curing, Test Specimen Extraction and Testing

Field and lab curing of SFRSC test panels and identification and packaging of test specimens will be as for plain shotcrete test panels and cores per Section 2.2.3. Do not disturb test panels within the first 24 hours after shooting. Do not take test specimens from the outer 150 mm of test panels measured in from the top outside edges of the panel form. Deliver test specimens to the testing lab within 48 hours of shooting the panels.

Cut a total of six 100x100x350 mm test beams from each preconstruction test panel in accordance with ASTM C1140 and have tested in accordance with ASTM C1018 for Flexural Strength and Residual Strength Factor, $R_{10,30}$. Saw beams and test perpendicular to the top surface of the panel. Have beam testing done by a qualified independent testing lab. Test 3 beams at 7 days and 3 beams at 28 days.

Also extract six 75 mm diameter cores for compressive strength testing. The same requirements as set forth for core sampling, testing and compressive strength of plain shotcrete cores will apply to the steel fiber reinforced cores. Three cores will be tested at 3-days and three cores will be tested at 28-days. Provide beam and core test results to the Engineer within 24 hours of testing.

Upon the preconstruction panel ASTM C1018 test results verifying that the fiber reinforced design mix will meet the specified Flexural and Residual Strength requirements, compressive strength core testing will then be used for acceptance testing during production. Subesequent during production test panel and test specimen reqirements will be the same as required for plain shotcrete using test cores taken from production test panels or from the wall face. The average compressive strength of each set of three production cores required for acceptance shall be a minimum of 85% of the average compressive strength of the cores from the preconstruction test panels.

END COMMENTARY

APPENDC1.SI

APPENDIX C2

FHWA GUIDE SPECIFICATION
FOR
PERMANENT SHOTCRETE FACING AND WALL DRAINAGE
(OWNER-DESIGN)

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDC2.SI
Last Revision 10/31/98

(An electronic version this section is available at www.fhwa.dot.gov/bridge)

FHWA GUIDE SPECIFICATION
FOR
PERMANENT SHOTCRETE FACING AND WALL DRAINAGE
(OWNER-DESIGN)

1.0 DESCRIPTION. Shotcrete facing and wall drainage work consists of furnishing all materials and labor required for placing and securing geocomposite drainage material, connection pipes, footing drains, weepholes and horizontal drains (if required), drainage gutter, reinforcing steel and shotcrete for the permanent shotcrete facing and nail head bearing plates and nuts for the soil nail walls shown on the Plans. The Work shall include any preparatory trimming and cleaning of soil/rock surfaces and shotcrete cold joints to receive new shotcrete.

Shotcrete shall comply with the requirements of ACI 506.2, "Specifications for Materials, Proportioning and Application of Shotcrete", except as otherwise specified. Shotcrete shall consist of an application of one or more layers of concrete conveyed through a hose and pneumatically projected at a high velocity against a prepared surface.

Shotcrete may be produced by either a wet-mix or dry-mix process. The wet-mix process consists of thoroughly mixing all the ingredients except accelerating admixtures, but including the mixing water, introducing the mixture into the delivery equipment and delivering it, by positive displacement, to the nozzle. The wet-mix shotcrete shall then be air jetted from the nozzle at high velocity onto the surface. The dry-mix process consists of shotcrete without mixing water which is conveyed through the hose pneumatically with the mixing water introduced at the nozzle. For additional descriptive information, the Contractor's attention is directed to the American Concrete Institute ACI 506R "Guide to Shotcrete."

CIP concrete facing construction (if required) is covered by the Standard Specifications and/or CIP Facing Special Provisions. Soil nails and wall excavation are covered by the Permanent Soil Nails and Wall Excavation Specification. Soil nail wall instrumentation (if required) is covered by the Soil Nail Wall Instrumentation Specification.

Where the imperative mood is used within this Specification for conciseness, "the Contractor shall" is implied.

1.1 Contractor's Experience Requirements. Workers, including foremen, nozzlemen, finishers and delivery equipment operators, shall be fully experienced to perform the work. All shotcrete nozzlemen on this project shall have experience on at least 3 projects in the past 3 years in similar permanent shotcrete application work totaling at least 1000 square meters of wall face area and shall demonstrate ability to satisfactorily place the shotcrete. Finishers shall have experience on at least 3 projects in the past 3 years in similar permanent shotcrete application work totaling at least 1000 square meters of wall face area.

Initial qualification of nozzlemen will be based either on previous ACI certification or satisfactory completion of preconstruction test panels. The requirement for nozzlemen to shoot preconstruction qualification test panels will be waived for nozzlemen who can submit documented proof they have been certified in accordance with the ACI 506.3R Guide to Certification of Shotcrete Nozzlemen. The Certification shall have been done by a recognized shotcrete testing lab and/or recognized shotcreting consultant and have covered the type of shotcrete to be used (plain wet-mix, plain dry-mix or steel fiber reinforced). (***See Commentary 1.1***). All nozzlemen will be required to periodically shoot production test panels during the course of the Work at the frequency specified herein.

Notify the Engineer not less than 2 days prior to the shooting of preconstruction test panels to be used to qualify nozzlemen without previous ACI certification. Use the same shotcrete mix and equipment to make qualification test panels as those to be used for the soil nail wall shotcrete facing. Initial qualification of the nozzlemen will be based on a visual inspection of the shotcrete density and void structure and on achieving the specified 3-day and 28-day compressive strength requirements determined from test specimens extracted from the preconstruction test panels. Preconstruction and production test panels, core extraction and compressive strength testing shall be conducted in accordance with ACI 506.2 and AASHTO T24/ASTM C42, unless otherwise specified herein. Nozzlemen without ACI Certification will be allowed to begin production shooting based on satisfactory completion of the preconstruction test panels and passing 3-day strength test requirements. Continued qualification will be subject to passing the 28-day strength tests and shooting satisfactory during production test panels.

1.2 Construction Submittals. At least 15 calendar days before the planned start of shotcrete placement, submit 5 copies of the following information, in writing, to the Engineer for review:

a. Written documentation listing at least 5 permanent structural shotcrete walls successfully completed within the past 3 years, including photographs of the project as well as names, addresses, and phone numbers of the owner's representative.

b. Written documentation of the finisher's and nozzlemen's qualifications including proof of ACI certification (if applicable).

c. Proposed methods of shotcrete placement and of controlling and maintaining facing alignment and location and shotcrete thickness.

d. Shotcrete mix design including:
Type of Portland cement.
Aggregate source and gradation.
Proportions of mix by weight and water-cement ratio.
Proposed admixtures, manufacturer, dosage, technical literature.
Previous strength test results for the proposed shotcrete mix completed within one year of the start of shotcreting may be submitted for initial verification of the required compressive strengths at start of production work.

e. Certificates of Compliance, manufacturers' engineering data and installation instructions for the drainage geotextile, geocomposite drain strip, drain grate and accessories.

f. Certificates of Compliance for bearing plates, nuts, drainage aggregate and PVC drain piping.

The Engineer will approve or reject the Contractor's Submittals within 10 calendar days after receipt of a complete submission. The Contractor will not be allowed to begin wall construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time will be allowed due to incomplete submittals.

Upon delivery to the project site, provide Certified mill test results for all reinforcing steel specifying the minimum ultimate strength, yield strength, elongation and chemical composition.

1.3 Pre-Construction Meeting. A pre-construction meeting scheduled by the Engineer will be held prior to the start of wall construction. Attendance is mandatory. The shotcrete Contractor, if different than the soil nail specialty Contractor, shall attend. See Section 1.4 of the Permanent Soil Nail and Wall Excavation Specification.

2.0 MATERIALS. All materials for shotcrete shall conform to the following requirements:

Cement	AASHTO M85/ ASTM C150, Type I, II, III or V.
Fine Aggregate	AASHTO M6/ASTM C33 clean, natural.
Coarse Aggregate	AASHTO M80, Class B for quality
Water	Clean and Potable. AASHTO M157/ASTM C94
Chemical Admixtures	
Accelerator	Fluid type, applied at nozzle, meeting requirements of AASHTO M194/ASTM C494/ASTM C1141
Air-Entraining Agent	AASHTO M154/ASTM C260
Water-reducer and Superplastisizer	AASHTO M194/ASTM C494 Type A,C,D,E,F, or G
Retarders	AASHTO M194/ ASTM C494 Type B or D.

Mineral Admixtures	
Fly Ash	AASHTO M295/ASTM C618 Type F or C, cement
Silica Fume	ASTM C1240, 90 percent minimum silicon dioxide solids content, not to exceed 12 percent by weight of cement.
Welded Wire Fabric	AASHTO M55/ASTM A185 or A497.
Reinforcing Bars for Shotcrete Facing	AASHTO M31/ASTM A615, Grade 420, deformed.
Bearing Plates	AASHTO M183/ASTM A36.
Nuts	AASHTO M291, grade B, hexagonal, fitted with beveled washer or spherical seat to provide uniform bearing.
Curing Compounds	AASHTO M148, Type 1D or Type 2
Prepackaged Shotcrete	ASTM C928.
Drainage Geotextile	
For Wall Footing Drain	AASHTO M288 Class 2, Permittivity min. 0.2 per second; AOS 0.25 mm max.
For Drain Strip	AASHTO M288 Class 3, Permittivity min. 0.2 per second;
Drainage Aggregate	AASHTO M43/ASTM C33 No. 67 with no more than two percent passing the 0.075 mm sieve.
Geocomposite Drain Strip	Miradrain 6000, Amerdrain 500 or approved equal.
Film Protection	Polyethylene films per AASHTO M-171.
PVC Connector and Drain Pipes:	
Pipe	ASTM 1785 Schedule 40 PVC, solid and perforated wall, cell classification 12454-B or 12354-C, wall thickness SDR 35, with solvent weld or elastomeric gasket joints.
Fittings	ASTM D3034, cell classification 12454-B or 12454-C, wall thickness SDR35, with solvent weld or elastomeric gasket joints.

Solvent Cement	ASTM D2564
Primer	ASTM F656

Materials shall be delivered, stored and handled to prevent contamination, segregation, corrosion or damage. Store liquid admixtures to prevent evaporation and freezing.

Drainage geotextile and geocomposite drain strips shall be provided in rolls wrapped with a protective covering and stored in a manner which protects the fabric from mud, dirt, dust, debris, and shotcrete rebound. Protective wrapping shall not be removed until immediately before the geotextile or drain strip is installed. Extended exposure to ultra-violet light shall be avoided. Each roll of geotextile or drain strip in the shipment shall be labeled to identify the production run.

2.1 Shotcrete Mix Design. The Contractor must receive notification from the Engineer that the proposed mix design and method of placement are acceptable before shotcrete placement can begin.

2.1.1 Aggregate. Aggregate for shotcrete shall meet the strength and durability requirements of AASHTO M6/M80 and the following gradation requirements: ***(See Commentary 2.1.1)***

Sieve Size	Percent Passing by Weight
12.5 mm	100
9.50 mm	90-100
4.75 mm	70-85
2.36 mm	50-70
1.18 mm	35-55
0.60 mm	20-35
0.30 mm	8-20
0.15 mm	2-10

2.1.2 Proportioning and Use of Admixtures. Proportion the shotcrete to be pumpable with the concrete pump furnished for the work, with a cementing materials content of at least 390 kilograms per cubic meter and water/cement ratio not greater than 0.45. ***(See Commentary 2.1.2).*** Do not use admixtures unless approved by the Engineer. Thoroughly mix admixtures into the shotcrete at the rate specified by the manufacturer. Accelerators (if used) shall be compatible with the cement used, be non-corrosive to steel and not promote other detrimental effects such as cracking or excessive shrinkage. The maximum allowable chloride ion content of all ingredients shall not exceed 0.10% when tested to AASHTO T260.

2.1.3 Air Entrainment. Air entrainment is required for wet-mix shotcrete. The air content measured at the truck shall be between 7 to 10 percent when tested in accordance with AASHTO T152/ASTM C231. Air entrainment is not required in dry-mix shotcrete. ***(See Commentary 2.1.3)***

2.1.4 Strength and Durability Requirements. Provide a shotcrete mix capable of attaining 14 MPa compressive strength in 3 days and 28 MPa in 28 days. ***(See Commentary 2.1.4a).*** The average compressive strength of each set of three test cores extracted from test panels or wall face must equal or exceed 85 percent of the specified compressive strength, with no individual core less than 75 percent of the specified compressive strength, in accordance with ACI 506.2. The boiled absorption of shotcrete, when tested in accordance with ASTM C642 at 7 days, shall not exceed 8.0 percent. ***(See Commentary 2.1.4b)***

2.1.5 Mixing and Batching. Aggregate and cement may be batched by weight or by volume in accordance with the requirements of ASTM C94 or AASHTO M241/ASTM C685. Mixing equipment shall thoroughly blend the materials in sufficient quantity to maintain placing continuity. Ready mix shotcrete shall comply with AASHTO M157. Shotcrete shall be batched, delivered, and placed within 90 minutes of mixing. The use of retarding admixtures may extend application time beyond 90 minutes if approved by the Engineer.

Premixed and packaged shotcrete mix may be provided for on-site mixing. The packages shall contain materials conforming to the Materials section of this specification. Placing time limit after mixing shall be per the manufacturers' recommendations.

2.2 Field Quality Control. Both preconstruction test panels (for nozzlemen without previous ACI certification) and production test panels or test cores from the wall facing are required. Shotcreting and coring of test panels shall be performed by qualified personnel in the presence of the Engineer. The Contractor shall provide equipment, materials, and personnel as necessary to obtain shotcrete cores for testing including construction of test panel boxes, field curing requirements and coring. Compressive strength testing will be performed by the Engineer. ***(See Commentary 2.2).*** Shotcrete final acceptance will be based on the 28-day strength.

Shotcrete production work may commence upon initial approval of the design mix and nozzlemen and continue if the specified strengths are obtained. The shotcrete work by a crew will be suspended if the test results for their work does not satisfy the strength requirements. The Contractor shall change all or some of the following: the mix, the crew, the equipment, or the procedures. Before resuming work, the crew must shoot additional test panels and demonstrate that the shotcrete in the panels satisfies the specified strength requirements. The cost of all work required to obtain satisfactory strength tests will be borne by the Contractor.

2.2.1 Preconstruction Test Panels. Each nozzleman without previous ACI certification shall furnish at least two preconstruction test panels for each proposed mixture being considered and for each shooting position to be encountered on the job. Preconstruction test panels shall be made prior to the commencement of production work using the same equipment, materials, mixture proportions and procedures proposed for the job

Make preconstruction test panels with minimum dimensions of 750 x 750 mm square and at least 100 mm thick. Slope the sides of preconstruction and production test panels at 45 degrees over the full panel thickness to release rebound. ***(See Commentary 2.2.1).*** One preconstruction test panel shall include the maximum anticipated reinforcing congestion shown on the Plans. Cores extracted from the test panel shall demonstrate encapsulation of the reinforcement in accordance with ACI 506.2 equal to core grade 2 or better. The other preconstruction test panel shall be constructed without reinforcement and have cores extracted for absorption and compressive strength testing.

2.2.2 Production Test Panels. Furnish at least one production test panel or, in lieu of production test panels, nine 75 mm diameter cores taken from the shotcrete facing, during the first production application of shotcrete and henceforth for every 500 m^2 of shotcrete placed. ***(See Commentary 2.2.2).*** Construct the production test panels simultaneously with the shotcrete facing installation at times designated by the Engineer. Make production test panels with minimum full thickness dimensions of 450x450 mm square and at least 100 mm thick.

2.2.3 Test Panel Curing, Test Specimen Extraction and Testing. Immediately after shooting, field moist cure the test panels by covering and tightly wrapping with a sheet of material meeting the requirements of ASTM C171 until they are delivered to the testing lab or test specimens are extracted. Do not immerse the test panels in water. Do not further disturb test panels for the first 24 hours after shooting. Provide at least three 75 mm diameter core samples cut from each preconstruction test panel with reinforcement, for core grading. ***(See Commentary 2.2.3a).*** Provide at least nine 75 mm diameter core samples cut from each unreinforced preconstruction and production test panel for absorption and compressive strength testing. Contractor has the option of extracting test specimens from test panels in the field or transporting to another location for extraction. Keep panels in their forms when transported. Do not take cores from the outer 150 mm of test panels measured in from the top outside edges of the panel form. ***(See Commentary 2.2.3b).*** Trim the ends of the compressive strength cores to provide test cylinders at least 75 mm long. Do not trim the ends of the cores to be tested for boiled absorption. If the Contractor chooses to take cores from the wall face in lieu of making production test panels, locations will be designated by the Engineer. Clearly mark the cores and container to identify the core locations and whether they are for preconstruction or production testing. If for production testing, mark the section of the wall represented by the cores on the cores and container. Immediately wrap cores in wet burlap or material meeting the requirements of ASTM C171 and seal in a plastic bag. Deliver cores to the Engineer or testing lab, as directed by the Engineer, within 48 hours of shooting the panels. The remainder of the panels will become the property of the Contractor. Compressive strength and boiled absorption testing will be performed by the Engineer. Upon delivery to the testing lab, samples will be placed in the moist room until the time of test. When the test length of a core is less than twice the diameter, the correction factors given in AASHTO T24/ASTM C42 will be applied to obtain the compressive strength of individual cores. Three cores will be tested at 3 days and three cores will be tested at 28 days for compressive strength per AASHTO T24/ASTM C42. Three cores will be tested at 7 days for boiled absorption per ASTM C642.

Fill core holes in the wall by dry-packing with non-shrink patching mortar after the holes are cleaned and dampened. Do not fill core holes with shotcrete.

3.0 CONSTRUCTION REQUIREMENTS

3.1 Wall Drainage Network. Install and secure all elements of the wall drainage network as shown on the Plans, specified herein, or as required by the Engineer to suit the site conditions. The drainage network shall consist of installing geocomposite drain strips, PVC connection pipes and wall footing drains as shown on the Plans or as directed by the Engineer. Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting.

Unanticipated subsurface drainage features exposed in the excavation cut face shall be captured independently of the wall drainage network and shall be mitigated prior to shotcrete application in accordance with Section 3.1 of the Soil Nail and Wall Excavation Specification. Costs due to the required mitigation will be paid for as Extra Work.

3.1.1 Geocomposite Drain Strips. Install geocomposite drain strips centered between the columns of nails as shown on the Plans. The drain strips shall be at least 300 mm wide and placed with the geotextile side against the ground. Secure the strips to the excavation face and prevent shotcrete from contaminating the ground side of the geotextile. Drain strips will be continuous. Splices shall be made with a 300 mm minimum overlap such that the flow of water is not impeded. Repair damage to the geocomposite drain strip, which may interrupt the flow of water. ***(See Commentary 3.1.1)***

3.1.2 Footing Drains. Install footing drains at the bottom of each wall as shown on the Plans. The drainage geotextile shall envelope the footing drain aggregate and pipe and conform to the dimensions of the trench. Overlap the drainage geotextile on top of the drainage aggregate as shown on the Plans. Replace or repair damaged or defective drainage geotextile.

3.1.3 Connection Pipes and Weepholes. Install connection pipes as shown on the Plans. Connection pipes are lengths of solid PVC pipe installed to direct water from the geocomposite drain strips into a footing drain or to the exposed face of the wall. Connect the connection pipes to the drain strips using either prefabricated drain grates as shown on the Plans or using the alternate connection method described below. Install the drain grate per the manufacturer's recommendations. The joint between the drain grate and the drain strip and the discharge end of the connector pipe shall be sealed to prevent shotcrete intrusion. Connection pipes that end at the footing drain shall be extended to the edge of the drain. Do not puncture the drainage fabric around the footing drain.

The alternative acceptable method for connection of the connector pipe to the drain strip involves cutting a hole slightly larger than the diameter of the pipe into the strip plastic core but not through the geotextile. Wrap both ends of the connection pipe in geotextile in a manner that prevents migration of fines through the pipe. Tape or seal the inlet end of the pipe where it penetrates the drain strip and the discharge end of the connector pipe in a manner that prevents penetration of shotcrete into the drain strip or pipe. To assure passage of groundwater from the drain strip into the connector pipe, slot the inlet end of the connector pipe at every 45 degrees around the perimeter of the pipe to a depth of 6 mm.

Weepholes, if required, shall be provided through the shotcrete facing to drain water from behind the facing. Install as shown on the Plans. Use PVC pipe to form the weephole through the shotcrete. Cover the end of the pipe contacting the soil with a drainage geotextile. Prevent shotcrete intrusion into the discharge end of the pipe. ***(See Commentary 3.1.3)***

3.2 Permanent Shotcrete Facing

3.2.1 Shotcrete Alignment and Thickness Control. Ensure that the thickness of shotcrete satisfies the minimum requirements shown on the Plans using shooting wires, thickness control pins, or other devices acceptable to the Engineer. Install thickness control devices normal to the surface such that they protrude the required shotcrete thickness outside the surface and maintain a plane surface. The maximum distance between the wires on any surface shall be equal to the vertical nail spacing. Ensure that the alignment wires are tight, true to line, and placed to allow further tightening. Remove shooting wires after completion of shotcreting and/or screeding. Ensure that the front face of the shotcrete does not extend beyond the limits shown on the Plans.

3.2.2 Surface Preparation. Clean the face of the excavation and other surfaces to be shotcreted of loose materials, mud, rebound, overspray or other foreign matter that could prevent or reduce shotcrete bond. Protect adjacent surfaces from overspray during shooting. Avoid loosening, cracking, or shattering the ground during excavation and cleaning. Remove any surface material which is so loosened or damaged, to a sufficient depth to provide a base that is suitable to receive the shotcrete. Remove material that loosens as the shotcrete is applied. Cost of additional shotcrete is incidental to the work. Divert water flow and remove standing water so that shotcrete placement will not be detrimentally affected by standing water. Do not place shotcrete on frozen surfaces.

3.2.3 Delivery and Application. Maintain at all times a clean, dry, oil-free supply of compressed air sufficient for maintaining adequate nozzle velocity and for simultaneous operation of a blow pipe for cleaning away rebound. The equipment shall be capable of delivering the premixed material accurately, uniformly, and continuously through the delivery hose. Control shotcrete application thickness, nozzle technique, air pressure, and rate of shotcrete placement to prevent sagging or sloughing of freshly-applied shotcrete.

Apply the shotcrete from the lower part of the area upwards to prevent accumulation of rebound. Orient nozzle at a distance and approximately perpendicular to the working face so that rebound will be minimal and compaction will be maximized. Pay special attention to encapsulating reinforcement. Care shall be taken while encasing reinforcing steel and mesh to keep the front face of the reinforcement clean during shooting operations, so that shotcrete builds up from behind, to encase the reinforcement and prevent voids and sand pockets from forming. Use a blowpipe to remove rebound and overspray immediately ahead of the nozzle. Do not work rebound back into the construction. Remove rebound that does not fall clear of the working area. Hardened rebound and hardened overspray shall be removed prior to application of additional shotcrete, using abrasive blast cleaning, chipping hammers, high pressure water blasting or other suitable techniques. When the thickness of a individual shotcrete layer is 150 mm or greater, or when shotcreting is conducted through two curtains of reinforcement, place shotcrete by the bench gunning method. The bench gunning method shall consist of building up a thick layer of shotcrete from the bottom of the lift and maintaining the top surface at approximately a 45-degree slope. Where shotcrete is used to complete the top ungrouted zone of the nail drill hole near the face, position the nozzle into the mouth of the drillhole to completely fill the void.

A clearly defined pattern of continuous horizontal or vertical ridges or depressions at the reinforcing elements after they are covered with shotcrete will be considered an indication of insufficient reinforcement cover or poor nozzle techniques. In this case the application of shotcrete shall be immediately suspended and the Contractor shall implement corrective measures before resuming the shotcrete operations. The shotcreting procedure may be corrected by adjusting the nozzle distance and orientation, by insuring adequate cover over the reinforcement, by adjusting the water content of the shotcrete mix or other means. Adjustment in water content of wet-mix will require requalifying the shotcrete mix.

When using multiple layer shotcrete construction, the surface of the receiving layer shall be prepared before application of a subsequent layer, by either: (a) Brooming the stiffening layer with a stiff bristle broom to remove all loose material, rebound, overspray or glaze, prior to the shotcrete attaining initial set: or (b) If the shotcrete has set, surface preparation shall be delayed at least 24 hours, at which time the surface shall be prepared by sandblasting or high pressure water blasting, to remove all loose material, rebound, hardened overspray, glaze, or other material that may prevent adequate bond.

3.2.4 Defective Shotcrete. The Engineer shall have authority to accept or reject the shotcrete work. Shotcrete which does not conform to the project specifications may be rejected either during the shotcrete application process, or on the basis of tests on the test panels or completed work. Repair shotcrete surface defects as soon as possible after placement. Remove and replace shotcrete which exhibits segregation, honeycombing, lamination, voids, or sand pockets. In-place shotcrete determined not to meet the specified strength requirement will be subject to remediation as determined by the Engineer. Possible remediation options include placement of additional shotcrete thickness or removal and replacement, at the Contractor's cost.

3.2.5 Construction Joints. Taper construction joints uniformly toward the excavation face over a minimum distance equal to the thickness of the shotcrete layer. Square joints are not permitted. The surface of the joints shall be rough, clean, and sound. Provide a minimum reinforcement overlap at reinforcement splice joints as shown on the Plans. Clean and wet the surface of a joint before adjacent shotcrete is applied. ***(See Commentary 3.2.5).*** Where shotcrete is used to complete the top ungrouted zone of the nail drill hole near the face, to the maximum extent practical, clean and dampen the upper grout surface to receive shotcrete, similar to a construction joint.

3.2.6 Final Face Finish. Shotcrete finish shall be either an undisturbed gun finish as applied from the nozzle or a rod, broom, wood float, rubber float, steel trowel or rough screeded finish as shown on the Plans or specified herein. ***(See Commentary 3.2.6).***

3.2.7 Attachment of Nail Head Bearing Plate and Nut. Attach a bearing plate and nut to each nail head as shown on the Plans. While the shotcrete is still plastic and before its initial set, uniformly seat the plate on the shotcrete by hand wrench tightening the nut. Where uniform contact between the plate and the shotcrete cannot be provided, set the plate in a bed of grout. After grout has set for 24 hours, hand wrench tighten the nut. Embed the bearing plate and nut in the wall as shown on the Plans. Ensure full shotcrete encapsulation of the bearing plate and nut free of any voids or pockets behind the plate. Ensure bearing plates with headed studs are located within the tolerances shown on the Plans or specified herein.

3.2.8 Weather Limitations. Protect the shotcrete if it must be placed when the ambient temperature is below 5°C and falling or when it is likely to be subjected to freezing temperatures before gaining sufficient strength. Maintain cold weather protection until the in-place compressive strength of the shotcrete is greater than 5 MPa. Cold weather protection includes blankets, heating under tents, or other means acceptable to the Engineer. The temperature of the shotcrete mix, when deposited, shall be not less than 10°C or more than 35°C. Maintain the air in contact with shotcrete surfaces at temperatures above 0° C for a minimum of 7 days shall.

If the prevailing ambient conditions (relative humidity, wind speed, air temperature and direct exposure to sunlight) are such that the shotcrete develops plastic shrinkage and/or early drying shrinkage cracking, shotcrete application shall be suspended. The Contractor shall: (a) Reschedule the work to a time when more favorable ambient conditions prevail; and/or (b) Adopt corrective measures, such as installation of sun-screens, wind breaks or fogging devices, to protect the work. Remove and replace newly placed shotcrete exposed to rain that washes out cement or otherwise makes the shotcrete unacceptable.

3.2.9 Curing. Protect permanent shotcrete from loss of moisture for at least 7 days after placement. Cure shotcrete by methods that will keep the shotcrete surfaces adequately wet and protected during the specified curing period. Commence curing within 1 hour of shotcrete application. When the ambient temperatures exceeds 27° C, plan the Work such that curing can commence immediately after finishing. Complete curing in accordance with the following requirements. ***(See Commentary 3.2.9)***

3.2.9.1 Water Curing. Regulate the rate of water application to keep the surface continuously wet and to provide complete surface coverage with a minimum of runoff. The use of intermittent wetting procedures which allow the shotcrete to undergo wetting and drying during the curing period is prohibited.

3.2.9.2 Membrane Curing. Do not use curing compounds on any surfaces against which additional shotcrete or other cementitious finishing materials are to be bonded unless the surface is thoroughly sandblasted in a manner acceptable to the Engineer. Membrane curing compounds are to be spray applied as quickly as practical after initial shotcrete set at a coverage of not less than 2.5 m^2/liter.

3.2.9.3 Film Curing. Film curing with polyethylene sheeting may be used to supplement water curing on shotcrete that will be covered later with additional shotcrete or concrete. Spray the shotcrete surface with water immediately prior to installation of the polyethylene sheeting. Polyethylene sheeting shall completely cover the surfaces. Overlap the sheeting edges for proper sealing and anchorage. Joints between sheets shall be sealed. Promptly repair any tears, holes, and other damage. Anchor sheeting as necessary to prevent billowing.

3.2.10 Permanent Shotcrete Facing Tolerances. Construction tolerances for the permanent shotcrete facing are as follows:

Horizontal Location of Wire Mesh; Rebar; Headed Studs on Bearing Plates,
from Plan location; + or - 10 mm

Headed studs location on bearing plate, from plan location: 6 mm

Spacing between reinforcing bars, from plan dimension: 25 mm

Reinforcing lap, from specified dimension: - 25mm

Complete thickness of shotcrete, from plan dimension:

If troweled or screeded: -15 mm
If left as shot: - 30 mm

Planeness of finish face surface-gap under 3 meter straightedge-any direction:
If troweled or screeded: 15 mm
If left as shot: 30 mm

Nail head bearing plate, deviation from parallel to wall face: 10 degrees

3.3 Backfilling Behind Wall Facing Upper Cantilever. Compact backfill within 1 meter behind the wall facing upper cantilever using light mechanical tampers.

3.4 Safety Requirements. Nozzlemen and helpers shall be equipped with gloves, eye protection, and adequate protective clothing during the application of shotcrete. The Contractor is responsible for meeting all federal, state and local safety code requirements.

4.0 METHOD OF MEASUREMENT. The shotcrete facing will be measured in square meters of the shotcrete area completed and accepted in the final work. The net area lying in a plane of the outside front face of the structure as shown on the Plans will be measured. No measurement or payment will be made for additional shotcrete or CIP concrete needed to fill voids created by irregularities in the cut face, excavation overbreak or inadvertent excavation beyond the plan final wall face excavation line, or failure to construct the facing to the specified line and grade and tolerances. The final pay quantity shall include all structural shotcrete, admixtures, reinforcement, welded wire mesh, wire holding devices, wall drainage materials, bearing plates and nuts, test panels and all sampling, testing and reporting required by the Plans and this Specification. The final pay quantity shall be the design quantity increased or decreased by any changes authorized by the Engineer.

5.0 BASIS OF PAYMENT. The accepted quantity measured as provided above will be paid for at the contract unit price per square meter. Payment will be full compensation for furnishing all equipment, materials, labor, tools and incidentals necessary to complete the work as specified and as detailed on the Plans, including the work required to provide the proper shotcrete facing alignment and thickness control. All wall drainage materials including geocomposite drain strips, connection pipes, drain grates, drain aggregate and geotextile, fittings, and accessories are considered incidental to the shotcrete facing and will not be paid separately.

Payment will be made for the following bid item included in the bid form:

Pay Item	Measurement Unit
Permanent Shotcrete Facing	Square Meter

END OF SECTION

APPENDC2.SI

COMMENTARY TO PERMANENT SHOTCRETE FACING AND WALL DRAINAGE GUIDE SPECIFICATION

1.1 EXPERIENCE REQUIREMENTS

1.1 Specifier: If shotcrete nozzlemen have not been previously certified, it may be practical for the Contractor to arrange to certify them during the preconstruction test program through local testing laboratories or other experienced ACI recognized shotcrete specialists.

2.1. SHOTCRETE MIX DESIGN

2.1.1 Specifier. The aggregate gradation presented is commonly used for dry-mix and wet-mix shotcrete and has been satisfactorily used for a number of soil nailing projects. It requires a minimum of 15% of coarse aggregate sizes, (i.e. greater than 4.75 mm). Higher coarse aggregate contents would be preferable from a quality standpoint but may present problems with regard to pumping and shooting. In some areas, a mix consisting of only fine aggregate may be proposed because the local shotcrete industry is accustomed to that material. This is particularly true for areas where dry-mix is prominent. Such mixes will have a higher propensity for shrinkage but will have reduced rebound. They are suitable for construction facings but not permanent facings.

2.1.2 Specifier. A higher maximum water cement ratio of 0.50 may be permissible in non-demanding exposures.

2.1.3 Specifier. Air content should only be required in the specification where wet-mix shotcrete is used and shotcrete is permanent (i.e., services the long-term permanent loads). It is not practical to require air entrainment in dry-mix shotcrete. However, such shotcrete has been demonstrated to generally contain some air voids and often enough for providing protection for freezing and thawing. The high 7 to 10 percent air content specified for wet-mix shotcrete reflects the fact that a significant percentage of the air is lost during application due to the impact velocities.

2.1.4a Specifier. A 3 day strength criterion has been selected in addition to the traditional 28 days because soil nailing shotcrete is normally required to accept loads at an early age and therefore it is the early strength that is critical. The 28 day strength is included because of the need to compare it with the specified ultimate strength of the facing as used in the structural design.

2.1.4b Specifier. The boiled absorption test is a measure of the shotcrete void content and therefore compaction. Boiled absorption tests are not required for temporary construction facings, only permanent. The value of 8% boiled absorption is known to be achievable with reasonable quality shotcrete containing coarse aggregate corresponding to the gradation specified here. It is commonly used in many other specifications. However, the majority of the *experience with the use of this value comes from good quality aggregates with correspondingly lower absorptions. Some areas of the United States have aggregates with high absorptions, eg. sedimentaries, and higher shotcrete absorption values will automatically result since some of the aggregrate is exposed in the cut face of the test core. In these cases, a higher absorption value, say 9%, should be specified.*

2.2 FIELD QUALITY CONTROL

2.2 Specifier. If testing is to be the Contractor's responsibility, modify the specification accordingly.

2.2.1 Specifier. Test panels with 45 degree sloped sides are recommended to minimize entrapment of rebound when shooting the test panel and for ease of stripping the form.

2.2.2 Specifier. The 500 square meter testing area requirement may need to be adjusted depending on the area of wall to be constructed on the contract.

2.2.3a Specifier. 75 mm diameter cores are adequate for reinforcement size No. 13M or smaller. Specify 100 mm diameter cores if reinforcement size is larger than No. 13M.

2.2.3b Specifier. Most Authorities require that the outside 150 mm measured in from the top outside edges of the panel form be discarded because it typically contains rebound or overspray and is not representative.

3.1 WALL DRAINAGE NETWORK

3.1.1 Specifier. At the designers option, horizontal geocomposite drain strips may be included behind horizontal shotcrete construction joints and/or where zones of localized groundwater seepage is encountered during construction.

3.1.3 Specifier. Longer horizontal drains are typically not included in the project specification unless more significant groundwater is anticipated at the excavation cut face. The project geotechnical engineer should assess the need for horizontal drains and determine the length and spacing. Most highway agencies have standard specifications and plan details covering horizontal drain material and installation requirements. Where deemed necessary, horizontal drains can be included on the Plans and the following verbiage added to the Specifications:

Slotted PVC horizontal drains, if required, shall be installed as shown on the Plans. Install horizontal drains in drill holes that slope upward at an inclination of 2 to 5 degrees. Provide PVC horizontal drain pipe meeting the materials requirements set forth in the Standard Specifications. Attach a solid inlet pipe to the slotted pipe approximately 300 mm behind the back of the shotcrete facing. Seal the annular space between the inlet pipe and the drill hole with bentonite pellets or non shrink grout. Connect the inlet pipe to the discharge pipe that will empty directly into the footing drain, as shown on the Plans.

3.2 PERMANENT SHOTCRETE FACING

3.2.5 Specifier: Where the finished walls are constructed "top-down" in one full thickness layer, i.e. the shotcrete serves as the final facing, the contractor should demonstrate that his techniques will preclude sag or joint separation between excavation lifts. This typically occurs in walls thicker than 100 mm due to consolidation of the loose backfill material placed to provide a reinforcing splice below the lift. It has been demonstrated that joint separation can be mitigated by building the shotcrete in each lift from the bottom while allowing adequate time for consolidation of loose splice backfill.

3.2.6 Specifier. Specify the type finish to be applied. If the final shotcrete finish face is to be painted, such as with a pigmented sealer, add here or reference the applicable specification.

3.2.9 Specifier. Curing requirements for shotcrete are only necessary where the shotcrete is designed to be permanent and has to service the long-term wall loadings. Shotcrete for temporary construction walls does not require curing.

END COMMENTARY

APPENDC2.SI

APPENDIX D

FHWA GUIDE SPECIFICATION AND EXAMPLE PLAN DETAILS FOR SOIL NAIL WALL INSTRUMENTATION

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDD.SI
Last Revision 10/31/98

(An electronic version of this section is available at www.fhwa.dot.gov/bridge)

FHWA GUIDE SPECIFICATION FOR SOIL NAIL WALL INSTRUMENTATION

1.0 DESCRIPTION. This work shall consist of furnishing all instruments, tools, materials, and labor and performing all work necessary to install soil nail wall instrumentation in accordance with the Plans and this specification. Rounded english to metric unit conversions have been used in this specification. Commercial instrumentation devices and materials having metric dimensions within 2 percent of the specified dimensions will be acceptable.

Instrumentation provided under this section shall be designed, fabricated, and assembled in proper operating condition and in full conformity with the manufacturer's requirements and these Plans and Specifications. Each component shall be furnished complete with all components indicated on the Plans or specified herein, all accessories required for proper operation, and all additional materials of construction required by the design of the system.

This work shall include the installation and wiring of instruments to a readout panel. Wiring to the readout panel shall be completed after installation of each instrument, after each instrument is tested by the Contractor to the satisfaction of the Engineer, and prior to excavation of subsequent soil nail lifts. Instrument testing shall demonstrate that the system is working according to the manufacturer's specifications. Any monitoring device components that are damaged or fail, for whatever reason, to perform the intended function shall be immediately repaired or replaced by the Contractor to the satisfaction of the Engineer and at no additional cost.

Inclinometers shall be furnished and installed at the locations shown on the Plans. The inclinometers shall be installed and initial readings taken prior to soil nail wall construction. The Contractor shall adjust soil nail installations at these locations as necessary to avoid damaging the inclinometer casing. ***(See Commentary 1.0)***

All instrumentation shall be protected by the Contractor during the term of the contract and shall be replaced or restored at the Contractor's expense and to the satisfaction of the Engineer if delivered defective or damaged during construction. The Contractor installing the instrumentation shall coordinate and cooperate as necessary with the Engineer and the Specialty Contractor installing the soil nails.

Visual observations, survey point, and instrumentation readings will be made by the Engineer during the course of soil nail wall construction. The Contractor shall cooperate as necessary with the Engineer in facilitating these readings. Any monitoring data that indicates excessive structure deflections, the potential for unstable conditions or damage to adjacent structures or facilities, as determined by the Engineer, shall be cause for suspension of soil nailing operations in the affected area until the causes are identified and resolved to the satisfaction of the Engineer. Upon completion of the wall to grade, post-construction readings will be taken by the Engineer for up to 24 months or as determined by the Engineer.

1.1 Experience Requirements and Submittals. At least 15 days prior to start of the soil nail wall excavation, the Contractor shall submit in writing to the Engineer five copies of; (1) a list of proposed instruments including instrument and readout unit specifications; (2) complete and detailed installation procedures, including both the manufacturer's recommendations and the Contractor's step-by-step field procedures; (3) a wiring diagram detailing the wiring of the instruments to the central readout panels; and (4) shop drawings and specifications for ancillary equipment such as readout panels, load cell blockouts and covers, other protective covers, conduit, and enclosures.

The Contractor shall install the instruments utilizing a qualified geotechnical instrumentation specialist having experience in the design and installation of similar instrumentation systems on a minimum of 3 similar projects. The Contractor shall submit the resume of the individual(s) responsible for instrument installation and testing. The submittal shall include at least three references, with current telephone numbers, of persons who can verify the experience requirements. Due to the close coordination required between the soil nail Contractor and instrumentation specialist, the instrumentation specialist will install the instrumentation under the direction of the soil nail specialty Contractor.

The Engineer will approve or reject the Contractor's submittals within 10 calendar days after receipt of a complete submission. Soil nail wall construction shall not begin until the Engineer has approved the instruments, installation procedures, and personnel submittals.

2.0 MATERIALS.

2.1 General. All instruments shall be compatible with and calibrated using readout devices approved by the Engineer. ***(See Commentary 2.1a and 2.1b)***

2.2 Inclinometers: Inclinometers shall consist of 70 mm O.D., internally grooved plastic, aluminum, fiberglass, or steel casing in 3 meter lengths, provided with all necessary end plugs, caps, and couplings. The spiral twist of casing grooves in one 3 meter section of casing shall not exceed one degree. The top of each casing shall be provided with plastic cap and locking steel protective monument cover cap. Casing shall be as manufactured by Slope Indicator Company of Seattle, Washington; Carlson/R.S.T. Instruments Inc. of Yakima, Washington; Roctest Inc., of Plattsburgh, New York; Geokon Inc. of Lebanon, New Hampshire; or an approved equal.

The inclinometer probe shall be a biaxial sensor such as the Digitilt manufactured by Slope Indicator Company of Seattle, Washington; the inclinometer probe manufactured by Carlson/R.S.T. Instruments Inc. of Yakima, Washington; the Accutilt RT-20 manufactured by Roctest Inc., of Plattsburgh, New York; the model 6000 manufactured by Geokon Inc. of Lebanon, New Hampshire; or an approved equal. The probe cable shall be heavy duty, waterproof, and designed to support the weight of the probe without stretch, slippage, or creep. The cable shall be clearly marked at 300 mm intervals. The readout unit shall be compatible with the inclinometer probe. The probe and cable shall be serviced by the manufacturer as a unit at least 30 days prior to construction. ***(See Commentary 2.2a)***

Backfill around the inclinometer casing shall be a pumpable mix of water-cement-lime grout consisting of one bag (43 kg) of cement to three bags (68 kg) of hydrated lime. Other mixes may be used, if approved by the Engineer. ***(See Commentary 2.2b)***

2.3 Strain Gages: The strain gages shall be weldable vibrating wire gages. Model No. VK 4100, manufactured by Geokon Inc. of Lebanon, New Hampshire; IRAD Model SM-2W manufactured by Roctest Inc. of Plattsburgh, New York; VWS 1.1 Spot Weldable Gage manufactured by Geo Group Inc. of Gaithersburg, Maryland; or approved equal. ***(See Commentary 2.3)***

2.4 Soil Nail Load Cells: Soil nail load cells shall have an ultimate capacity not to exceed 450 kN. The load cells shall be center hole load cells with a minimum hole diameter of 38 mm. The load cells shall be the center hole load cell manufactured by Slope Indicator Company of Seattle, Washington; the 3000 or 4900 series manufactured by Geokon Inc. of Lebanon, New Hampshire; the HLC series manufactured by Geo Group Inc. of Gaithersburg, Maryland; the SGA or VWA series manufactured by Carlson/R.S.T. Instruments Inc. of Yakima, Washington; the ANCLO series manufactured by Roctest Inc. of Plattsburgh, New York; or approved equal. Load cells shall be temperature compensated or provided with temperature sensors as recommended by the manufacturer. ***(See Commentary 2.4)***

2.5 Readout Panels: The readout panels shall be of sufficient size and capacity to handle the specified number of instruments for each instrumented section. Each instrument shall have an isolated channel and shall be readily identified by waterproof labels resistant to vandalism and tampering.

2.6 Data Logger System: Data logger devices compatible with the instrumentation shall be provided for acquisition of strain gage and load cell data. The data logger shall be compatible with the strain gages and load cells installed without degrading the accuracy of the instruments. The data logger shall have programmable reading intervals, data storage, and capability of downloading to a computer. Software to communicate with the data logger, and for downloading data, shall also be provided. The data logger shall be fully programmed for the project with software customized to this particular system and application and shall be compatible with the Owner's portable PC system. The Owner's personnel shall be trained in the use of the data acquisition system to the satisfaction of the Engineer.

3.0 CONSTRUCTION REQUIREMENTS

3.1 General: The Engineer shall be notified prior to any work on instrument installation and monitoring. All instrumentation field installations shall be performed in the presence of the Engineer. All instruments requiring wiring shall be wired to a readout panel adjacent to the instrument station. Each instrument shall be wired to the readout panel after the instrument has been installed and demonstrated by the Contractor to be functioning in accordance with the manufacturer's specifications.

3.2 Inclinometers: Inclinometer casing shall be installed in vertical drill holes and fully grouted in place to the depths and at the locations shown on the plans. Drilled hole diameter shall be not less than 96 mm. Inclinometer installation shall be completed at least one week prior to the beginning of wall excavation. One of the casing grooves shall be aligned normal to the wall to a tolerance of +/- 5 degrees throughout the length. Casings adjacent to the soil nail wall shall be installed to a minimum penetration of 5 meters below the wall base. ***(See Commentary 3.2)***

3.3 Strain Gages: Installation and protection of the strain gages and connections shall be in accordance with the manufacturer's specifications. Each strain gage shall be field tested to verify that it is fully operational prior to mounting on the nail. Defective gages shall be rejected.

Encapsulation corrosion protection shall not be required for the instrumented nails and plan details. Each instrumented nail shall be epoxy coated per the soil nail wall materials specification. The epoxy coating shall be removed as necessary to install the gages. The gages shall be mounted to the bar in pairs at each location shown on the Plans. All gage pairs shall be mounted on opposite sides of the bar 180 degrees apart. All gage pairs shall be mounted in the same plane. The end of each nail bar shall be inscribed along the plane of orientation of the strain gages.

All gages, sensors, and wire assemblies shall be protected from moisture. All wire connections shall be of an approved waterproof type and shall be fitted with at least two waterproof, tamper-resistant labels spaced three meters apart at the readout panel end of the wire. Signal cables shall not be spliced unless approved by the Engineer. Centralizers for the instrumented nails shall be used to ensure that the bar is located within 25 mm of the center of the drillhole. The nail shall be installed so that the final locations of the gage pairs are at the 6- and 12-o'clock positions +/- 10 degrees.

3.4 Nail Load Cells: The load cell shall be mounted on the nail between the bearing plate and the nut as shown on the Plans. All bearing surfaces shall be clean. Spherical bearings shall be well-lubricated with suitable grease. The Contractor shall attach the cells and protect the connections according to the manufacturer's specifications. All wire connections shall be of an approved waterproof type.

A 300 mm square or circular blockout shall be provided in the cast-in-place wall facing for the load cell assembly. A steel cover plate shall be installed over the blockout to protect the load cell. The cover plate shall be painted, galvanized, or otherwise protected from corrosion. The cover plate shall be installed in such a manner as to allow easy future access to the load cell. ***(See Commentary 3.4)***

3.5 Readout Panels: One readout panel shall be located at each instrumentation section unless otherwise approved by the Engineer. The readout panel shall be attached to a steel or treated wooden post that is firmly secured in the ground and located a distance of approximately 1 meter behind the top of the nailed wall or at another convenient location as directed or approved by the Engineer. All instrumentation wiring to the readout panel shall be done in accordance with the manufacturer's recommendations. The readout panel shall be securely sealed and shall be rated NEMA 4X or better. The readout panel shall be protected from vandalism and tampering. All above-ground wiring shall be enclosed in a steel conduit that is firmly attached to the readout panel. All instrument readout panel wiring shall be done during instrument installation.

4.0 METHOD OF MEASUREMENT: No separate measurement will be made for the materials and work specified in this Section. The unit of measurement for Soil Nail Wall Instrumentation will be lump sum.

5.0 BASIS OF PAYMENT: Soil nailing instrumentation will be paid for at the contract lump sum amount for the item Soil Nail Wall Instrumentation. Payment will be full compensation for furnishing all materials, labor, equipment, tools, and incidentals necessary to complete the work as specified in this Specification and as shown on the Plans.

Upon satisfactory installation and final acceptance by the Engineer, all instruments and readout units furnished and installed under this Section shall become the property of the Owner.

Pay Item	Measurement Unit
Soil Nail Wall Instrumentation	Lump Sum

END OF SECTION

APPENDD.SI

COMMENTARY TO SOIL NAIL WALL INSTRUMENTATION SPECIFICATION

1.0 DESCRIPTION

1.0 Specifier. In some cases, the Owner may elect to install the inclinometers prior to the start of the Contractor's activities. In this case, the words "by the Owner" should be added after "furnished and installed". Sections 2.2 and 3.2 should be deleted if the inclinometers are installed by the Owner.

2.0 MATERIALS

2.1a Specifier. If the Owner already has in his possession a particular readout device and/or data logger, the specifications should require the instruments to be compatible with this equipment.

2.1b Specifier. The following materials sections refer to the specified instruments by trade name and part number (prescriptive specification). This is not intended to require a sole source procurement of instruments but rather to ensure installation of the correct instrument for the desired application. As such, each trade name instrument can be substituted with an approved equal. Owners and Engineers should assure themselves that a true equal has been provided.

2.2a Specifier. Several factors should be considered when specifying a readout unit for inclinometers. Some inclinometer readout units from one manufacturer will work with probes from another manufacturer. The Owner may already have a readout unit and data reduction software. Some projects may generate a considerable amount of data, and automated recording and downloading (which requires a more expensive readout unit) may be justified. If necessary, the specification for the inclinometer readout unit should be modified to reflect any particular requirements of the project.

2.2b Specifier. The backfill around the inclinometer casing must completely fill the annulus. For this reason, pouring sand into the top of the borehole is generally not recommended due to potential bridging problems. Pumping a cement or bentonite grout upward from the bottom of the borehole is preferred. The grout must provide adequate support so that the casing deforms in the same way as the surrounding ground. However, it must not be so strong as to function structurally. Particularly in soft soils, relatively strong cement backfills could resist ground movement and under-register or redistribute the actual deformation. Proposed backfill materials should be reviewed by the Engineer to ensure that they are suitable for the ground conditions.

2.3 Specifier. As discussed in Chapter 6 of this manual, if posible, use of a grout breaker assembly at the strain gage location is recommended to reduce the uncertainty associated with interpreting strain gage data. The prototype assembly shown on figure D-2 is presented as an example of one possible method. To date this prototype assembly has only been tested in the lab and was specially fabricated and is not readily commercially available. Other approaches may be equally (or more) effective, depending on project-specific conditions such as type of soil, installation method, and available budget. The purpose of the grout breaker assembly is to introduce a break *in the grout column across which no tension can be transmitted except through the nail bar. To accomplish this, the breaker must occupy a sufficient percentage of the cross-sectional area of the borehole to disrupt the grout column. On the other hand, grout must be able to pass through or around the breaker if tremied from the bottom of the hole or if the instrumented nail is pushed into a grout-filled hole. Therefore, this type of assembly will work best with neat cement grout. Thick sanded grout mixes will make this operation more difficult. The example shown on figure D-2 utilizes the principle of two disks butted together. Alternatively, a single disk coated with release agent could be used. Another potential approach involves installing a much less expensive styrofoam disk around the strain gages (the styrofoam should have a low stiffness). Other approaches may be possible. In any case, the Engineer should coordinate with the Contractor to select an approach that can be expected to break the grout column with a high degree, yet is feasible to install under field conditions.*

2.4 Specifier. The load cell capacity of 450 kN will be suitable for most soil nailing projects. In the rare situation where the wall design will require higher nail loads, a higher capacity load cell should be specified.

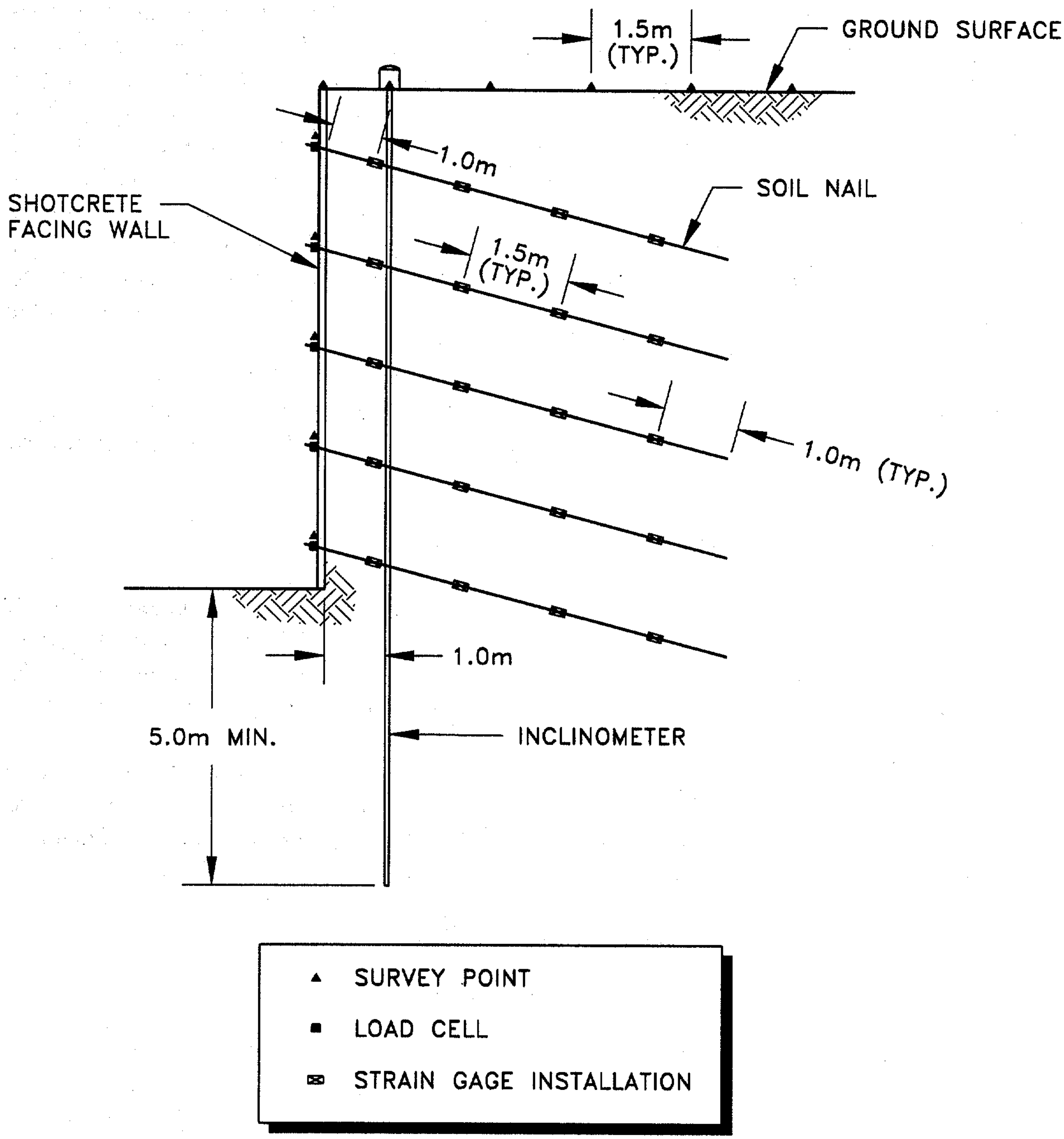
3.0 CONSTRUCTION REQUIREMENTS

3.2 Specifier. An example of a protective enclosure for the top of an inclinometer casing is shown on figure D-1. Different configurations may be required for additional protection, where surcharge will be placed, where traffic may be present, or for other project-specific conditions. The associated materials and installation requirements should be specified to the extent necessary.

3.4 Specifier. An example load cell installation is shown on figure D-1. While it is desirable to entirely recess the load cell as shown, this may not always be possible if the CIP wall is thin or the load cell is large. Covers, enclosures, and other protective equipment should be adequately designed and specified to provide protection under project conditions.

END OF COMMENTARY

APPENDD.SI

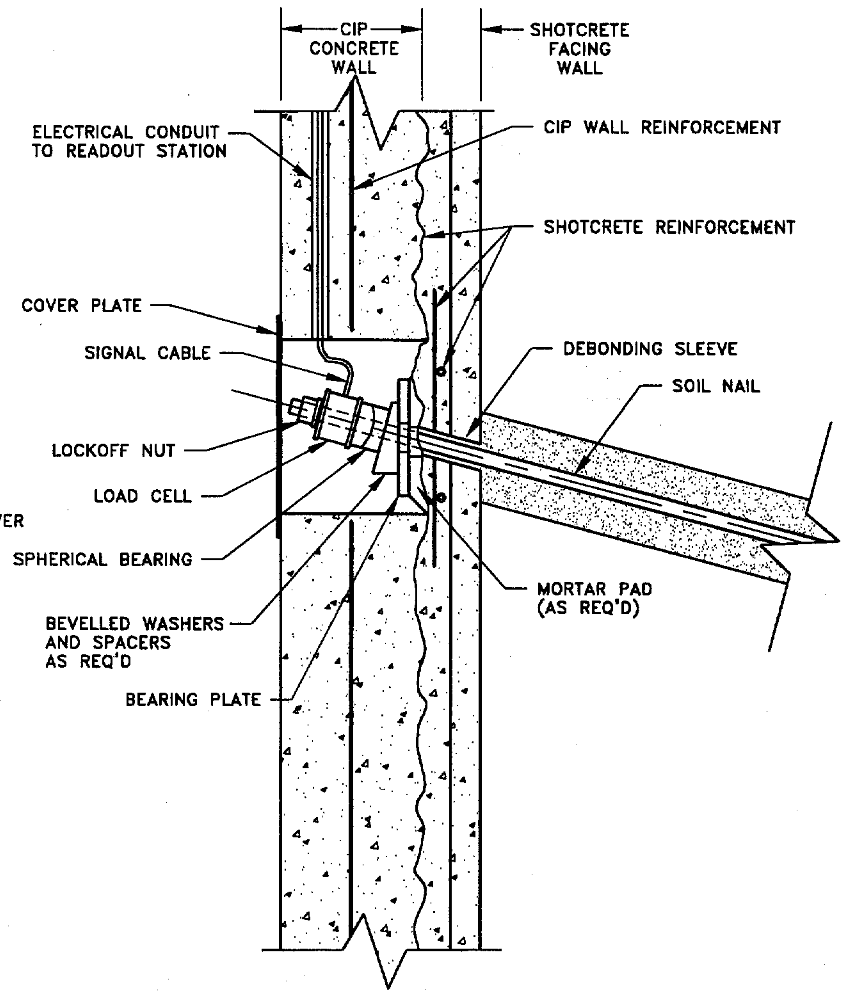


NOTE:
Number and location of instruments
may be adjusted to suit field conditions.

TYPICAL INSTRUMENTATION LAYOUT

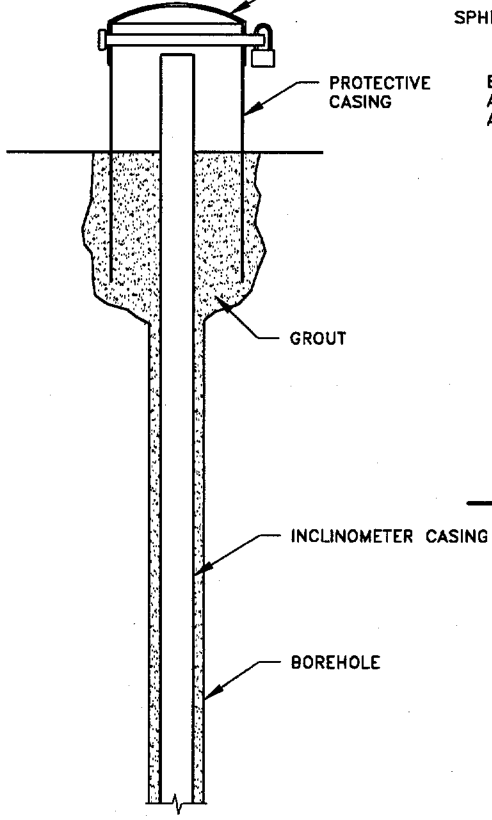
NOT TO SCALE

FIGURE **D-1A**
INSTRUMENTATION LAYOUT AND DETAILS



LOAD CELL INSTALLATION DETAILS

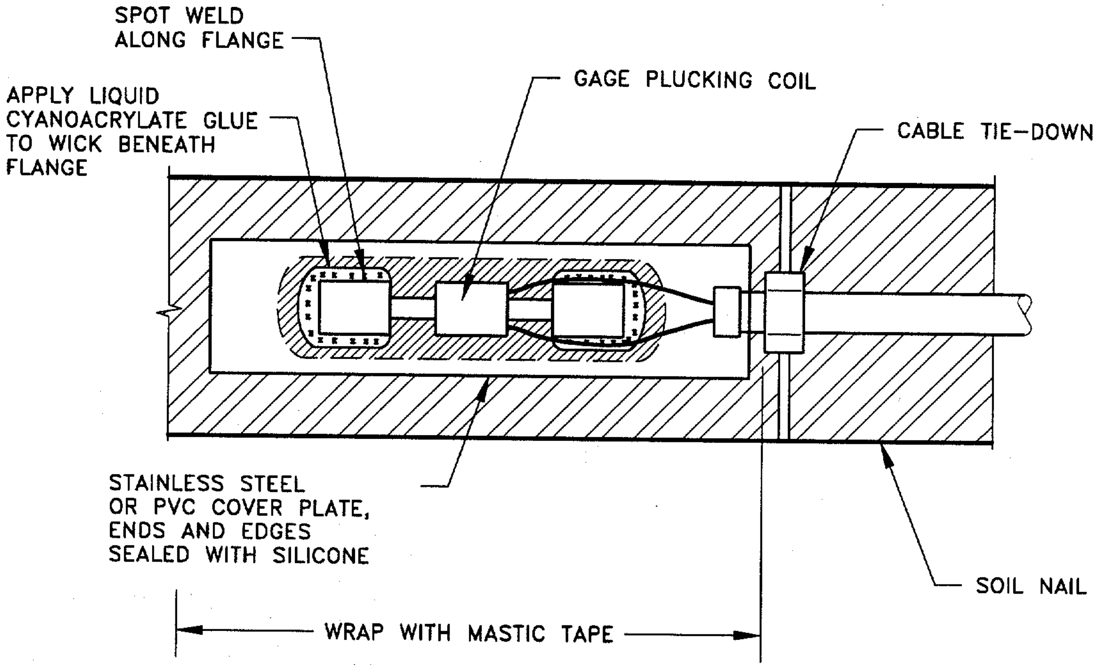
NOT TO SCALE



INCLINOMETER INSTALLATION DETAILS

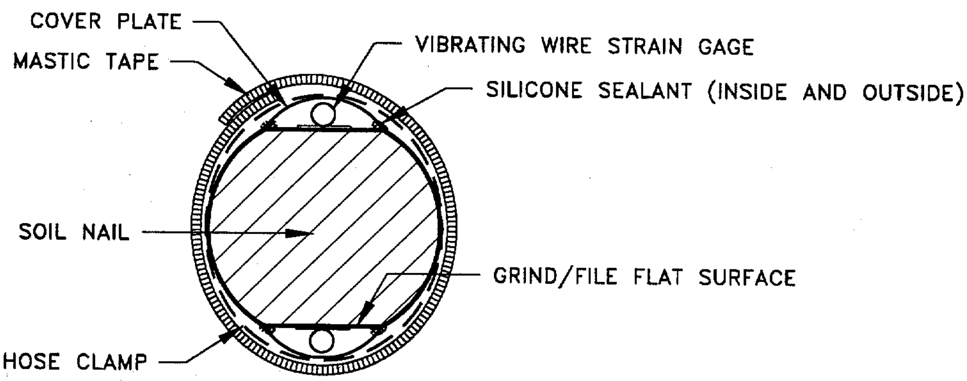
NOT TO SCALE

FIGURE **D-1B**
INSTRUMENTATION LAYOUT AND DETAILS



STRAIN GAGE INSTALLATION

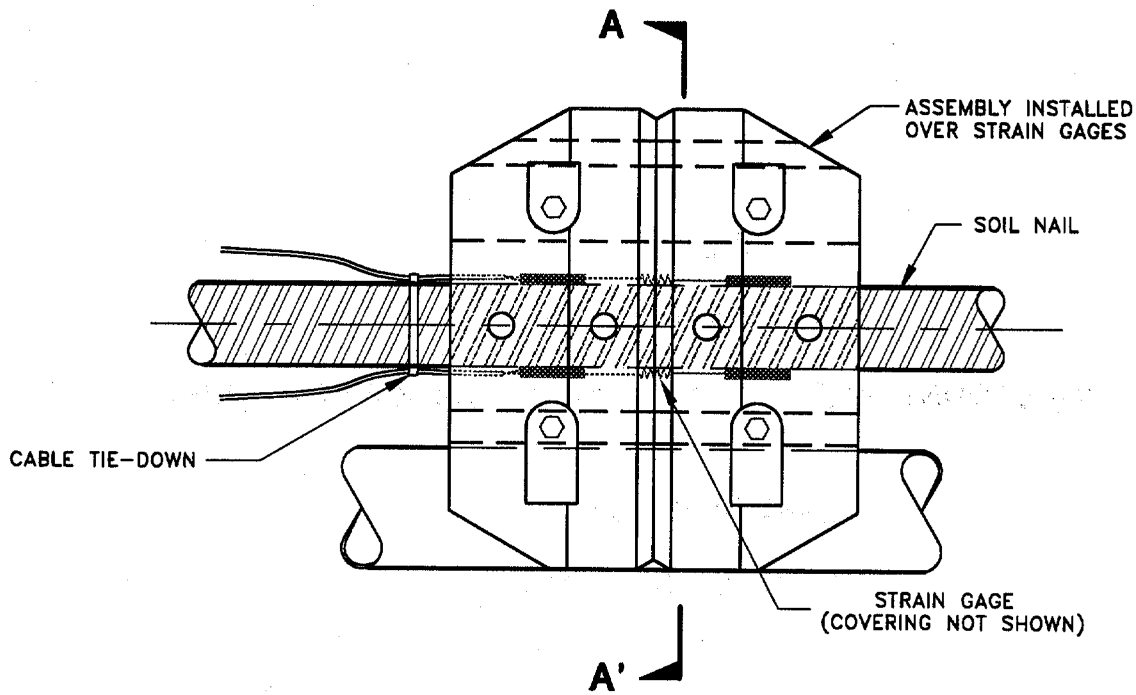
NOT TO SCALE



SECTION THROUGH STRAIN GAGES

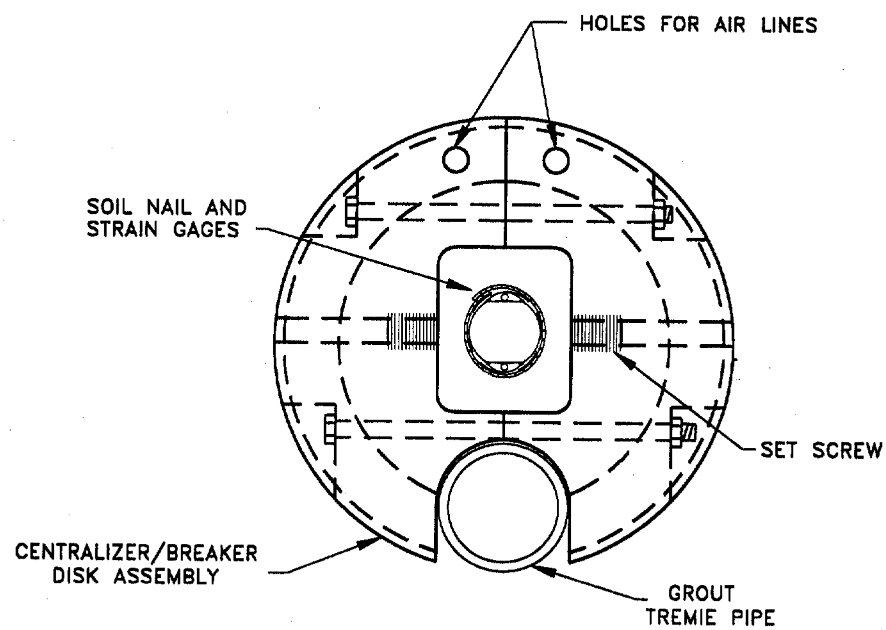
NOT TO SCALE

FIGURE **D-2A**
STRAIN GAGE INSTALLATION DETAILS



PROTOTYPE CENTRALIZER/GROUT BREAKER DISK ASSEMBLY

NOT TO SCALE



SECTION A-A'

NOT TO SCALE

FIGURE **D-2B**
STRAIN GAGE INSTALLATION DETAILS

APPENDIX E

SOIL NAIL WALL

DESIGN AND CONSTRUCTION QUALITY ASSURANCE CHECKLIST

METRIC (SI) UNITS

(WITH COMMENTARY)

APPENDE.SI
Last Revision 10/31/98

SOIL NAIL WALL
DESIGN AND CONSTRUCTION QUALITY ASSURANCE CHECKLIST

E.1 Site Characterization

The following items should be addressed in characterizing the suitability of the site for soil nailing and, for suitable sites, in defining the required design parameters.

Reconnaissance

The reconnaissance phase of the site characterization program is concerned primarily with scoping the overall project and determining the suitability of the site for a soil nailing application. The following is recommended:

- Define the ground type(s) and groundwater conditions present over the area to be reinforced by soil nails. This is achieved by surface mapping, inspection of landforms and outcrops, mapping of existing excavations in the immediate region of the site, and review of existing subsurface data in the area of the site.

- Determine the general technical suitability of the ground conditions for soil nailing. Suitable conditions include weathered rock; granular materials with adequate stand-up time as a result of density, some clay content, natural cementation or capillary cohesion; and stiff cohesive soils (silts and low to moderate plasticity clays) that are not prone to creep or excessive swelling/shrinking. Unsuitable conditions include loose/uniform cohesionless soils that do not exhibit the required stand-up time; ground below the water table, and soil that is sufficiently saturated such that wet zones cause face stability problems; low strength, highly plastic cohesive soils that creep and do not bond satisfactorily with the nails; highly frost susceptible soils and swelling soils; and highly fractured rock with adverse structure that will cause face stability problems.

- Identify any right-of-way restrictions and easement requirements for both the surface and subsurface components of the soil nail wall.

- Identify existing and future facilities that might physically interfere with the soil nail wall construction (e.g., buried utilities) or that could potentially be adversely impacted by construction or exploration (e.g., adjacent buildings).

- Record sites access conditions for both exploration and construction.

- Record surface drainage patterns to estimate surface water control requirements.

- Define the lateral and depth limits for more detailed site exploration activities, should the reconnaissance indicate the technical, economic and environmental suitability of the site for soil nailing.

Exploration and Testing

The detailed exploration and testing phase of the site characterization program is concerned with developing a design subsurface model of the site (e.g., ground stratigraphy), performing field tests to determine the constructibility of the wall, and determining the values of the parameters required for the soil nail wall design. The following activities are typically undertaken:

- Excavate test pits in representative material types to evaluate stand-up time, especially soils with inadequate short-term cohesion or residual soils with adversely oriented structure.

- Drill exploratory holes along the proposed wall and behind the wall line. Perform standard or cone penetration tests, obtain disturbed samples for classification testing, and take undisturbed tube samples if cohesive soils are encountered.

- Note groundwater occurrences and install piezometers if a water table is encountered at depths that will impact the soil nail wall construction.

- Perform laboratory or field tests to determine soil/rock unit weights. Determine material shear strengths based on laboratory and/or field testing, field correlations with SPTs/CPTs/Rock Quality, and/or local experience.

- Perform electrochemical tests to evaluate the aggressivity of the ground environment, in order to determine the nail corrosion protection requirements.

- Estimate ground-grout pullout resistance values based on local experience, published information, or correlations with in situ tests. These estimates may be suitable for design but must be checked by field pullout testing during construction. For critical structures, large projects or unusual ground conditions, a separate design phase field nail pullout test program to better define actual ground-grout pullout resistance values should be considered.

E.2 Soil Nail Wall Design

The following activities should be undertaken for the design of a soil nail retaining wall, once the technical, environmental and cost feasibility has been established and the preliminary design information has been gathered from the site characterization program.

- Prepare a layout of the wall, showing its location in plan, together with other details of the wall facing geometry such as required height, face batter angle, stepped facing, etc.

- **Accurately survey the ground elevation along the proposed top of wall, so that areas of cut, fill and regrading can be identified and the location of the top row of nails can be established in detail along the entire length of the wall. This information should be**

obtained for use in design. Failure to provide this information has been one of the most common causes of design changes and construction delays during soil nail wall construction startup on many projects.

- Identify the critical design cross-sections along the length of the wall, based upon the ground profile behind the wall, the wall height and geometry, any special surcharge loading conditions, and the distribution of the ground stratigraphy/strengths/pullout resistances established from the site characterization program.

- Prepare an initial trial nail layout (length, spacing, declination, nail tendon diameter and steel grade, typical drillhole diameter) based on experience, preliminary analyses, or the use of preliminary design charts. Nail declinations should be no steeper than the minimum required for constructibility, taking account of subsurface obstructions. Nail spacings should be as large as possible consistent with face stability during construction, but generally no greater than 2 meters. Drillhole size should consider the type of drilling equipment likely to be used and minimum requirements based on the estimated nail grout-ground pullout resistance.

- Identify initial shotcrete construction facing and final permanent facing designs, including reinforcement requirements and details of the connections between the facings and the nails. The permanent facing design should consider the requirement for special protection of the facing against frost loading, in frost-susceptible soils in cold climates.

- *Determine the design strengths of the nail heads, considering the potential for shear and flexural failure of the wall facing (for both the temporary shotcrete construction facing and final permanent facing), and the potential for failure of the facing-nail connector system. The connector system should address the potential for failure in the following modes: punching or cone shear through the facing; flexural failure of the facing; and tensile strength of headed studs, if used with the permanent facing.*

(Note that the above section in italics will be revisited once the detailed design is finalized)

- Determine the design strength of the nails, as a function of the location along the length of the nail, based on the nail head strength, the nail grout-ground pullout resistance, and the tensile strength of the nail.

- Determine the suitability of the design by calculating the minimum factor of safety for the global stability critical failure surface, taking account of the soil strengths and the reinforcement contribution of the trial nail pattern. Iterate on the nail layout (spacing and/or length) and/or the facing designs until a satisfactory factor of safety is indicated. Consider all potential loading conditions including dead loads, live loads, external surcharge loads and seismic loads.
- Check nail head/facing designs for serviceability and ductility requirements.

- Check designs of the upper cantilevers for both the construction and the permanent facings.

Check maximum allowable height of construction facing upper cantilevers.

- Determine the final facing reinforcement details including reinforcement distribution requirements, minimum and maximum reinforcement ratios, minimum cover requirements and reinforcement development and splice requirements.

- Check estimated excavation induced service deflections of the wall.

- Determine the corrosion protection requirements for the nail and the nail head, based on the ground conditions and the location/nature of the structure.

- Define surface water control measures, methods for handling subsurface water encountered during construction, and permanent wall face and subsurface drainage requirements.

E.3 Plan Preparation

The design drawings should be prepared, based on the above design factors, and taking account of the following construction related issues:

- Nail layout pattern.

- Nail locations with respect to top and bottom of wall, end of wall, and permanent facing construction/expansion joints.

- Nail row grades.

- Uniform nail spacing and inclination, where possible.

- Maximum upper cantilever height for the construction facing (i.e., to avoid rotational failures of facing during the initial stages of construction).

- Use of temporary open cuts for loose upper soils, utility avoidance, etc.

- Location of topmost row of nails with respect to construction head-room (e.g., beneath bridge decks).

The following minimum information should be included on conceptual level plans for owner design or design/build contracts.

- Geometric Requirements
  - Wall plan and profile.
  - Beginning and end of wall stations.
  - Wall alignment topographic survey.
  - Existing and finish grade profiles both behind and in front of the wall.
  - Cross sections showing the limits of construction at the retaining wall location intervals of 15 m or closer.
  - Horizontal and vertical curve data and wall control points.
  - Required wall appurtenances such as traffic barriers, coping, drainage, etc.
  - Right-of-way and permanent or temporary construction easement limits, location of existing and future utilities, adjacent structures or other potential interferences.
  - Staged excavation sequencing.
  - Quantity tables showing estimated square meters of wall areas and other pay items.
- Structural Requirements
  - Conceptual details should be shown of facing reinforcement and of connectors between the nail and the facing.
  - Level of corrosion protection required (none, fusion bonded epoxy or full encapsulation).
  - Limits and requirements for drainage features beneath, behind or through the structure.
  - Facing finishes, color and architectural requirements for wall facing elements.
  - Required nail and facing reinforcement; steel grades and strengths; and, shotcrete, concrete, and nail grout strengths.

- Geotechnical Requirements

    - 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 wall.

    - Graphic logs of borings and test pits and a generalized description of each deposit, including soil/rock classification, color, density, moisture, plasticity, rock RQD, groundwater levels, SPT N values, and logs of CPT soundings (if performed). Refer to availability of subsurface investigation report.

    - Subsurface cross sections adequate to define representative conditions in front of and behind the wall, along the full wall length.

    - Advisory notes to describe anticipated difficult installation conditions (e.g. cobbles and boulders or anticipated marginal excavation face instability) or to warn the contractor of known latent subsurface conditions such as existing foundations, utilities, obstructions, etc.

- Design Requirements

    - Applicable code requirements.

    - Magnitude, location, and direction of external loads, surcharges, piezometric levels, etc.

    - Reference to design methods to be utilized or other provisions such as minimum nail lengths/diameters and minimum shotcrete construction facing wall thickness.

    - References for design of facing and nail-facing connectors.

    - Design parameters including seismic coefficients, together with estimated soil/rock shear strengths (friction angle and cohesion), unit weights, and design nail grout-ground pullout resistances for each ground type.

    - Minimum partial safety factors (for Service Load Design) or load and resistance factors (for Load Resistance Factor Design) to be used on the soil/rock shear strength and pullout resistance; surcharges; unit weights; and steel, shotcrete, and concrete materials.

For final plans, the following additional information should be included.

- General notes specifying construction sequencing or other special construction requirements.
- Special notes to indicate cross sections or specific nails that may require special treatment.
- Nail wall typical sections.
- Nail details including spacing, size, inclination, and corrosion protection details.
- Details, dimensions, and schedules for all nails, reinforcing steel, wire mesh, bearing plates, etc. and/or attachment devices for cast-in-place or prefabricated facings.
- A typical cross section of production and test nails defining the nail length, minimum drillhole diameter, inclination, and test nail minimum required bonded and unbonded test lengths.
- Wall elevation view showing nail locations and elevations; vertical and horizontal spacing; and, the location of wall drainage elements.
- A wall reference baseline and elevation datum to fix all wall and nail locations.

E.4 Specification Preparation

The following items should be addressed in the specification:

- General scope of work.
- Contractor experience qualification requirements.
- Submittals which the contractor must provide.
- Design parameters including, estimated soil/rock shear strengths (friction angle and cohesion), soil/rock unit weights, pullout resistances and seismic coefficient.
- Material specification requirements, including nail corrosion protection requirements.
- Materials handling and storage requirements.
- Percentage of nails to be tested, testing procedures and acceptance criteria.
- Excavation tolerances and sequencing, including the maximum allowable height of

excavation lifts.

- Dewatering/drainage requirements.

- Maximum time duration of finish cut face exposure prior to nail installation and closure with structural shotcrete.

- Minimum required nail grout and shotcrete strengths prior to allowing excavation to proceed to the next lift.

- Construction quality control testing requirements.

- Wall construction monitoring requirements.

- Soil nail and wall facing construction tolerances.

- Records requirements.

- Method of measurement and basis of payment.

E.5 Construction Monitoring and Testing

(Note: For more in-depth coverage of construction inspection and testing, including inspection checklists and example forms for nail pullout testing, refer to FHWA-SA-93-068, "Soil Nailing Field Inspector's Manual")

Field Quality Control of Materials

- For steel components, centralizers, and drainage materials, check all Certificates of Compliance and mill test certificates for compliance with the specifications.

- Visually check all soil nail tendons and facing reinforcing steel for damage and defects upon delivery and prior to use.

- Visually check epoxy coated or encapsulated soil nail tendons for compliance with the specifications and for any damage to the corrosion protection.

- Confirm mix design compliance of soil nail grout and facing shotcrete.

- When specified, take grout (cubes) and/or shotcrete samples (test panels and cores) for testing.

- Verify adequacy of field storage of construction materials to prevent damage or degradation.

Excavation

- Prior to start of any wall construction, check for any variance between the actual ground surface elevations along the wall line and those shown on the plans.

- **Verify that the Contractor is providing accurate field survey control of the final wall face excavation and shotcrete facing alignment and thickness. This is especially critical for the upper initial shotcrete lift, since the excavation and facing control for the following lifts will typically guide off the upper lift.**

- Frequently determine that stable excavation conditions are being maintained both for general mass excavation and for wall face final excavation. Make daily inspections of the ground next to the wall excavation including behind the top of wall.

- Verify that excavations are constructed within specification tolerances of the design line and grade.

- For each excavation lift, verify that the contractor is not overexcavating.

- Verify that the encountered ground conditions are consistent with those anticipated.

Soil Nail Hole Drilling

- Document construction on the "Soil Nail Installation and Summary Forms" contained in FHWA SA-93-068, "Soil Nailing Field Inspector's Manual".

- Verify that the soil nail hole is drilled within acceptable tolerances of the specified alignment, length, and minimum diameter.

- Verify that loss of ground or drillhole interconnection is not occurring.

Nail Bar Tendon Installation

- Inspect open soil nail holes for caving or excess loose cuttings using a high intensity light.

- Verify that tendons are inserted to at least the minimum specified length.

- Verify that centralizers are installed at the specified intervals and will provide clearance for the minimum specified grout cover and that openings through the centralizer support arms are not obstructed.
- Carefully observe that workers handle and insert the tendons carefully to prevent damage to the corrosion protection.

Grouting

- Verify that grout is batched in accordance with approved mix designs.
- In open drillholes, verify that grout is injected by tremie pipe starting at the bottom of the hole, and that the end of the tremie pipe always remains below the level of the grout as it is extracted.
- Verify that grout continues to be pumped as the grout tube, auger, or casing, is removed.
- Verify that the contractor does not reverse the auger rotation while grouting through hollow-stem augers, except as necessary to initially release the tendon.
- Record the grout pressure when pressure grouting is used.
- Confirm that any required grout strength test samples have been obtained in accordance with the specifications. These are typically 50x50 mm grout cubes.
- Verify the bonded and unbonded lengths of test nails.

Shotcrete Facing and Drainage

- Verify that the geocomposite drain strips and PVC connection/weep hole outlet pipes are securely installed and as specified and that drain elements are interconnected and provide continuous drainage paths.
- Verify that the reinforcing steel has been installed at the locations and to the dimensions specified. Particular care should be given to ensure it is tied securely and is clean.
- Verify that wall thickness, finish line and grade will be in accordance with the plans and specifications.
- Verify that shotcrete will be applied and cut face closure will occur within the specified time limits.
- Verify that construction joints are clean and acceptable for shotcrete placement.
- Verify that shotcrete is batched in accordance with the approved mix design and that the specified strength requirements are met.
- Verify that shotcrete test panels and/or cores (if specified) are prepared, cured, and transported to the testing lab in accordance with specifications.

- Verify that shotcrete is applied as specified, and in accordance with recommended good practice.

- Verify that the bearing plates and nuts/washers are installed as specified.

- Verify that the specified finish is provided immediately after shooting and curing/protection (if required) are promptly implemented.

- Verify that expansion joints (if required in permanent facings) are installed per the plans and specifications and that nails do not encroach within 300mm of expansion joints.

- Verify that cold or hot weather shotcreting is conducted in accordance with the specifications.

- Verify that footing drains are constructed in accordance with the plans/specifications.

Nail Testing

- Make copies of all appropriate test forms (sample forms are provided in appendix A of the FHWA "Soil Nailing Field Inspector's Manual") and record all readings and other pertinent information during testing. Be certain to accurately record the test nail identification number, station, and elevation on the test form.

- Verify that the ground above the soil nail and behind the structure has been graded as required by the plans, prior to testing upper row nails.

- Verify nail properties necessary to calculate elastic elongation, i.e., bar steel grade, cross-sectional area, and unbonded test length.

- Ascertain that test loading in excess of the allowable tendon structural strength will <u>not</u> occur.

- Verify that test nail length is sufficient to accommodate all test equipment or use a coupler if allowed.

- Verify the bonded and unbonded lengths of the test nail.

- Verify that the dial gauges are in proper working order (i.e., not broken or sticking) and have an appropriate travel length (50 mm is a recommended minimum).

- Verify that the jack is in good working order, that the jack and pressure gauge have been calibrated as a set, and that a calibration graph is provided. If a load cell is required to maintain constant load during creep testing, a calibration graph should be provided for that as well. Verify that the identification numbers on the field equipment match the identification numbers on the calibration data sheets.

- Prior to conducting the pullout test, calculate the required maximum test load and list all test load increments (and the corresponding jack pressures) on the test form.

- Verify that the jack bearing pads will not interfere with the nail/grout column during testing.

- Verify that the jack can incrementally load and unload the tendon (i.e., that the jack or pump has a bleed-off valve).

- Verify that the load cell (if used) and jack are in alignment with each other and with the soil nail tendon.

- Verify that the dial gauges are aligned within 5 degrees of the axis of the soil nail and the gauges are mounted independent of the nail and testing apparatus. Also verify that the dial gauges do not walk excessively on the tendon reference plate. This observation may be made by scribing a circle on the reference plate around the gauge head after the alignment load is applied.

- Verify that the jack does not drop onto the soil nail or lie on it. This could cause bending of the soil nail tendon, or eccentric loading of the tendon during testing.

- Verify that the minimum alignment load is maintained at all times.

- Periodically verify that interference between the jacking set-up and the nail tendon has not occurred due to misalignment. This could lead to erroneous readings.

- Verify that constant load is maintained during the creep test. The load should be held within 175 kPa if a jack pressure gauge is used, or within 1 kN if a load cell is used.

- Verify that load increments are applied and held within the specified time limits for the test.

- Verify that the unbonded test length has been properly filled with grout after completion of the test for <u>all</u> test nails, including sacrificial nails. No voids should be left in the ground.

CIP Face Form Connection to Shotcrete Construction Facing (if applicable)

- When anchors embedded into the shotcrete facing will be used to support a 1-sided CIP face form, verify that pullout testing of the anchors is performed in accordance with the specifications.

END OF SECTION

APPENDE.SI

APPENDIX F

NAIL HEAD STRENGTH CALCULATIONS

NAIL HEAD STRENGTH CALCULATIONS

The main objectives of this appendix are to provide example hand calculations and to present design summaries for the nominal nail head strengths of soil nail wall facing systems using the methodology described in section 4.5.

First, example calculations are presented for both a temporary shotcrete construction facing and a permanent cast-in-place (CIP) concrete facing. The details of both facings are illustrated on figure F.1.

Second, the nominal nail head strengths have been determined for a number of temporary shotcrete facings used in practice. The results are tabulated for easy reference for use as a preliminary design tool. Final designs should always be verified by performing the calculations described in section 4.5.

Table F.1 shows the dimensions of common stock styles of welded wire fabric. Table F.2 shows the dimensions of available reinforcing bars, specified by both their english and soft metric designations. Table F.3 shows the dimensions of common stock size headed studs.

F.1. EXAMPLE CALCULATIONS

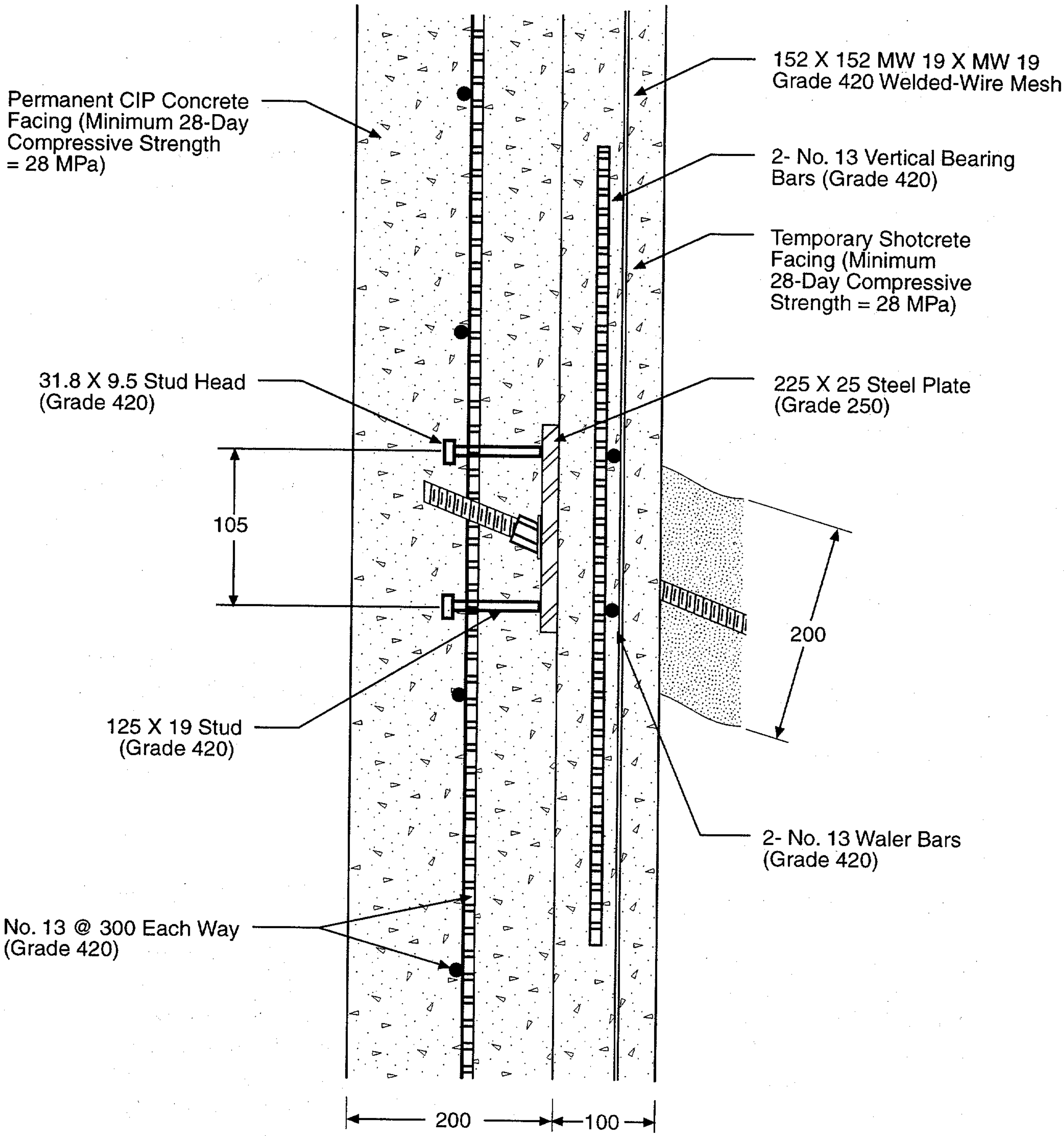
The example problem of chapter 5 specifies a 100-mm-thick shotcrete construction facing and a 200-mm-thick permanent cast-in-place (CIP) concrete facing (figure F.1). The nail head strengths for the temporary facing are computed first, after which the permanent facing is examined.

F.1.1 Temporary Shotcrete Construction Facing

(a) Strength Criteria 1: Facing Flexure (section 4.5.2, equation 4.1)

Based on soil and constructability considerations as well as local practice, the vertical and horizontal nail spacings have both been selected to be 1.50 meters. The connection plate will be 225 mm wide and 25.0 mm thick. The facing reinforcement is selected by considering the available mesh sizes, local practice, and strength requirements. Try a 152x152xMW18.7xMW18.7 mesh, with two No. 13 (Soft Metric designation) waler bars and two No. 13 vertical bearing bars. All the facing steel is assumed to be nominally located at the center of the section. The yield stress of the reinforcement is specified as 420 MPa and the specified design concrete compressive strength at 28 days is 28 MPa.

The first step to evaluate equation 4.1 is to compute the negative and positive nominal unit moment resistances of the facing in the vertical direction. The equation for the nominal unit moment resistance of a singly-reinforced, rectangular-shaped reinforced-concrete beam is as follows [1]:



All dimensions in millimeters

Figure F.1 Facing Details

$$m_{V,POS} = m_{V,NEG} = 17.4 \text{ kN-m/m}$$

From table 4.2, the facing flexure pressure factor C_F for a 200-mm-thick permanent facing is 1.0. Substituting the corresponding values into equation 4.1, the nominal nail head strength for the criteria of facing flexure may be computed as follows:

$$T_{FN} = 1.0(17.4 \text{ kN-m/m} + 17.4 \text{ kN-m/m})(8)\left(\frac{1.50 \text{ m}}{1.50 \text{ m}}\right) = 278 \text{ kN}$$

(b) Strength Criteria 2: Facing Punching Shear (section 4.5.3, equation 4.3)

The dimensions of the headed studs must be selected in order to calculate the geometry of the potential punching cone. Per section 4.5.4, the headed-stud dimensions are first selected to satisfy the provisions of ACI Committee 349 [2]. Try a 22.0 mm body diameter, a 35.0 mm head diameter, a 9.5-mm head thickness, an overall length of about 125 mm (corresponding to a $^7/_8$x$5^3/_{16}$ anchor size, table F.3), and a stud spacing of 105 mm. The provisions are checked as shown below:

<u>Provision 1</u>:

$$\begin{aligned} d_H &\geq \sqrt{2.5}\, d_{HS} \\ 35.0 \text{ mm} &\geq \sqrt{2.5}\,(22.0 \text{ mm}) \\ 35.0 \text{ mm} &\geq 35.0 \text{ mm} \qquad \text{(OK)} \end{aligned}$$

<u>Provision 2</u>:

$$\begin{aligned} t_H &\geq 0.5(d_H - d_{HS}) \\ 9.5 \text{ mm} &\geq 0.5(35.0 \text{ mm} - 22.0 \text{ mm}) \\ 9.5 \text{ mm} &\geq 6.5 \text{ mm} \qquad \text{(OK)} \end{aligned}$$

The nominal internal punching shear strength of the facing is computed from equation 4.2, where h_C and D'_C are as indicated below:

$$h_C = 25 \text{ mm} + 125 \text{ mm} = 150 \text{ mm}$$

$$D'_C = S_{HS} + h_C = 105 \text{ mm} + 150 \text{ mm} = 255 \text{ mm}$$

The resulting nominal internal punching shear strength of the facing is computed to be:

$$V_N = 0.33\sqrt{28.0 \text{ MPa}}\,(\pi)(255 \text{ mm})(150 \text{ mm}) = 210 \text{ kN}$$

From table 4.2, the pressure factor for punching shear for a 200-mm thick permanent facing is 1.0. The punching cone bottom diameter D_C is equal to $D'_C + h_C = 405$ mm. The diameter of the grout column is estimated to be about 200 mm. The corresponding areas are computed as follows:

$$A_C = 0.25(\pi)(D_C)^2 = 0.25(\pi)(405 \text{ mm})^2 = 1.29 \times 10^5 \text{ mm}^2$$

$$A_{GC} = 0.25(\pi)(D_{GC})^2 = 0.25(\pi)(200 \text{ mm})^2 = 3.14 \times 10^4 \text{ mm}^2$$

Substituting the above values into equation 4.3, the nominal nail head strength for the criteria of punching shear is computed as follows:

$$T_{FN} = (210 \text{ kN})\left(\frac{1}{1 - 1.0\frac{(1.29 \times 10^5 \text{ mm}^2 - 3.14 \times 10^4 \text{ mm}^2)}{[(1500 \text{ mm})(1500 \text{ mm}) - 3.14 \times 10^4 \text{ mm}^2]}}\right) = 219 \text{ kN}$$

(c) Strength Criteria 3: Headed-Stud Tension (section 4.5.6, equation 4.4)

The nominal nail head strength associated with the criteria of headed-stud tension is computed by equation 4.4. For 22.0-mm-diameter studs and a specified headed-stud ultimate tensile stress of 420 MPa, the nominal nail head strength is computed to be:

$$T_{FN} = 4(0.25)(\pi)(22.0 \text{ mm})^2(420 \text{ MPa}) = 639 \text{ kN}$$

F.1.3 Example Problem Nail Head Strength Summary

The nominal nail head strengths computed for the temporary shotcrete construction facing and the permanent CIP concrete facing are summarized below:

Strength Criteria	Temporary Facing Nominal Nail Head Strength T_{FN} (kN)	Permanent Facing Nominal Nail Head Strength T_{FN} (kN)
Facing Flexure	135	278
Facing Punching Shear	204	219
Headed-Stud Tension	Not Applicable	639

F.2. TABULATED NOMINAL NAIL HEAD STRENGTHS

Table F.4 summarizes nominal nail head strengths for facing flexure and facing punching shear failure modes, for common temporary and permanent facing designs. The allowable nail head load (SLD) or design nail head strength (LRFD) is obtained by adjusting these nominal values by the appropriate nail head strength factor or resistance factor, respectively, as described in chapter 4 of this manual.

For the temporary shotcrete construction facing, the flexural failure mode considers a standard 100 mm thick facing of shotcrete compressive strength equal to 28 MPa, with two No. 13 continuous waler bars at each row of nails, and various nail head spacings and sizes of steel mesh reinforcement with and without vertical bearing bars at each nail head location. For the punching shear failure mode of a bearing plate through the shotcrete construction facing, both the internal and total nominal nail head strengths are given for different sizes of bearing plate (nail spacing and drill hole diameter are fixed at typical values as the results are relatively insensitive to these parameters).

For the permanent CIP or shotcrete facing, the flexural failure mode considers a standard fixed pattern of facing reinforcement (No. 13 bars at 300 mm spacing each way) and two facing thicknesses of 200 mm and 150 mm that represent the practical minimum facing thickness that can be constructed for CIP facings and permanent shotcrete facings respectively. For the punching shear failure mode of a headed stud wedge pulling out of the permanent facing, both the internal and total nominal nail head strengths are given for different depths of stud embedment (determined by the bearing plate thickness and overall stud length) and different representative stud spacings.

TABLE F.1

COMMON STYLES OF METRIC WELDED WIRE REINFORCEMENT WITH EQUIVALENT US CUSTOMARY UNITS[1]

Metric Styles (MW = Plain Wire)[2]	A (mm^2/m)	Wt. (kg/m^2)	Equivalent US Customary Styles (W= Plain Wire)[3]	A (in^2/ft)	Wt. (lbs/ft^2)
102x102 - MW9xMW9	88.9	1.51	4x4 - W1.4xW1.4	0.042	3.1
102x102 - MW13xMW13	127.0	2.15	4x4 - W2.0xW2.0	0.060	4.4
102x102 - MW19xMW19	184.2	3.03	4x4 - W2.9xW2.9	0.087	6.2
102x102 - MW26xMW26	254.0	4.30	4x4 - W4.0xW4.0	0.120	8.8
152x152 - MW9xMW9	59.3	1.03	6x6 - W1.4xW1.4	0.028	2.1
152x152 - MW13xMW13	84.7	1.46	6x6 - W2.0xW2.0	0.040	3.0
152x152 - MW19xMW19	122.8	2.05	6x6 - W2.9xW2.9	0.058	4.2
152x152 - MW26xMW26	169.4	2.83	6x6 - W4.0xW4.0	0.080	5.8

[1]Wire sizes may also be deformed, use prefix MD or D, except where only MW or W is required by building codes (usually less than a MW26 or W4). Also wire sizes can be specified in 1 mm^2 (metric) or 0.001 in^2 (US Customary) increments. Areas are based on lower bound tolerances for diameter.

[2]For other available styles or wire sizes, consult other WRI publications or discuss with WWR manufacturers.

[3]Styles may be obtained in roll form. Note: It is recommended that rolls be straightened and cut to size before placement.

Courtesy of Wire Reinforcement Institute (WRI), Findlay Ohio 45839-0450; 419-425-9473

TABLE F.2
BAR SIZES (ENGLISH AND SOFT METRIC*)

Bar Designation No.	Nominal Diameter, in. [mm]	Nominal Area, in^2 [mm^2]
3 [10]	0.375 [9.6]	0.11 [71]
4 [13]	0.500 [12.7]	0.20 [129]
5 [16]	0.625 [15.9]	0.31 [199]
6 [19]	0.750 [19.1]	0.44 [284]
7 [22]	0.875 [22.2]	0.60 [387]
8 [25]	1.000 [25.4]	0.79 [510]
9 [29]	1.128 [28.7]	1.00 [645]
10 [32]	1.270 [32.3]	1.27 [819]
11 [36]	1.410 [35.8]	1.56 [1006]
14 [43]	1.693 [43.0]	2.25 [1452]
18 [57]	2.257 [57.3]	4.00 [2581]

* Soft metric bar designation numbers, nominal diameters and areas are the values enclosed within brackets. Bar designation numbers approximate the number of millimeters of the nominal diameter of the bar.

TABLE F.3
DIMENSIONS OF STOCK SIZE HEADED STUDS

(1.) Anchor Size -in	(2.) A.W. Length -in (mm)	Head Diameter - in (mm)	Head Thickness - in (mm)
$^1/_4$ x $2^{11}/_{16}$	$2^9/_{16}$ (65)	.500 (12.7)	.187 (4.7)
$^1/_4$ x $4^1/_8$	4 (102)	.500 (12.7)	.187 (4.7)
$^3/_8$ x $4^1/_8$	4 (102)	.750 (19.1)	.281 (7.1)
$^3/_8$ x $6^1/_8$	6 (152)	.750 (19.1)	.281 (7.1)
$^1/_2$ x $2^1/_8$	2 (51)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $3^1/_8$	3 (76)	1.00 (25.4)	.312(7.9)
$^1/_2$ x $4^1/_8$	4 (102)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $5^5/_{16}$	$5^3/_{16}$ (132)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $6^1/_8$	6 (152)	1.00 (25.4)	.312 (7.9)
$^1/_2$ x $8^1/_8$	8 (203)	1.00 (25.4)	.312 (7.9)
$^5/_8$ x $2^{11}/_{16}$	$2^1/_2$ (64)	1.250 (31.8)	.312 (7.9)
$^5/_8$ x $6^9/_{16}$	$6^3/_8$ (162)	1.250 (31.8)	.312 (7.9)
$^5/_8$ x $8^3/_{16}$	8 (203)	1.250 (31.8)	.312 (7.9)
$^3/_4$ x $3^3/_{16}$	3 (76)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $3^{11}/_{16}$	$3^1/_2$ (89)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $4^3/_{16}$	4 (102)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $5^3/_{16}$	5 (127)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $6^3/_{16}$	6 (152)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $7^3/_{16}$	7 (178)	1.250 (31.8)	.375 (9.5)
$^3/_4$ x $8^3/_{16}$	8 (203)	1.250 (31.8)	.375 (9.5)
$^7/_8$ x $3^{11}/_{16}$	$3^1/_2$ (89)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $4^3/_{16}$	4 (102)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $5^3/_{16}$	5 (127)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $6^3/_{16}$	6 (152)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $7^3/_{16}$	7 (178)	1.375 (34.9)	.375 (9.5)
$^7/_8$ x $8^3/_{16}$	8 (203)	1.375 (34.9)	.375 (9.5)

NOTES: (1.) Stock Anchor Sizes
(2.) A.W. - Length overall after welding to plate

TABLE F.4
NOMINAL NAIL HEAD STRENGTH

Temporary Shotcrete Construction Facing

Facing Flexure:

Facing Thickness: 100 mm
Steel Yield: 420 MPa
Shotcrete Comp. Strength: 28 MPa
Walers: 2 X No. 13

Nail Spacing (m)	WW Mesh	Vert. Bearing Bars	T_{FN} (kN)
1.25 X 1.25	152X152 MW13XMW13	- 2 X No. 13	58 122
	152X152 MW19XMW19	- 2 X No. 13	81 145
	152X152 MW26XMW26	- 2 X No. 13	111 166*
	102X102 MW9XMW9	- 2 X No. 13	59 124
	102X102 MW13XMW13	- 2 X No. 13	86 149
	102X102 MW19XMW19	- 2 X No. 13	119 170*
1.5 X 1.5	152X152 MW13XMW13	- 2 X No. 13	58 112
	152X152 MW19XMW19	- 2 X No. 13	81 135
	152X152 MW26XMW26	- 2 X No. 13	111 163
	102X102 MW9XMW9	- 2 X No. 13	59 113
	102X102 MW13XMW13	- 2 X No. 13	86 139
	102X102 MW19XMW19	- 2 X No. 13	119 170*
1.75 X 1.75	152X152 MW13XMW13	- 2 X No. 13	58 105
	152X152 MW19XMW19	- 2 X No. 13	81 127
	152X152 MW26XMW26	- 2 X No. 13	111 156
	102X102 MW9XMW9	- 2 X No. 13	59 106
	102X102 MW13XMW13	- 2 X No. 13	86 132
	102X102 MW19XMW19	- 2 X No. 13	119 164

* Calculated capacity limited by maximum reinforcement ratio (based on gross area) of 0.35%.

TABLE F.4 (Cont'd)
NOMINAL NAIL HEAD STRENGTH

Facing Punching Shear:

Facing Thickness:	100 mm
Shotcrete Comp. Strength:	28 MPa
Drill Hole Diameter:	200 mm
Nail Spacing:	1.5 m X 1.5 m

Bearing Plate Width (mm)	**V_N(kN)**	**T_{FN} (kN)**
200	165	184
225	178	204
250	192	224

Permanent Facing

Facing Flexure:

Steel Yield:	420 MPa
Shotcrete Comp. Strength:	28 MPa
Reinforcement:	No. 13 bars @ 300 mm
Nail Pattern:	Vertical Spacing = Horizontal Spacing

Facing Thickness (mm)	**T_{FN} (kN)**
150	206
200	278

Facing Punching Shear:

Shotcrete Comp. Strength:	28 MPa
Drill Hole Diameter:	200 mm
Nail Spacing:	1.5 m X 1.5 m

Pl. Thick.+Stud Lgth. (h_C) (mm)	**Stud Spacing (S_{HS}) (mm)**	**V_N (kN)**	**T_{FN} (kN)**
150	80	189	197
	105	210	219
	130	230	243
145	80	179	185
	105	199	207
	130	219	230
125	80	141	144
	105	158	163
	130	175	182
120	80	132	135
	105	148	152
	130	165	170
100	80	99	100
	105	112	115
	130	126	129
95	80	91	92
	105	104	106
	130	117	120

F.2.1 References

1. MacGregor, J.G. (1988), "Reinforced Concrete: Mechanics and Design," Prentice-Hall: Englewood Cliffs, New Jersey, 799 pages.

2. ACI Committee 349 (1990), "Code Requirements for Nuclear Safety Related Concrete Structures," American Concrete Institute: Detroit, MI.

APPENDIX G

DESIGN EXAMPLE 5.2

AND

SIMPLIFIED DESIGN CHARTS FOR

CUT SLOPE WALLS

5.2 Design Example 2 - Bridge Abutment Wall

5.2.1 Service Load Design (SLD)

5.2.1.1 Static Loading Condition

In accordance with the Standard Specifications for Highway Bridges, 15th Edition [30], Service Load Groups I and IV (table 4.2) define the static loading conditions for this problem.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

It is required to replace a bridge abutment end slope with a vertical soil nail wall in order to add an additional traffic lane and shoulder. The bridge abutment is pile-supported by two rows of 400 mm diameter 15 m long friction piles, at spacings of 1.5 meters along the length of the footing (see figure 5.13). The vertical wall will be located 2 meters in front of the pile cap, resulting in a wall height above road grade of 4.65 meters. Removal of the 1.5H:1V end slope will therefore provide an additional 7 meters of roadway width and this is sufficient for the additional traffic lane and shoulder. The seismic design condition corresponds to a peak ground acceleration of 0.40 g.

The native soils at the site consist of a silty clayey sand that exploration test pitting demonstrated will stand in 2.5 meter high unsupported cuts for a minimum of several days. The site investigation has confirmed that the ground water table is located well below the base of the wall. The approach fill material behind the abutment backwall is a select granular material. Average in-situ densities are 20.0 kN/m^3, and the soil strength parameters are estimated at a friction angle of 34.0 degrees and a cohesion of 5.0 kN/m^2 for the native soils, and a friction angle of 34 degrees for the overlying approach fill. It is estimated that an ultimate pullout resistance on the order of 60.0 kN/m should be achievable with drillhole diameters on the order of 200 mm, within the native soils. This estimated pullout resistance will have to be demonstrated for the contractor's proposed installation method during the initial stages of the wall construction.

Because the soil nail wall will comprise part of a bridge abutment and is a critical structure, encapsulated nails will be used for corrosion protection. Based on experience in these types of materials, anticipated wall displacements associated with wall construction are in the range of 10-15 mm (see step 11), and these displacements have been determined to be acceptable.

The wall will have a maximum height of 5.25 meters (from ground surface immediately behind the wall to the base of the wall), with a vertical face. The critical design cross section is shown on figure 5.13 and will have the center of the pile cap located 2.9 meters behind the facing and the base of the pile cap at a depth of 3.7 meters below the ground surface. The trial vertical nail spacing will be at 1.25 meters as shown on figure 5.13, with a horizontal nail spacing of 1.5 meters to conform to the pile spacing, and the nails will be installed at 15.0 degrees below horizontal for constructability reasons.

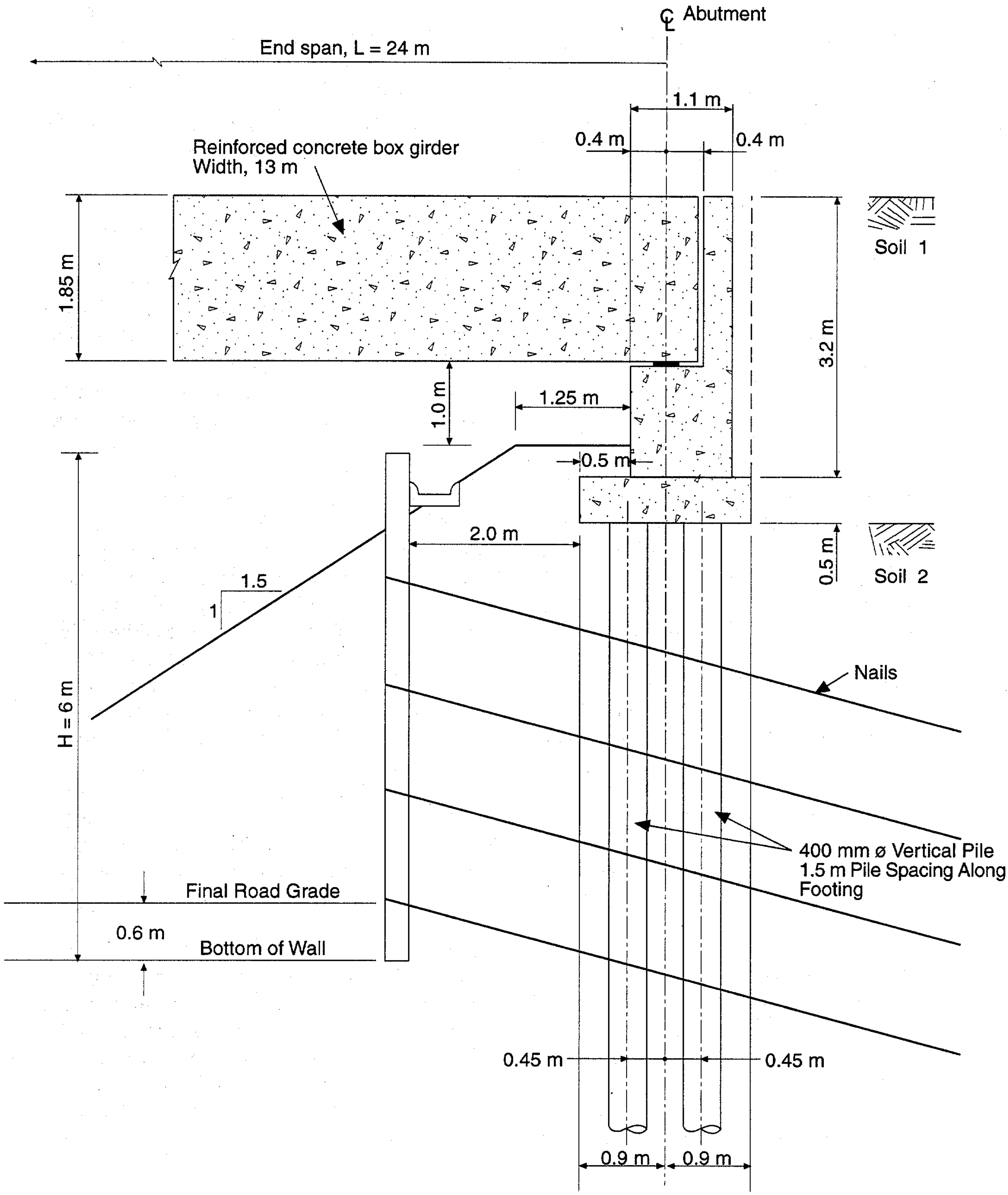


Figure 5.13 Bridge Abutment Wall

It is anticipated that No. 25 or No. 29 minimum Grade 420 bars (Soft Metric designation - see table F.2), approximately 7 to 8 meters long, will be required for support of the bridge abutment wall. Based on local practice and material availability, the trial design will assume use of a "standard" temporary shotcrete construction facing (28-day compressive strength of 28 Mpa) will have a nominal thickness of 100 mm, and will be reinforced with a single layer of 152 x 152 MW18.7 x MW18.7 (6 x 6-W2.9 x W2.9) welded-wire mesh, and two No. 13 Grade 420 continuous horizontal waler bars along each row of nails.

The nails will be connected to the construction facing by a 225 mm square, 25 mm thick bearing plate. The nails will be installed in vertical columns on 1.5 meter centers, to conform to the layout of the piles. The permanent facing will be a cast-in-place (CIP) concrete wall (28-day compressive strength of 28 MPa), 200 mm thick, reinforced with No. 13 Grade 420 deformed bars on 300 mm centers vertically and horizontally, and connected to the nail heads with a headed-stud connection system. Seismic loading will be evaluated only for the permanent facing.

Step 2 - Compute the Allowable Nail Head Loads

The nominal nail head strengths for all credible failure mechanisms are calculated using equations 4.1 through 4.4 and the methodology demonstrated in appendix F. The nominal strengths are shown below and differ from the values calculated for the cutslope design example because of a) the absence of vertical bearing bars in the shotcrete construction facing and b) the reduced vertical nail spacing for the bridge abutment wall versus the cutslope wall. The allowable nail head loads for Service Load Groups I and IV are computed from the nominal strengths as indicated in the tables below for both the temporary shotcrete construction facing and the permanent CIP concrete facing.

SHOTCRETE CONSTRUCTION FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load (Group I) T_F (kN)	Allowable Nail Head Load (Group IV) T_F (kN)
Facing Flexure	97	0.67[a](97) = 65	0.83[a](97) = 81
Facing Punching Shear	210	0.67[a](210) = 141	0.83[a](210) = 174

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load (Group I) T_F	Allowable Nail Head Load (Group IV) T_F (kN)
Facing Flexure	232	$0.67^a(232) = 155$	$0.83^a(232) = 193$
Facing Punching Shear	222	$0.67^a(222) = 149$	$0.83^a(222) = 184$
Headed-Stud Tensile Fracture	639	$0.50^a(639) = 320$	$0.63^a(639) = 403$

[a] See Table 4.4.

Therefore, the allowable nail head load T_F is computed to be 65 kN for Service Load Group I and 81 kN for Service Load Group IV. Facing flexure of the temporary facing is the controlling mode of failure.

Step 3 - Minimum Allowable Nail Head Service Load Check

Since the design problem has a somewhat complex surcharge loading distribution, the estimated nail head service load will be calculated from equation 4.7, with the active earth pressure load P_A determined from a Coulomb-type computer solution i.e., identify the critical active slip surface that results in the highest calculated earth pressure loading. Figure 5.14 shows the design loadings on the pile-supported bridge abutment, with the Group I pile loadings corresponding to 301 kN/m vertically and 51.3 kN/m laterally. Because of the twin pile configuration, the pile head is assumed to have rotational fixity. The pile loadings are transmitted to the soil through pile-soil interaction. Vertical (theoretical t-z curves taken from Vijayvergiya [41] and using the method of Coyle and Reese [42]) and lateral (computer program LPILE [43]) soil-pile interaction analyses established that the proportions of the vertical and horizontal pile head loads transmitted to the nailed soil are a function of the depth at which potential slip surfaces intersect the piles. These proportions are shown plotted on figure 5.14. The pile loads were simulated by applying equivalent vertical and horizontal surcharges at the base of the pile cap, as shown on figure 5.15 (calculated on figure 5.14). The equivalent vertical and horizontal surcharge loadings to be applied at the base of the pile cap are given on figure 5.14 in terms of the fractions of the applied pile loads (f_V vertical and f_H horizontal) transmitted to the soil. The pile load fractions are a function of the depth at which the critical slip surface intersects the piles (as shown on figure 5.14), and this depth must be estimated. For evaluation of the active earth pressure load (figure 5.15), the depth at which the critical slip surface intersects the piles is on the order of 2 meters, giving an f_V value of 0.07 (i.e., a vertical surcharge pressure of 11.7 kN/m^2) and a k_H value of 0.8 (i.e., a horizontal surcharge pressure of 22.8 kN/m^2). For simplicity, the retained earth

Service Load Design

Pile Loading	'Equivalent' Surcharge Pressure Beneath Pile Cap (Zone 1)	Surcharge Pressure from Fill and Live Loading (Zone 2)
Group I		
Vertical $= R_D + R_L + V_L + W$ $= 301$ kN/m	$q_V = (301)\,(f_V^*)/1.8$ $= 167.2\,(f_V)$ kN/m^2	$W_V = W_F + W_L$ $= 86$ kN/m^2
Horizontal $= H_E + H_L$ $= 51.3$ kN/m	$q_H = (51.3)\,(f_H^*)/1.8$ $= 28.5\,(f_H)$ kN/m^2	$W_H = 0$ kN/m^2
Group IV		
Vertical $= R_D + R_L + V_L + W$ $= 301$ kN/m	$q_V = (301)\,(f_V^*)/1.8$ $= 167.2\,(f_V)$ kN/m^2	$W_V = W_F + W_L$ $= 86$ kN/m^2
Horizontal $= H_E + H_L + H_{TS}$ $= 73.3$ kN/m	$q_H = (73.3)\,(f_H^*)/1.8$ $= 40.7\,(f_H)$ kN/m^2	$W_H = 0$ kN/m^2
Group VII		
Vertical $= R_D + W$ $= 226$ kN/m	$q_V = (226)\,(f_V^*)/1.8$ $= 125.6\,(f_V)$ kN/m^2	$W_V = W_F$ $= 74$ kN/m^2
Horizontal $= H_E + H_E^{EQ} + H_R^{EQ} + H_W^{EQ}$ $= 149.9$ kN/m (0.42g) $= 125.8$ kN/m (0.32g)	$q_H = (149.9)\,(f_H^*)/1.8$ $= 83.3\,(f_H)$ kN/m^2 (0.42g) $q_H = (125.8)\,(f_H^*)/1.8$ $= 69.9\,(f_H)$ kN/m^2 (0.32g)	$W_H = W_F^{EQ}$ $= 31.1$ kN/m^2 (0.42g) $= 23.7$ kN/m^2 (0.32g)

* Pile load applied as uniform surcharge over base of 1.8 m wide pile cap.
Vertical Surcharge = Fraction "f_V" of Pile Vertical Loading
Horizontal Surcharge = Fraction "f_H" of Pile Lateral Loading

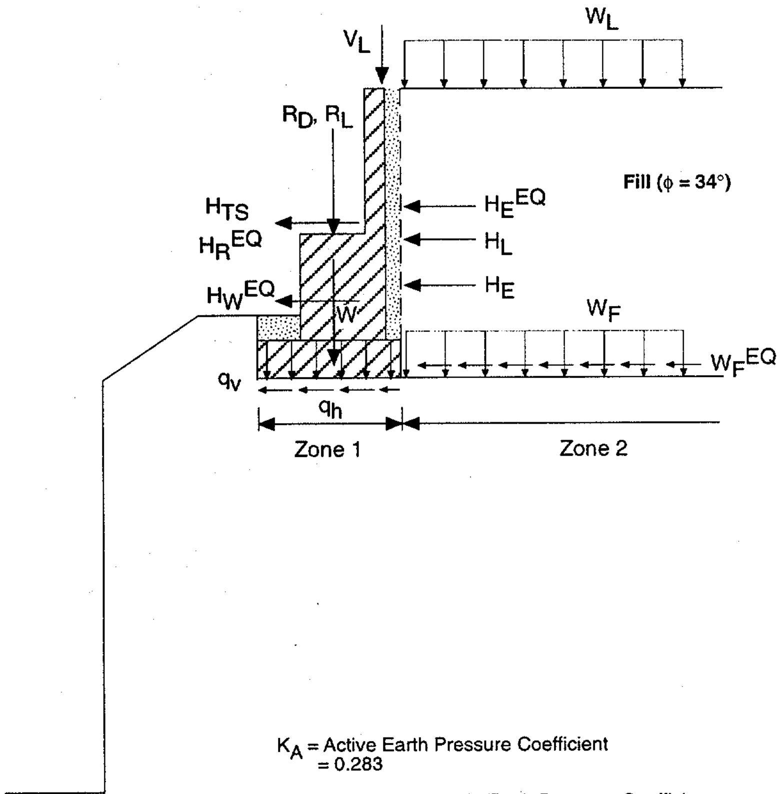
Load and Resistance Factor Design

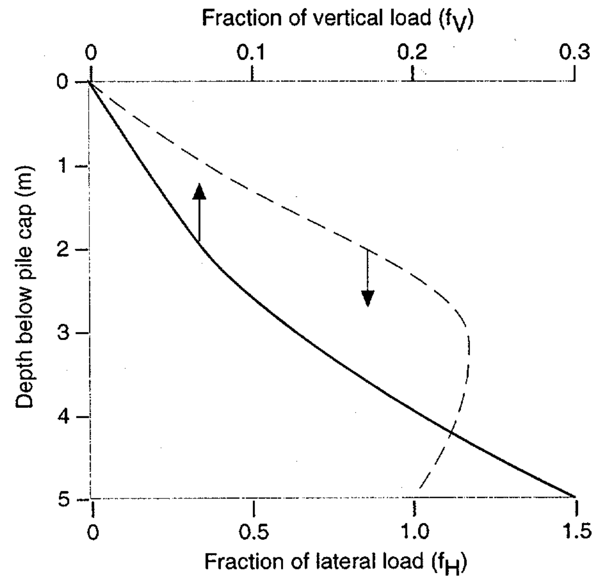
Pile Loading (Factored)	'Equivalent' Surcharge Pressure Beneath Pile Cap (Zone 1)	Surcharge Pressure from Fill and Live Loading (Zone 2)
Strength I		
Vertical $= R_D\,(1.25) + R_L\,(1.75) + V_L\,(1.75) + W\,(1.25)$ $= 413.8$ kN/m	$q_V = (413.8)\,(f_V^*)/1.8$ $= 229.9\,(f_V)$ kN/m^2	$W_V = W_F\,(1.35) + W_L\,(1.75)$ $= 120.9$ kN/m^2
Horizontal $= H_E\,(1.5) + H_L\,(1.75) + H_{TS}\,(0.5)$ $= 91.1$ kN/m	$q_H = (91.1)\,(f_H^*)/1.8$ $= 50.6\,(f_H)$ kN/m^2	$W_H = 0$ kN/m^2
Strength IV		
Vertical $= R_D\,(1.5) + W\,(1.5)$ $= 339$ kN/m	$q_V = (339)\,(f_V^*)/1.8$ $= 188.3\,(f_V)$ kN/m^2	$W_V = W_F\,(1.35)$ $= 99.9$ kN/m^2
Horizontal $= H_E\,(1.5) + H_{TS}\,(0.5)$ $= 69.1$ kN/m	$q_H = (69.1)\,(f_H^*)/1.8$ $= 38.4\,(f_H)$ kN/m^2	$W_H = 0$ kN/m^2
Extreme Event I		
Vertical (Max‡) $= R_D\,(1.25) + W\,(1.25)$ $= 282.5$ kN/m	q_V (max) $= (282.5)\,(f_V^*)/1.8$ $= 156.9\,(f_V)$ kN/m^2	W_V (max) $= W_F\,(1.35)$ $= 99.9$ kN/m^2
Vertical (Min‡) $= R_D\,(0.9) + W\,(0.9)$ $= 203.4$ kN/m	q_V (min) $= (203.4)\,(f_H^*)/1.8$ $= 113.0\,(f_H)$ kN/m^2	W_V (min) $= W_F\,(1.0)$ $= 74$ kN/m^2
Horizontal $= H_E\,(1.5) + H_E^{EQ}\,(1.0) + H_W^{EQ}\,(1.0) + H_R^{EQ}\,(1.0)$ $= 169.3$ kN/m (0.42g) $= 145.2$ kN/m (0.32g)	$q_H = (169.3)\,(f_H^*)/1.8$ $= 94.1\,(f_H)$ kN/m^2 (0.42g) $q_H = (145.2)\,(f_H^*)/1.8$ $= 80.7\,(f_H)$ kN/m^2 (0.32g)	$W_H = W_F^{EQ}\,(1.0)$ $= 31.1$ kN/m^2 (0.42g) $= 23.7$ kN/m^2 (0.32g)

* Pile load applies as uniform surcharge over base of 1.8 m wide pile cap.
Vertical Surcharge = Fraction f_V of Pile Vertical Loading
Horizontal Surcharge = Fraction f_H of Pile Lateral Loading

‡ Refers to maximum and minimum load factors per Table 4.6.

Figure 5.14 Bridge Abutment Wall Design Example Surcharge Load Calculation (continued on next page)





Fraction of Pile Head Load Transmitted to Soil as Function of Depth

K_A = Active Earth Pressure Coefficient = 0.283

K_{AE} (0.32g) = Dynamic Earth Pressure Coefficient = 0.231

K_{AE} (0.42g) = 0.344

R_D = 140 kN/m (Dead Load from Bridge Superstructure)

R_L = 69 kN/m (Live Load from Bridge Superstructure)

W = 86 kN/m (Dead Load from Concrete Abutment and Soil above Pile Cap)

H_{TS} = 22 kN/m (Temperature/Shrinkage Load)

$W_F = 3.7 \times 20 = 74$ kN/m^2 (Fill Dead Load Surcharge)

$W_L = 0.6 \times 20 = 12$ kN/m^2 (Live Load Surcharge = 0.6m soil)

$V_L = 0.5 \times 12 = 6$ kN/m (Live Load Surcharge above Pile Cap)

$H_E = 1/2 \times 0.283 \times 20 \times 3.7^2 = 38.7$ kN/m (Lateral Earth Pressure Load)

$H_L = .283 \times 12 \times 3.7 = 12.6$ kN/m (Lateral Earth Pressure Load from Live Load Surcharge)

* $H_R^{EQ} = 0.2 \times 140 = 28$ kN/m (Lateral Earthquake Loading from Bridge Superstructure)

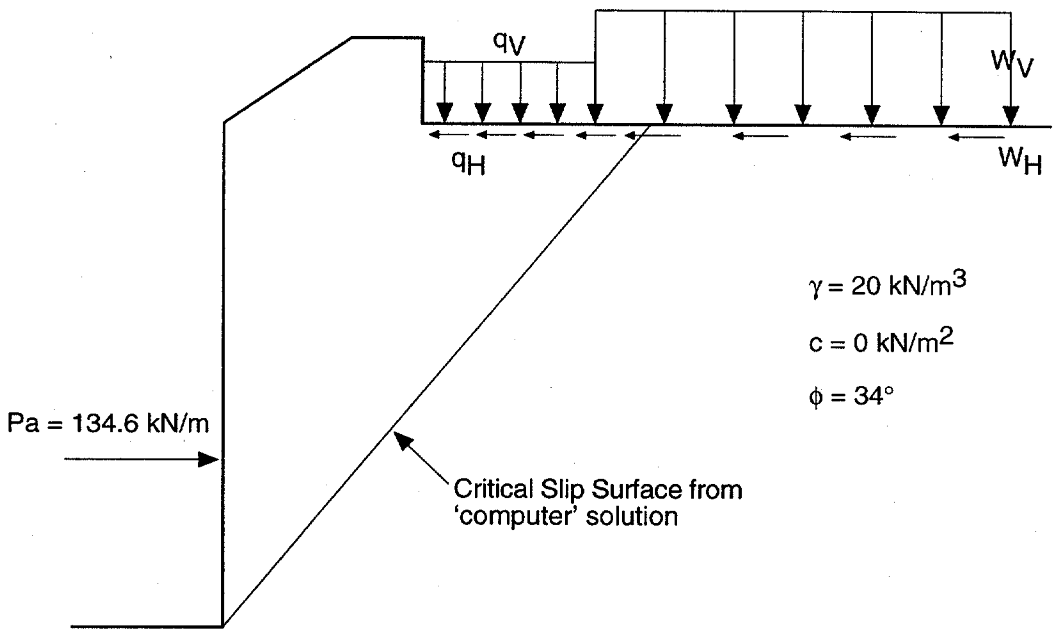
H_W^{EQ} (0.42g) $= 0.42 \times 86 = 36.1$ kN/m
H_W^{EQ} (0.32g) $= 0.32 \times 86 = 27.5$ kN/m
} (Lateral Earthquake Loading from Abutment)

H_E^{EQ} (0.42g) $= 1/2 \times .344 \times 20 \times 3.7^2 = 47.1$ kN/m
H_E^{EQ} (0.32g) $= 1/2 \times .231 \times 20 \times 3.7^2 = 31.6$ kN/m
} (Lateral Earth Pressure Load from Earthquake)

W_F^{EQ} (0.42g) $= 0.42 \times 74 = 31.1$ kN/m^2
W_F^{EQ} (0.32g) $= 0.32 \times 74 = 23.7$ kN/m^2
} (Earthquake Shear Stress from Fill Surcharge)

* Limited to 0.2 times dead load form Bridge Superstructure (by slip of bearing pad)

Figure 5.14(cont.) Bridge Abutment Wall Design Example Surcharge Load Calculation



$q_V = 11.7$ kN/m^2

$q_H = 22.8$ kN/m^2

$W_V = 86.0$ kN/m^2

$W_H = 0$ kN/m^2

Figure 5.15 Cross-Section for Determination of 'Active' Face Loads for Nail Head Load Determination

and live load surcharge behind the bridge abutment are modeled as an equivalent vertical surcharge, as shown on figure 5.15. For a friction angle of 34 degrees, and the Group I surcharge loadings discussed above, the active earth pressure load corresponding to a horizontal, triangular earth pressure distribution is given by P_A equal to 134.6 kN/m (ignoring the cohesive component of soil strength in accordance with the discussion of section 2.4.5). Values for other parameters are as follows:

S_H = 1.50m
S_V = 1.25m
H = 5.25 m
F_F = 0.50

Substituting the above values into equation 4.7, to determine the nail head service load:

$t_F = 2F_F\, P_A\, S_H\, S_V/H$
$t_F = (2.0)(0.50)(134.6\ \text{kN/m})(1.50\ \text{m})(1.25\text{m})/(5.25\text{m})$
$t_F = 48.1\ \text{kN} \quad < \quad 65\ \text{kN}$

(OK - the estimated nail head service load does not exceed the allowable nail head load)

Step 4 - Define the Allowable Nail Load Support Diagrams

Develop the allowable nail load diagram for each nail by determining the allowable pullout resistance, the allowable nail head load, and the allowable nail tendon tensile load.

Allowable Nail Head Load

Per step 2, the allowable nail head load is 65 kN (Load Group I) and 81 kN (Load Group IV).

Allowable Pullout Resistance (Ground-Grout Bond)

$Q = \alpha_Q Q_U$
$Q_U = 60.0\ \text{kN/m}$

Load Group I:
$\alpha_Q = 0.50$ (table 4.5)
$Q = (0.50)(60.0\ \text{kN/m}) = 30.0\ \text{kN/m}$

Load Group IV:
$\alpha_Q = 0.63$ (table 4.5)
$Q = (0.63)(60.0\ \text{kN/m}) = 37.8\ \text{kN/m}$

Allowable Nail Tendon Tensile Load

$T_N = \alpha_N T_{NN}$
$T_{NN} = A_N\, F_Y = (510\ \text{mm}^2)(0.42\ \text{kN/mm}^2) = 214\ \text{kN}$ (for No. 25 nail tendon)

Load Group I:

α_N = 0.55 (table 4.5)

T_N = (0.55)(214 kN) = 118 kN

Load Group IV:

α_N = 0.69 (table 4.5)

T_N = (0.69)(214 kN) = 148 kN

Step 5 - Select Trial Nail Spacing and Lengths

In accordance with section 4.7.1 and figure 4.11, the nail length distribution for static design purposes (Load Group I) is as shown on figure 5.16. The maximum nail length has been calculated iteratively to be 6.6 meters (see step 7, below). The trial nail length distribution is:

Nail No.	Length (m)
1	6.6
2	6.6
3	5.6
4	3.6

The above nail length distribution for Load Group I is obtained from figure 4.11, as follows:

- The dimensionless nail pullout resistance, Q_D, is calculated:

  $Q_D = \alpha_Q Q_U/(\gamma S_V S_H) = (0.50)(60.0 \text{ kN/m})/[(20 \text{ kN/m}^3)(1.25 \text{ m})(1.50 \text{ m})]$
  $= 0.8$

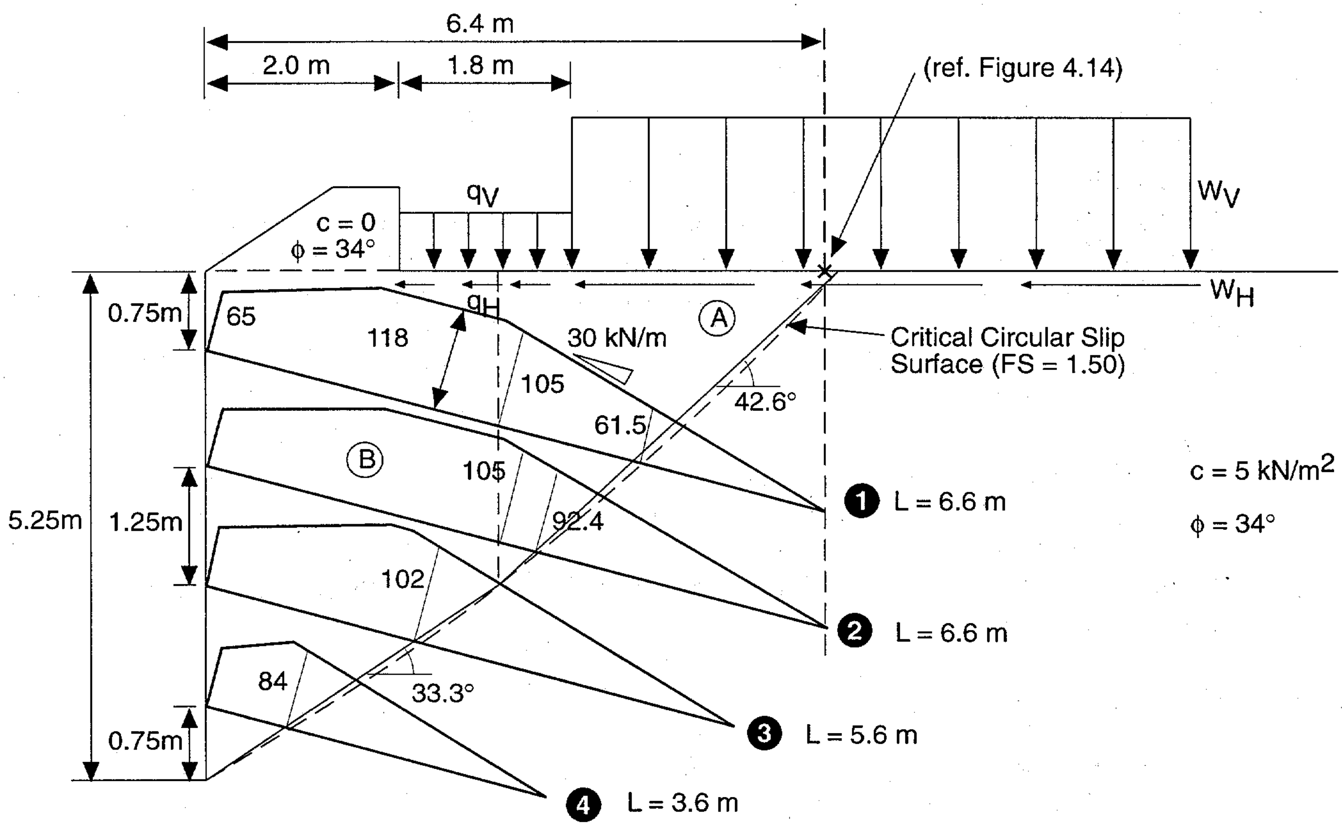
- The dimensionless nail length is:

  $L/H = (6.6 \text{ m})/(5.25 \text{ m}) = 1.26$

- $Q_D / (L/H) = 0.8/1.26 = 0.63$, giving an "R" factor of 0.37.

- Relative nail lengths are calculated from figure 4.11 for the nail head elevations shown on figure 5.13 and an "R" value of 0.37.

The allowable nail load support diagrams are shown graphically on figure 5.16, prepared in accordance with the procedure previously presented on figure 4.3.



Surcharge:

$q_V = 33.4\ kN/m^2$

$q_H = 32.8\ kN/m^2$

$W_V = 86.0\ kN/m^2$

$W_H = 0\ kN/m^2$

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

Figure 5.16 Bridge Abutment Wall Design Example Critical Cross-Section Static Loading - Load Group I (SLD)

Step 6 - Define the Ultimate Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0°
Ultimate Cohesion, c_U = 5.0 kN/m^2

Step 7 - Calculate the Factor of Safety

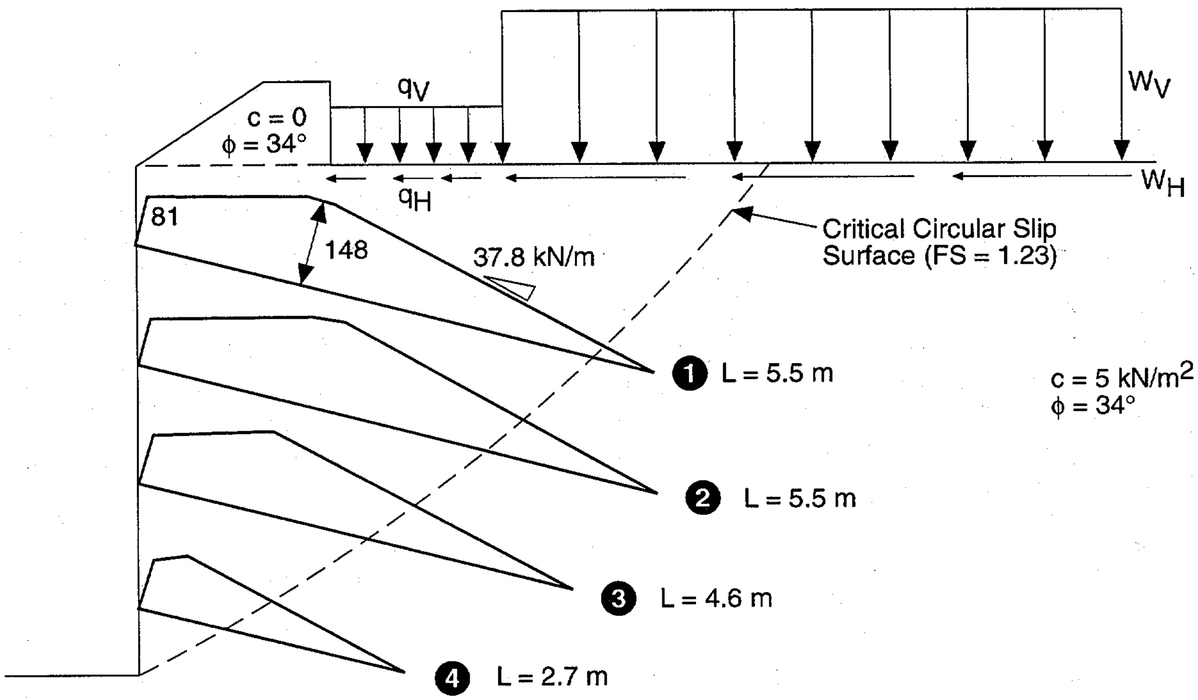
For the trial design cross-section and nail pattern shown on figure 5.16, the above soil strengths and allowable nail load diagrams, a calculated minimum global factor of safety of 1.50 (required factor of safety is 1.50 for Group I loading for this critical structure) is obtained based on a computer solution. **For conciseness of presentation of this design example, the initially chosen trial design is in fact the design that meets the minimum required factor of safety. In a real world production design, some design re-runs and iterations would be required to arrive at an acceptable solution.** For slip surfaces passing through the toe of the wall, the critical slip surface is estimated to intersect the piles at a depth of about 4 meters, giving an f_V value of 0.2 (vertical surcharge of 33.4 kN/m^2) and an f_H value of 1.15 (horizontal or lateral surcharge of 32.8 kN/m^2) - see figures 5.14 and 5.16. The computer solution critical slip surface is shown on figure 5.16. For this application, Group IV loading is less critical than Group I loading. Figure 5.17 shows the allowable nail load diagrams for Group IV loading for a nail pattern that provides a calculated minimum factor of safety of 1.23 (required Group IV factor of safety is 1.20). As can be seen from a comparison of figures 5.16 and 5.17, Load Group I requires longer nail lengths and is more critical in this case.

Hand Calculation Check

The following provides an approximate method for performing a hand calculation check, if required, for the Group I loading problem described on figure 5.16. The method is approximate in that it is based on force balance only and more reliable results will generally be obtained using methods that address both force and moment equilibrium. In order to facilitate the hand calculation check, the critical circular slip surface may be approximated by a bilinear wedge, as shown on figure 5.16. The forces on the 'active' Block A and the 'resisting' Block B are illustrated on figure 5.18 for the bilinear wedge, and are computed below:

Block A

Base Slope Angle, α_A = 42.6°
Base Length, L_A = 4.85 m
Block Weight, W_A = (5.85 m^2)(20.0 kN/m^3) = 117.0 kN/m
Vertical Surcharge, Q_{VA} = (0.8 m)(33.4 kN/m^2) + (2.77 m)(86 kN/m^2)
= 264.9 kN/m
Horizontal Surcharge, Q_{HA} = (0.8 m)(32.8 kN/m^2) = 26.2 kN/m



Surcharge:

$q_V = 33.4\ kN/m^2$

$q_H = 46.8\ kN/m^2$

$W_V = 86.0\ kN/m^2$

$W_H = 0\ kN/m^2$

Notes:

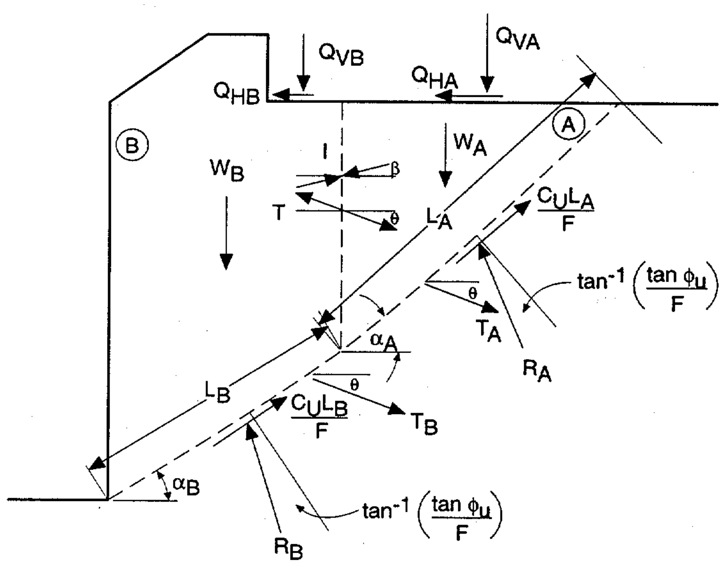
All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

Figure 5.17 Bridge Abutment Wall Design Example Critical Cross-Section Static Loading - Load Group IV (SLD)



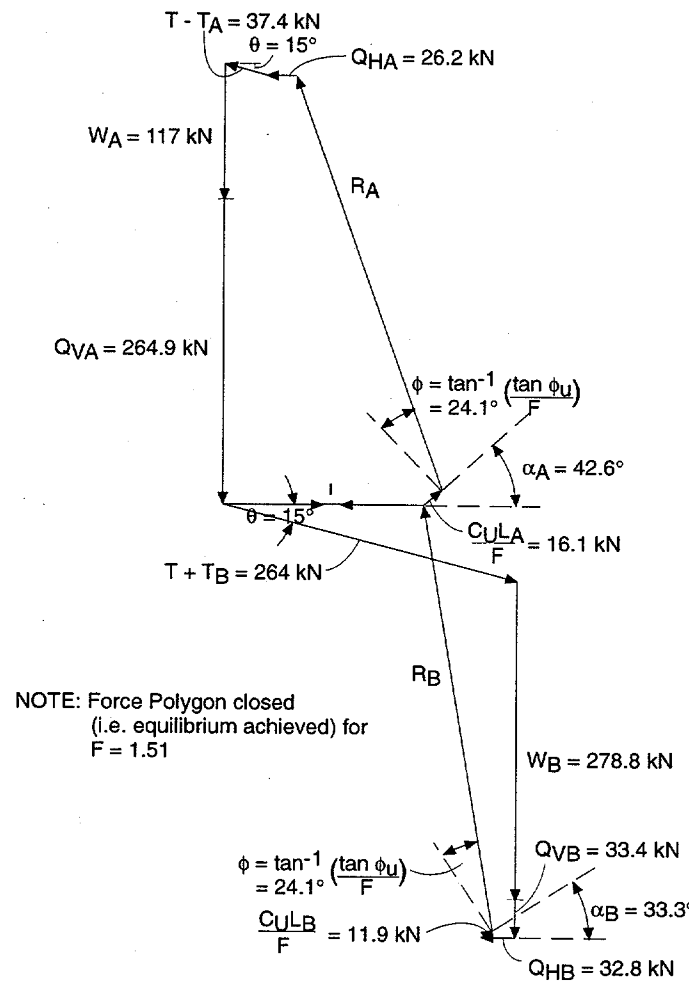


Figure 5.18 Bridge Abutment Design Example Force Diagram and Polygon (SLD) Hand Calculation Check

Nail Force, $T_A = [(61.5 \text{ kN})^{(1)} + (92.4 \text{ kN})^{(2)}]/(1.50 \text{ m})$
$= 102.6$ kN/m

$^{(n)}$ Nail Number

Block B

Base Slope Angle, $\alpha_B = 33.3°$
Base Length, $L_B = 3.59$ m
Block Weight, $W_B = (13.94 \text{ m}^2)(20.0 \text{ kN/m}^3) = 278.8$ kN/m
Vertical Surcharge, $Q_{VB} = (1.0 \text{ m})(33.4 \text{ kN/m}^2) = 33.4$ kN/m
Horizontal Surcharge, $Q_{HB} = (1.0 \text{ m})(32.8 \text{ kN/m}^2) = 32.8$ kN/m
Nail Force, $T_B = [(102.0 \text{ kN})^{(3)} + (84.0 \text{ kN})^{(4)}]/(1.5 \text{ m})$
$= 124.0$ kN/m

Interslice

Nail Force, $T = [(105.0 \text{ kN})^{(1)} + (105.0 \text{ kN})^{(2)}]/(1.5 \text{ m})$
$= 140.0$ kN/m

Assume that the interslice soil force, I, is inclined at an angle 'β' to the horizontal (figure 5.18). The factor of safety F is then applied to the soil strengths, as indicated below.

Equilibrium

Define $\phi = \tan^{-1}[\tan(\phi_U)/F]$.

Block A

Vertical

$$W_A + Q_{VA} + (T_A - T)\sin(\theta) - (I)\sin(\beta) - c_U(L_A)\sin(\alpha_A)/F - (R_A)\cos(\alpha_A - \phi) = 0$$

Horizontal

$$(I)\cos(\beta) + (T_A - T)\cos(\theta) - Q_{HA} + c_U(L_A)\cos(\alpha_A)/F - (R_A)\sin(\alpha_A - \phi) = 0$$

Block B

Vertical

$$W_B + Q_{VB} + (T_B + T)\sin(\theta) + (I)\sin(\beta) - c_U(L_B)\sin(\alpha_B)/F - (R_B)\cos(\alpha_B - \phi) = 0$$

Horizontal

$$(I)\cos(\beta) - (T_B + T)\cos(\theta) + Q_{HB} - c_U(L_B)\cos(\alpha_B)/F + (R_B)\sin(\alpha_B - \phi) = 0$$

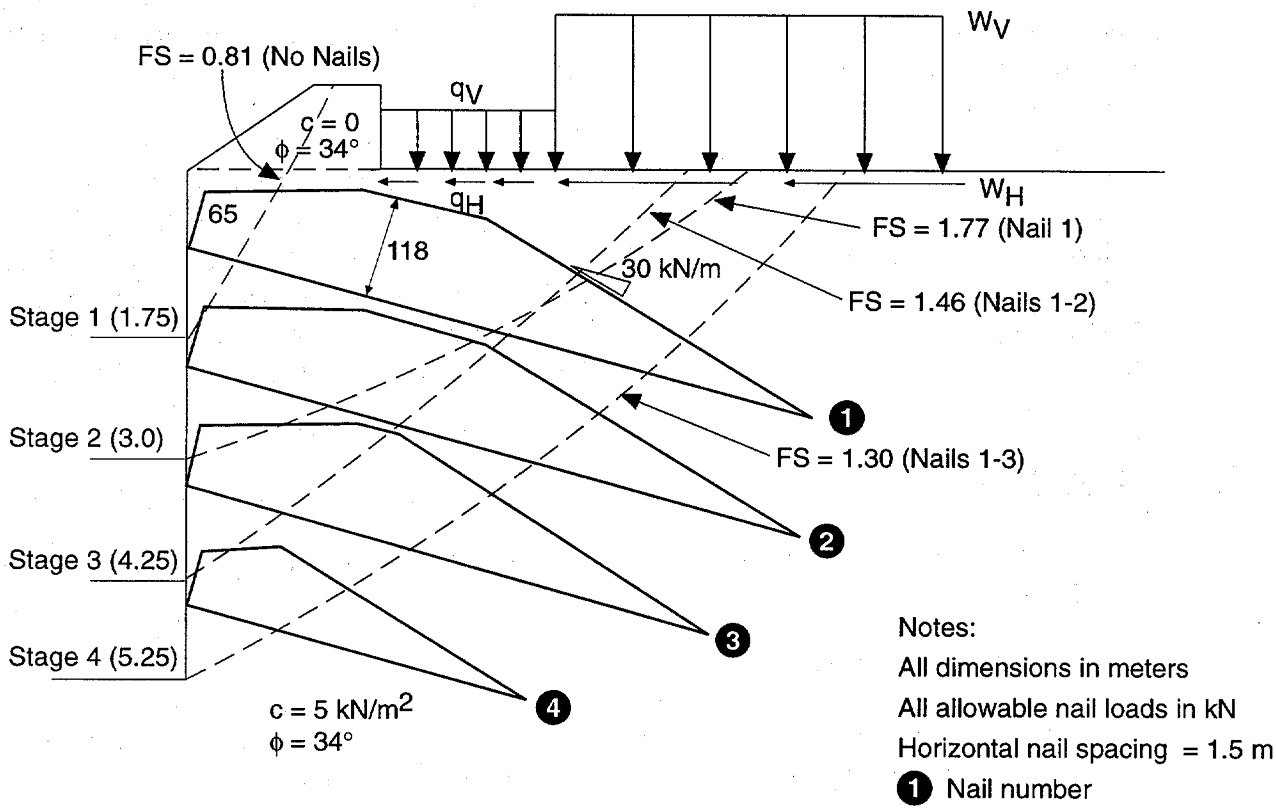
The above equations can be reduced to the following:

$[1+\tan(\beta)\tan(\alpha_A-\phi)]\,[\{-W_B - Q_{VB} - (T_B + T)\sin(\theta)+ c_U(L_B)\sin(\alpha_B)/F\}\tan(\alpha_B-\phi)+ (T_B + T)\cos(\theta) - Q_{HB} + c_U(L_B)\cos(\alpha_B)/F] = [1+\tan(\beta)\tan(\alpha_B-\phi)]\,[\{W_A + Q_{VA} + (T_A - T)\sin(\theta)- c_U(L_A)\sin(\alpha_A)/F\}\tan(\alpha_A-\phi)-(T_A - T)\cos(\theta) + Q_{HA} - c_U(L_A)\cos(\alpha_A)/F]$

The above equation can be solved iteratively for the factor of safety F (e.g., by use of a spreadsheet), if the interslice force angle β is assumed. For example, assuming $\beta = 0$, the calculated factor of safety F = 1.51, which is similar to the computer-generated solution. The force polygon for the hand calculation is shown on figure 5.18.

The results of stability assessments during construction are shown on figure 5.19. Calculated factors of safety for each stage of construction, following excavation of each lift and prior to installation of the associated row of nails, are shown together with the corresponding critical slip surface in each case. As noted on figure 5.19, the "equivalent" surface surcharge representing the load transmitted by the piles to the reinforced soil zone is modified as the depth of the critical slip surface increases, in accordance with the information presented on figure 5.14. The estimated "equivalent" surface surcharges representing the pile loads transmitted to the soil are shown on figure 5.19, for each stage of construction. These estimated "equivalent" surcharges were determined by estimating the average depth at which the critical slip surface intersected the piles in each case, and applying the appropriate fractions of the total pile vertical and lateral loads, as determined from figure 5.14. Figure 5.19 indicates that prior to the installation of the first row of nails, the calculated factor of safety is 0.81, which is well below the required value of 1.35 (table 4.5). However, as noted previously, this initial lift condition has been evaluated in the field by design phase test pitting. If extra precaution is considered desirable, the first row of nails can be installed in stages with a limited length of open unsupported face at any time. In addition, the nails can be installed through a protective stabilization berm (see Example Plans, appendix A).

It can also be seen that for this problem, while the stage 2 and stage 3 excavation conditions satisfy the minimum construction phase factor of safety requirements, the stage 4 condition (i.e., following the final cut but before installation of the final row of nails) has a calculated factor of safety of 1.30 which is less than the required value of 1.35. In general, this situation could be addressed by a) limiting the length of open excavation in the final cut before nail installation or b) increasing the nail lengths slightly (about 5 percent in this case) to achieve the required factor of safety. In this case, however, as will be seen in the following discussion, seismic loading conditions require that longer nails of greater capacity be used and the final nail pattern will therefore satisfy the required construction phase factor of safety values.



Stage	q_V (kN/m^2)	q_H (kN/m^2)	W_V (kN/m^2)	W_H (kN/m^2)
1	–	–	86	0
2	8.4 (f_V = .05)*	17.1(f_H = 0.60)	86	0
3	13.4(f_V = .08)	27.1(f_H = 0.95)	86	0
4	21.7(f_V = .13)	32.8(f_H = 1.15)	86	0

* See Figure 5.14

Figure 5.19 Bridge Abutment Wall Design Example Construction Stability - Load Group I (SLD)

Step 8 - External Stability Check of Nailed Block

Defer the external stability check of the nailed block until the final soil nail lengths are defined for the seismic loading condition.

Step 9 - Check the Facing Upper Cantilever

The height of the upper cantilever from the top row of nails to the top of retained soil is identical (0.75 meters) for both the temporary shotcrete and permanent CIP wall facings. Therefore, the static loading is identical in the two cases. Because both the facing thickness and steel content are increased in the permanent facing, the permanent facing is less critical by inspection. Therefore, for the static loading condition, only the construction facing upper cantilever needs to be evaluated.

For the method of construction, the appropriate earth pressure coefficient for the upper cantilever design is an active earth pressure coefficient. For a soil friction angle of 34°, a soil-to-wall interface friction angle of (2/3)(34°) = 22°, a vertical wall, and a soil profile behind the wall as shown on figure 5.13, the active earth pressure load P_A determined from a Coulomb-type computer solution is 2.95 kN/m length of wall. The load component normal to the wall has a value of (2.95)cos(22°) =2.74 kN/m, applied at the lower third point of the cantilever.

Shear Check

As noted above, the one-way unit service shear force for the facing at the level of the upper row of nails (conservative), is:

$$v = 2.74 \text{ kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 8-49 of the Standard Specifications for Highway Bridges, 15th Edition [30]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28 \text{ MPa}}\,(0.05 \text{ m}) \quad = 43.9 \text{ kN/m}$$

From table 4.4, the facing shear strength factor, α_F, equals 0.67. Therefore, the allowable one-way unit shear is computed to be:

$$V = \alpha_F V_{NS} = (0.67)(43.9 \text{ kN/m}) = 29.4 \text{ kN/m}$$

Since v < V, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the one-way unit service moment for the facing at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever:

$$m_S = (0.33)(H)(v)$$
$$m_S = (0.33)(0.75\text{ m})(2.74\text{ kN/m}) = 0.678\text{ kN-m/m}$$

Compute the nominal unit moment resistance of the facing. Using the procedure demonstrated in appendix F, the nominal unit moment resistance in the vertical direction over the nail locations, $m_{V,NEG}$, is computed to be 2.53 kN-m/m. From table 4.4, the strength factor, α_F, for facing flexure is 0.67. Therefore, the allowable one-way unit moment for the upper cantilever is:

$$M = \alpha_F\, m_{V,NEG} = (0.67)(2.53\text{ kN-m/m}) = 1.70\text{ kN-m/m}$$

Since $m_S < M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

Shotcrete Construction Facing

Check 1 - Waler Reinforcement

Two No. 13 horizontal waler bars, attached beneath the bearing plate, will be placed continuously between nail heads in each nail row.

Check 2 - Minimum Reinforcement Ratios

The minimum reinforcement ratio requirements in the Standard Specifications for Highway Bridges, 15th Edition [30] are waived for the temporary shotcrete construction facing per the discussion presented in section 4.7.1.

Check 3 - Minimum Cover Requirements

For the temporary shotcrete construction facing, the reinforcement is placed at the center of the facing (figure F.1).

Check 4 - Development and Splices of Reinforcement

Splicing of No. 13 Waler Bars

Per section 8.32 of AASHTO [30], the splice length, L_S, for a Class C splice must equal or exceed the greater of 0.30 m or $L_D = 1.7L_{DB}$, where L_{DB} is computed from section 8.25:

$$L_{DB} = 0.019(A_B\,(\text{mm}^2))(F_Y\,(\text{MPa}))/\sqrt{f'_C\,(\text{MPa})}$$
$$L_{DB} = 0.019(129\text{ mm}^2)(420\text{ MPa})/\sqrt{28.0\text{ MPa}} = 195\text{ mm}$$

$L_D = 1.7L_{DB} = \quad 1.7(195\text{ mm}) \quad = \quad 332\text{ mm}$

Therefore, provide 350 mm of splice length.

Splicing of MW 18.7 Mesh

Per section 8.30 of AASHTO [30], the splice length between outermost crosswires must equal or exceed the greater of (S_{WIRE} + 50.0 mm), $1.5L_{DB}$, or 150 mm. L_{DB} is computed from AASHTO equation 8-68:

$$L_{DB} = 3.25(A_{WIRE}\ (\text{mm}^2))(F_Y\ (\text{MPa}))/(((S_{WIRE}\ (\text{mm}))\sqrt{f'_C\ (\text{MPa})}\ ))$$
$$L_{DB} = 3.25(18.7\text{ mm}^2)(420\text{ MPa})/((152\text{ mm})\sqrt{28.0\text{ MPa}}\ )$$
$$L_{DB} = 31.7\text{ mm}$$
$$L_D = 1.5L_{DB} = 1.5(31.7\text{ mm}) = 47.6\text{ mm}$$
$$S_{WIRE} + 50.0\text{ mm} = 152\text{ mm} + 50.0\text{ mm} = 202\text{ mm}$$

Therefore, use a minimum of 200-mm splices for the wire mesh.

Permanent Facing

Check 1 - Waler Reinforcement

There are no applicable requirements.

Check 2 - Minimum Reinforcement Ratios

Per section 8.20 of AASHTO [30], the minimum required amount of shrinkage and temperature reinforcement near exposed surfaces of walls and slabs is 265 mm^2 per lineal meter. The No. 13 bars on 300 mm centers provides (129 mm^2)/(0.3 m) = 430 mm^2 per meter, which is adequate.

Check 3 - Minimum Cover Requirements

Per section 8.22 of AASHTO [30], the minimum cover on the front side of the facing is specified at 50 mm. Based on the design illustrated on figure F.1, the cover to the headed studs is as indicated below:

$$t_C = 200\text{ mm} - t_{PL} - L_{HS} = 200\text{ mm} - 25\text{ mm} - 125\text{ mm} = 50\text{ mm}$$
$$50\text{ mm} \geq 50\text{ mm} \quad \text{(OK)}$$

The minimum required cover between the permanent facing reinforcing steel and the CIP concrete/temporary shotcrete interface is 38 mm. Based on figure F.1, this cover is:

$$t_c = 100\text{ mm} - 12.7\text{ mm} = 87.3\text{ mm} > 38\text{ mm}$$

For corrosion protection purposes, there must also be a minimum 75 mm of cover between the facing steel and the soil. Based on figure F.1, the 100 mm thick temporary shotcrete provides adequate cover for the permanent facing steel.

Check 4 - Development and Splices of Reinforcement

The permanent facing reinforcement consists entirely of No. 13 deformed bars. Therefore, the development lengths and splice lengths are identical to those computed for the temporary facing.

Step 11 - Serviceability Checks

Shotcrete Construction Facing

Because of the temporary nature of the wall, the serviceability requirements are waived for the construction facing.

Permanent Facing

Upper Cantilever Serviceability Check - Reinforcement Distribution

Per section 8.16.8 of AASHTO [30], the reinforcement must be distributed such that the steel stress does not exceed that given by the following equation:

$$F_S = \frac{z}{(d_C A_E)^{1/3}} \leq 0.6F_Y$$

$$z = 130 \text{ k/in} = 2.28 \times 10^4 \text{ kN/m}$$
$$d_C = 0.05 \text{ m}$$
$$A_E = (100 \text{ mm})(S_H)/(A_{TOTAL}/A_{LB})$$
$$A_E = (100 \text{ mm})(1500 \text{ mm})/((645 \text{ mm}^2)/(129 \text{ mm}^2))$$
$$A_E = 0.03 \text{ m}^2$$
$$0.6F_Y = 0.6(420 \text{ MPa}) = 252 \text{ MPa}$$
$$F_S = \frac{2.28 \times 10^4 \text{ kN/m}}{((0.05 \text{ m})(0.03 \text{ m}^2))^{1/3}} = 199 \text{ MPa}$$

From step 9, the service moment is 0.678 kN-m/m. The corresponding service steel stress is determined from straight-line theory of reinforced concrete:

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$$
$$\rho = A_S/(bd)$$
$$n = E_S/E_C$$
$$j = 1 - k/3$$
$$f_S = m_S\, b/(A_S j d)$$
$$E_C = 4734\sqrt{f'_c \text{ (MPa)}} = 4734\sqrt{28 \text{ MPa}}$$
$$E_C = 2.50 \times 10^4 \text{ MPa}$$

$$E_S = (29{,}000{,}000 \text{ psi})(1 \text{ MPa})/(145 \text{ psi}) = 2.00 \times 10^5 \text{ MPa}$$

Substituting the correct values into the above expressions, the service steel stress is computed to be:

$$n = (2.00 \times 10^5 \text{ MPa})/(2.50 \times 10^4 \text{ MPa}) = 8.00$$
$$\rho = A_S/(bd) = (645 \text{ mm}^2)/((1500 \text{ mm})(100 \text{ mm})) = 0.0043$$
$$\rho n = (0.0043)(8.00) = 0.0344$$
$$k = \sqrt{2(0.0344) + (0.0344)^2} - 0.0344 = 0.230$$
$$j = 1 - 0.230/3 = 0.923$$
$$f_S = (0.678 \text{ kN-m/m})(1.50 \text{ m})/((645 \text{ mm}^2)(0.923)(0.10 \text{ m}))$$
$$f_S = 17.1 \text{ MPa}$$

Since $f_S \leq F_S$, the steel distribution is adequate. (OK)

Overall Displacements of the Wall

Per section 2.4.6, the construction-induced vertical and horizontal permanent displacements at the top of the wall can be expected to be on the order of 0.2 percent of the height of the wall, or about 10 to 15 mm, for the given site soil conditions (medium dense to dense silty clayey sands). Displacements can be anticipated to decrease back from the wall in general accordance with the recommendations given on figure 2.7.

5.2.1.2 Seismic Loading Condition

In accordance with AASHTO [30], Service Load Group VII defines the seismic loading condition. Because of the temporary nature of the shotcrete construction facing, only the permanent facing is considered for the seismic loading condition.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

Step 1 is identical to that presented for the static loading condition.

Step 2 - Compute the Allowable Nail Head Loads

The nominal nail head strengths for all credible failure mechanisms are calculated using equations 4.1 through 4.4 and the methodology demonstrated in appendix F. The allowable nail head loads are computed from the nominal strengths in the following table for the permanent CIP concrete facing:

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Allowable Nail Head Load T_F (Group VII) (kN)
Facing Flexure	232	$0.89^a(232) = 206$
Facing Punching Shear	222	$0.89^a(222) = 198$
Headed-Stud Tensile Fracture	639	$0.67^a(639) = 428$

[a] Per section 3.22.2 of AASHTO [30].
See Table 4.3 for Group VII loading.

Therefore, the allowable nail head load T_F is computed to be 198 kN. That is, facing punching shear of the permanent facing is the controlling mode of failure.

Step 3 - Minimum Allowable Nail Head Service Load Check

This check is not applied to extreme event loading combinations.

Step 4 - Define the Allowable Nail Load Support Diagrams

Develop the allowable nail load diagram for each nail, by determining the allowable pullout resistance, the allowable nail head loads, and the allowable nail tendon tensile loads.

Allowable Nail Head Load

Per step 2, the allowable nail head load is 198 kN.

Allowable Pullout Resistance (Ground-Grout Bond)

$$Q = 1.33\alpha_Q Q_U$$
$$\alpha_Q = 0.50$$
$$Q_U = 60.0 \text{ kN/m}$$
$$Q = 0.67 (60.0 \text{ kN/m}) \text{ (table 4.5)}$$
$$= 40.2 \text{ kN/m}$$

Allowable Nail Tendon Tensile Load

$$T_N = 1.33\alpha_N T_{NN}$$
$$\alpha_N = 0.55$$
$$T_{NN} = A_N F_Y = (819 \text{ mm}^2)(0.420 \text{ kN/mm}^2) = 344 \text{ kN (for No. 32 nail tendon)}$$
$$T_N = 0.73(344 \text{ kN}) \text{ (table 4.5)} = 251 \text{ kN}$$

Step 5 - Select Trail Nail Lengths

In accordance with section 4.7.1 and figure 4.11, the nail length distribution for design purposes is as shown on figure 5.20. The maximum nail length has been calculated iteratively to be 8.5 meters (see below). The nail length distribution is:

Nail No.	Length (m)
1	8.5
2	8.5
3	7.2
4	4.7

The above nail length distribution is obtained from figure 4.11, as follows:

- The dimensionless nail pullout resistance, Q_D, is calculated:

$$Q_D = 1.33\alpha_Q Q_U/(\gamma S_V S_H) = (0.67)(60.0 \text{ kN/m})/[(20 \text{ kN/m}^3)(1.25 \text{ m})(1.50 \text{ m})]$$
$$= 1.07$$

- The dimensionless nail length is:

$$L/H = (8.5 \text{ m})/(5.25 \text{ m}) = 1.62$$

- Q_D / (L/H) = 1.07/1.62 = 0.66, giving an "R" factor of 0.37.

- Relative nail lengths are calculated from figure 4.11, for the nail head elevations shown on figure 5.20 and an "R" value of 0.37.

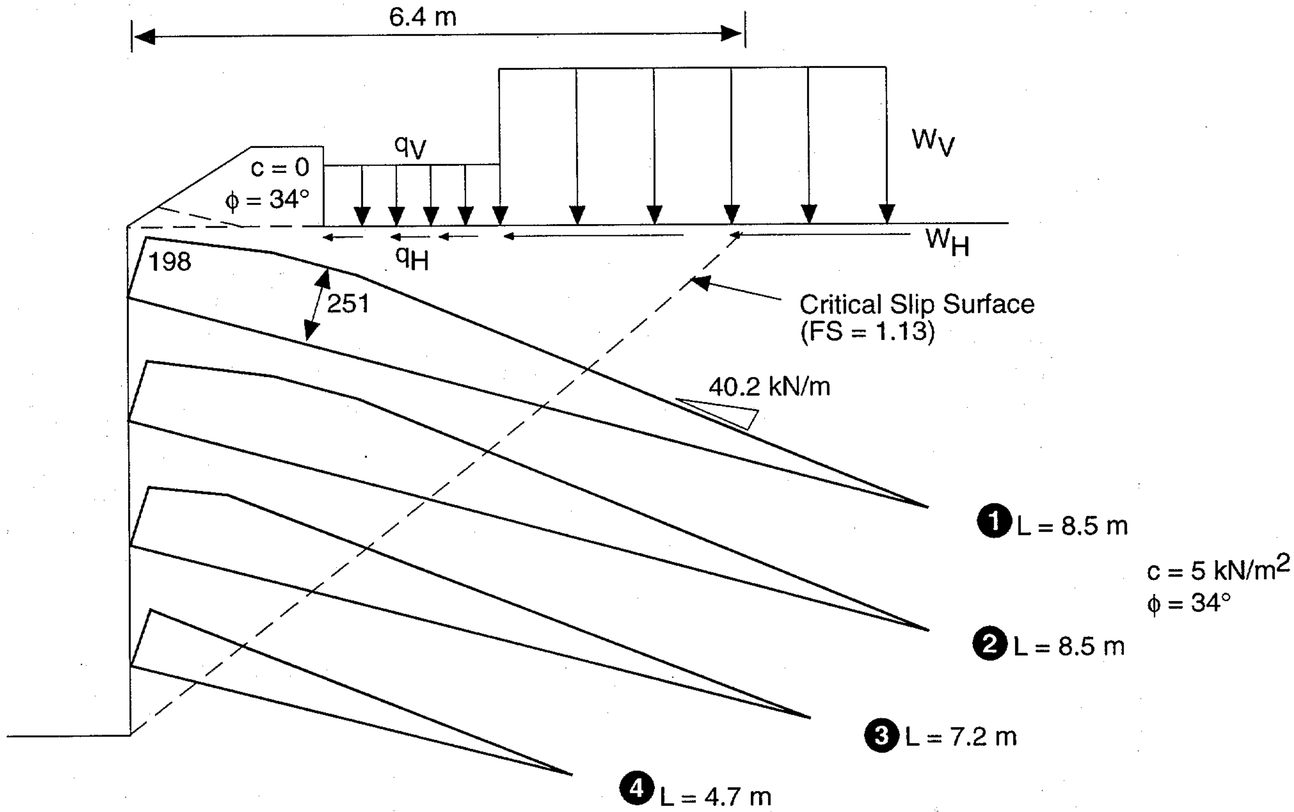
The allowable nail load support diagrams shown graphically on figure 5.20 were prepared in accordance with the procedure previously presented on figure 4.3.

Step 6 - Define the Ultimate Soil Strengths

Ultimate Friction Angle, $\phi_U = 34°$
Ultimate Cohesion, $c_U = 5$ kN/m^2

Step 7 - Calculate the Seismic Factor of Safety

In accordance with Manual table 4.5 and section 3.22 of the Standard Specifications for Highway Bridges, 15th Edition [30], the required minimum global soil factor of safety for seismic loading for this critical structure is 1.50/1.33 = 1.13. For the design trial cross-section and nail pattern shown on figure 5.20, the above soil strengths and allowable nail load diagrams, a calculated minimum factor of safety of 1.13 is obtained based on a computer solution for the following seismic loading conditions:



Surcharge:

$q_V = 25.1\ kN/m^2\ (f_V = 0.2)$

* $q_H\ (0.42g) = 95.8\ kN/m^2\ (f_H = 1.15)$

$q_H\ (0.32g) = 80.4\ kN/m^2\ (f_H = 1.15)$

$W_V = 74.0\ kN/m^2$

* $W_H\ (0.42g) = 31.1\ kN/m^2$

$W_H\ (0.32g) = 23.7\ kN/m^2$

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

* Slip surfaces for seismic co-efficient of 0.42g limited to exit depth of 6.4 m (per figures 5.16 and 4.14)

Figure 5.20 Bridge Abutment Wall Design Example
Critical Cross-Section
Seismic Loading (SLD)

- For slip surfaces exiting the reinforced slope within a horizontal distance of 6.4 meters from the top of wall facing (determined from figure 5.16 in accordance with figure 4.14), the applied pseudo-static seismic coefficient for a 0.40 g peak ground acceleration was taken as:

$$A = (1.45 - A_{PK})A_{PK}$$
$$A = (1.45 - 0.40)(0.40\ g) = 0.42\ g$$

- For more deep-seated slip surfaces, an applied pseudo-static seismic coefficient of A = (0.8)(0.40 g) = 0.32 g (i.e., 80 percent of peak ground acceleration) was chosen, which is consistent with allowable overall displacements of up to 15 mm for the retaining wall system under design seismic loading (in accordance with Section 4.7.1, Seismic Design, Item 3). The 15 mm permanent seismically-induced displacement for the bridge abutment is indicated by the structural engineers as being tolerable.

For the seismic loading condition, the equivalent vertical and horizontal surcharge loads, representative of the pile loads transmitted to the reinforced soil, are computed using the same f_V and f_H factors used for the Group I loading condition i.e., f_V value of 0.2 and f_H value of 1.15.

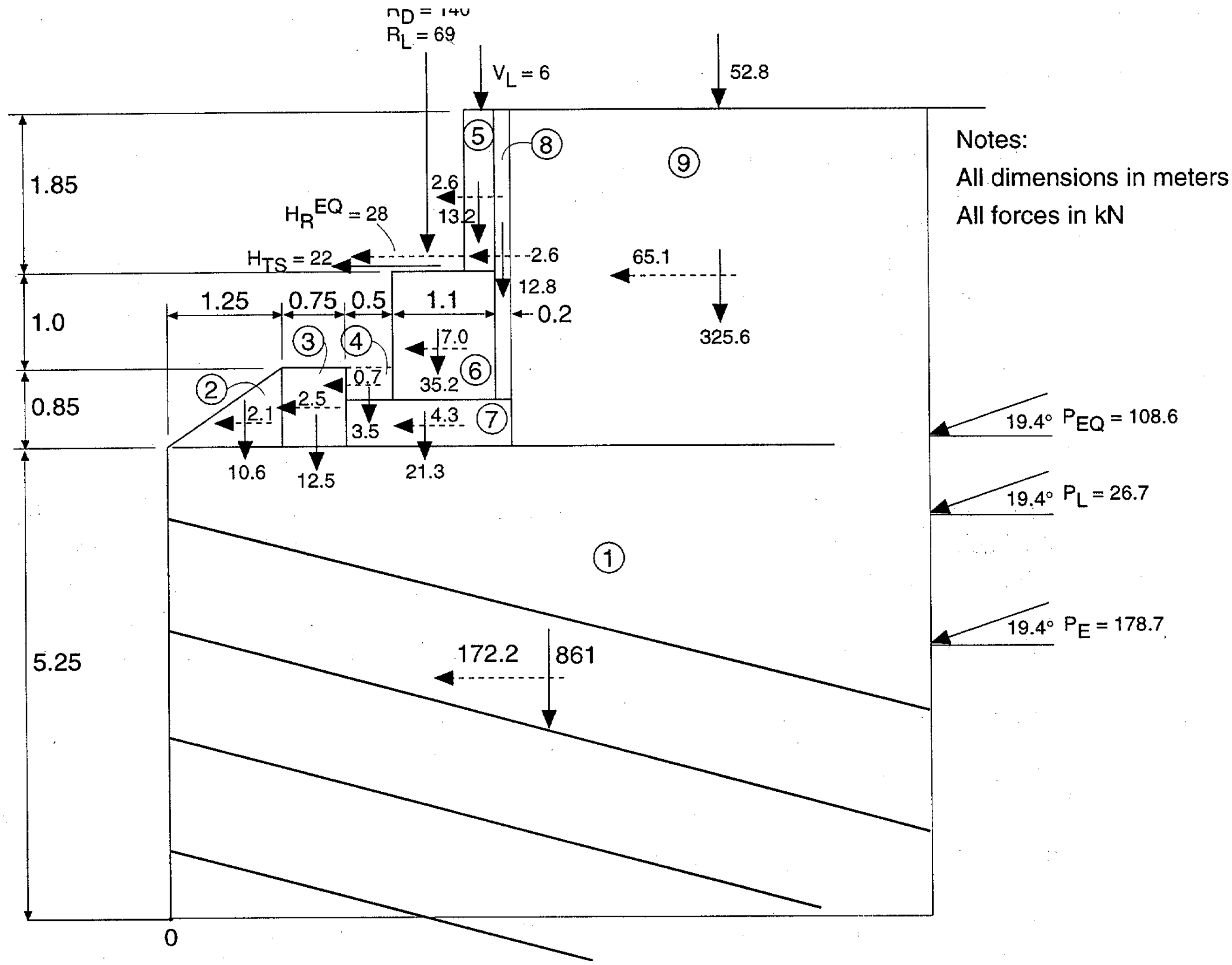
Limiting-equilibrium analysis for seismic loading corresponding to a peak ground acceleration of 0.40g has indicated that the nail tendon size should be increased from No. 25 (for static) to No. 32 (for seismic), and the maximum nail lengths should be increased from 6.6 meters (for static loading) to 8.5 meters (for seismic loading), i.e., about a 30 percent increase in length. For this design example, seismic heading controls the design.

Step 8 - External Stability Check of Nailed Block (Static and Seismic)

For the design nail length of 8.5 m determined in step 7, the static and seismic bearing capacity checks are performed for the soil nail reinforced block of ground acting as a gravity wall structure (figure 4.12). Pertinent parameters for the analysis include the following:

Soil Unit Weight, γ = 20.0 kN/m^3
Ultimate Soil Friction Angle, ϕ_U = 34°
Ultimate Soil Cohesion, c_U = 5.0 kN/m^2
Ultimate Fill Friction Angle, ϕ_U = 34°
Ultimate Fill Cohesion, c_U = 0 kN/m^2
Seismic Coefficient, A = 0.2g (= 0.5A_{PK}, per Section 4.7.1, Seismic Design, Item 4)

In accordance with section 6, Division I-A of the Standard Specifications for Highway Bridges, 15th Edition [30], the lateral forces due to the active block behind the reinforced soil are computed using the methods of Coulomb and Mononobe and Okabe (including the cohesive component of soil strength). Figure 5.21 shows the static and seismic earth pressure loads, inclined at an angle of 19.4 degrees above horizontal in accordance with the recommendations of figure 4.12. These are computed to be:



Load	Vertical Force (kN)	Horizontal Movement Arm (m)	Horizontal Force (kN)	Vertical Movement Arm (m)
P_E	59.4	8.2	168.6	2.98
P_L	8.9	8.2	25.2	4.48
P_{EQ}	36.1	8.2	102.4	5.37
R_D	42.0	2.9	–	–
R_L	20.7	2.9	–	–
V_L	1.8	3.45	–	–
V_{LL}	52.8	6.00	–	–
1	861	4.1	172.2	2.62
2	10.6	0.83	2.1	5.53
3	12.5	1.62	2.5	5.67
4	1.1	2.25	0.7	5.92
5	4.0	3.45	2.6	8.02
6	10.6	3.05	7.0	6.42
7	6.4	2.90	4.3	5.50
8	3.8	3.70	2.6	7.35
9	325.6	6.00	65.1	7.10
H_{TS}	–	–	22	7.10
H_R^{EQ}	–	–	28	7.10

Figure 5.21 Bridge Abutment Wall Design Example Bearing Capacity Stability Check (SLD)

Static Active Earth Load, P_E = 178.7 kN/m

Horizontal component = $P_E \cos(19.4°)$ = 168.6 kN/m

Vertical component = $P_E \sin(19.4°)$ = 59.4 kN/m

Live Earth Pressure Load, P_L = 26.7 kN/m

Horizontal component = $P_L \cos(19.4°)$ = 25.2 kN/m

Vertical component = $P_L \sin(19.4°)$ = 8.9 kN/m

Seismic Earth Load, P_{EQ} = 108.6 kN/m

Horizontal component = $P_{EQ} \cos(19.4°)$ = 102.4 kN/m

Vertical component = $P_{EQ} \sin(19.4°)$ = 36.1 kN/m

For moment equilibrium calculations, the assumed points of application of P_E, P_L and P_{EQ} are at the back of the reinforced block of ground at 0.33H, 0.5H and 0.6H above the base of the soil block, respectively:

Point of Application of P_E = 0.33H = 0.33(8.95 m) = 2.98 m

Point of Application of P_L = 0.5H = 0.50(8.95 m) = 4.48 m

Point of Application of P_{EQ} = 0.6H = 0.60(8.95 m) = 5.37 m

Static Loading (Load Group I)

For Load Group I loading (table 4.3), forces and moments are computed below (see figure 5.21):

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_E(E)$[a]	59.4	8.20	487.1
$P_L(L)$[a]	8.9	8.20	73.0
$R_D(D)$[a, b]	42.0	2.90	121.8
$R_L(L)$[a, b]	20.7	2.90	60.0
$V_L(L)$[a, b]	1.8	3.45	6.2
$V_{LL}(L)$[a]	52.8	6.00	316.8
1 (D)[a]	861.0	4.10	3530.1
2 (D)[a]	10.6	0.83	8.8
3 (D)[a]	12.5	1.62	20.3
4(D)[a, b]	1.1	2.25	2.4
5(D)[a, b]	4.0	3.45	13.7
6(D)[a, b]	10.6	3.05	32.2
7(D)[a, b]	6.4	2.90	18.5
8(D)[a, b]	3.8	3.70	14.2
9(D)[a]	325.6	6.00	1953.6
Sum	1421.1 (N)	N/A	6658.6

Horizontal Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
P_E (E)[a]	168.6	2.98	-502.4
P_L(L)[a]	25.2	4.48	-112.9
Sum	193.8 (H)	N/A	-615.3

[a] Notations used in the Standard Specifications for Highway Bridges, 15th Edition [30].

[b] For the nail reinforced block shown on figure 5.21, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14 for a depth of intersection of the piles by the base of the reinforced block of 5.25.m)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

$N \;=\; 1421.1 \text{ kN}$
$X_O \;=\; (6658.6 \text{ kN-m/m} - 615.3 \text{ kN-m/m})/(1421.1 \text{ kN/m}) \;=\; 4.25 \text{ m}$
Since $2X_o > B$
$q_{MAX} \;=\; N/(B) \;=\; (1421.1 \text{ kN/m})/(8.20 \text{ m})] \;=\; 173 \text{ kN/m}^2$

In accordance with equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30], the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B$ (see above) = 8.20 m. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$q_{ULT} = (5.0 \text{ kN/m}^2)(42.0)(1.19)(0.77) + 0.5(20.0 \text{ kN/m}^3)(8.20 \text{ m})(41.0)(0.89)(0.68)$
$q_{ULT} \;=\; 2{,}227 \text{ kN/m}^2$

For a required factor of safety of 2.5 (see section 4.7.1, step 8), the allowable bearing pressure is:

$q_{ALL} \;=\; (q_{ULT})/FS$
$q_{ALL} \;=\; (2{,}227 \text{ kN/m}^2)/2.5 = 891 \text{ kN/m}^2$

Since $q_{MAX} < q_{ALL}$, the bearing capacity is adequate. (OK)

As noted above, the eccentricity of the vertical resultant force is negative so the eccentricity check is satisfied.

Static Loading (Load Group IV)

For Load Group IV loading (table 4.3), forces and moments are computed below (see figure 5.21):

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_E(E)$[a]	59.4	8.20	487.1
$P_L(L)$[a]	8.9	8.20	73.0
$R_D(D)$[a, b]	42.0	2.90	121.8
$R_L(L)$[a, b]	20.7	2.90	60.0
$V_L(L)$[a, b]	1.8	3.45	6.2
$V_{LL}(L)$[a]	52.8	6.00	316.8
1 (D)[a]	861.0	4.10	3530.1
2 (D)[a]	10.6	0.83	8.8
3 (D)[a]	12.5	1.62	20.3
4(D)[a, b]	1.1	2.25	2.4
5(D)[a, b]	4.0	3.45	13.7
6(D)[a, b]	10.6	3.05	32.2
7(D)[a, b]	6.4	2.90	18.5
8(D)[a, b]	3.8	3.70	14.2
9(D)[a]	325.6	6.00	1953.6
Sum	1421.1 (N)	N/A	6658.6

Horizontal Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
P_E (E)[a]	168.6	2.98	-502.4
$P_L(L)$[a]	25.2	4.48	-112.9
$H_{TS}(RST)$[a, b]	22.0	7.10	-156.2
Sum	215.8 (H)	N/A	-771.5

[a] Notations used in the Standard Specifications for Highway Bridges, 15th Edition [30].

[b] For the nail reinforced block shown on figure 5.21, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14 for a depth of intersection of the piles by the base of the reinforced block of 5.25.m)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

$N = 1421.1$ kN

$X_O = (6658.6 \text{ kN-m/m} - 771.5 \text{ kN-m/m})/(1421.1 \text{ kN/m}) = 4.14$ m

Since $2X_o > B$

$q_{MAX} = N/(B) = (1421.1 \text{ kN/m})/(8.20 \text{ m})] = 173 \text{ kN/m}^2$

In accordance with equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30], the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, B′ = B (see above) = 8.20 m. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$$q_{ULT} = (5.0 \text{ kN/m}^2)(42.0)(1.19)(0.75) + 0.5(20.0 \text{ kN/m}^3)(8.20 \text{ m})(41.0)(0.89)(0.65)$$
$$q_{ULT} = 2{,}132 \text{ kN/m}^2$$

For a required factor of safety of 2.0 (see section 4.7.1, step 8), the allowable bearing pressure is:

$$q_{ALL} = (q_{ULT})/FS$$
$$q_{ALL} = (2{,}132 \text{ kN/m}^2)/2.0 = 1{,}066 \text{ kN/m}^2$$

Since $q_{MAX} < q_{ALL}$, the bearing capacity is adequate. (OK)

As noted above, the eccentricity of the vertical resultant force is negative so the eccentricity check is satisfied.

Seismic Loading

For the seismic loading condition, Load Group VII (table 4.3) is of interest. The forces and moments are computed below (figure 5.21):

Vertical Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
$P_E(E)$[a]	59.4	8.20	487.1
$P_{EQ}(EQ)$[a]	36.1	8.20	295.8
$R_D(D)$[a, b]	42.0	2.90	121.8
1 (D)[a]	861.0	4.10	3530.1
2 (D)[a]	10.6	0.83	8.8
3 (D)[a]	12.5	1.62	20.3
4(D)[a, b]	1.1	2.25	2.4
5(D)[a, b]	4.0	3.45	13.7
6(D)[a, b]	10.6	3.05	32.2
7(D)[a, b]	6.4	2.90	18.5
8(D)[a, b]	3.8	3.70	14.2
9(D)[a]	325.6	6.00	1953.6
Sum	1373.0 (N)	N/A	6498.4

Horizontal Loads and Moments

Item	Force (kN/m)	Moment Arm (m)	Moment About Point O (kN-m/m)
P_E (E)[a]	168.6	2.98	-502.4
P_{EQ} (EQ)[a]	102.4	5.37	-550.1
H_R^{EQ}(EQ)[a, b]	28.0	7.10	-198.8
1 (EQ)[a]	172.2	2.62	-451.2
2 (EQ)[a]	2.1	5.53	-11.7
3 (EQ)[a]	2.5	5.67	-14.2
4 (EQ)[a, b]	0.7	5.92	-4.1
5 (EQ)[a, b]	2.6	8.02	-21.2
6 (EQ)[a, b]	7.0	6.42	-45.2
7 (EQ)[a, b]	4.3	5.50	-23.4
8 (EQ)[a, b]	2.6	7.35	-18.8
9 (EQ)[a]	65.1	7.10	-462.2
Sum	558.2 (H)	N/A	-2303.3

[a] Notations used in the Standard Specifications for Highway Bridges, 15th Edition [30].

[b] For the nail reinforced block shown on figure 5.21, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14 for a depth of intersection of the piles by the base of the reinforced block of 5.25.m)

Stability Criteria

Referring to figure 4.12, the maximum bearing pressure and the location of the resultant force are checked as follows:

$N = 1373.0$ kN
$X_O = (6498.4$ kN-m/m - 2303.3 kN-m/m)/(1373.0 kN/m) $= 3.06$ m
$q_{MAX} = N/(2X_o) = (1373.0$ kN/m)/[2(3.06 m)] $= 224$ kN/m^2

In accordance with equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30], the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) = 2(3.06 m) = 6.12 m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:
$q_{ULT} =$ (5.0 kN/m^2)(42.0)(1.14)(0.38)+ 0.5(20.0 kN/m^3)(6.12 m)(41.0)(0.92)(0.24)
$q_{ULT} =$ 645 kN/m^2

For a required seismic factor of safety of 1.9 (see section 4.7.1, step 8), the allowable bearing pressure is:

$q_{ALL} = (q_{ULT})/FS$
$q_{ALL} =$ (645 kN/m^2)/1.9 = 339 kN/m^2

Since $q_{MAX} < q_{ALL}$, the bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle third of the block of reinforced soil:

$e = B/2 - X_o =$ (8.20 m)/2 - 3.06 m = 1.04 m
$B/6 =$ (8.20 m)/6 = 1.37 m

Since $e < B/6$, the resultant is within the middle third. (OK)

Overall Stability

For the uniform nail length pattern selected (see figure 5.22), the deep ground water table, and the overall geometry of the slope, deep seated failure surfaces passing beneath the toe of the wall will not be critical.

Step 9 - Check the Upper Cantilever

In accordance with section 6 Commentary, division I-A of the Standard Specifications for Highway Bridges, 15th Edition [30], equation C6-4, the approach developed by Mononobe and Okabe for a free-standing retaining structure is used to develop the active seismic loading on the upper cantilever. For a soil friction angle of 34°, a soil-to-wall interface friction angle of (2/3)34° = 22°, a vertical wall, a soil profile behind the wall as shown on figure 5.13, and a horizontal pseudo-static seismic coefficient of 0.32g (i.e. $0.8A_{PK}$), the combined static and dynamic active earth pressure load P_{AE} determined from a Mononobe-Okabe type computer solution is 9.14 kN/m length of wall. This can be considered to consist of a static earth pressure load of 2.95 kN/m (see step 9 for the static loading condition) and a dynamic earth pressure load of 6.19 kN/m. The load components normal to the wall are (2.95)cos(22°) = 2.74 kN/m (static) and (6.19)cos(22°) = 5.74 kN/m (dynamic).

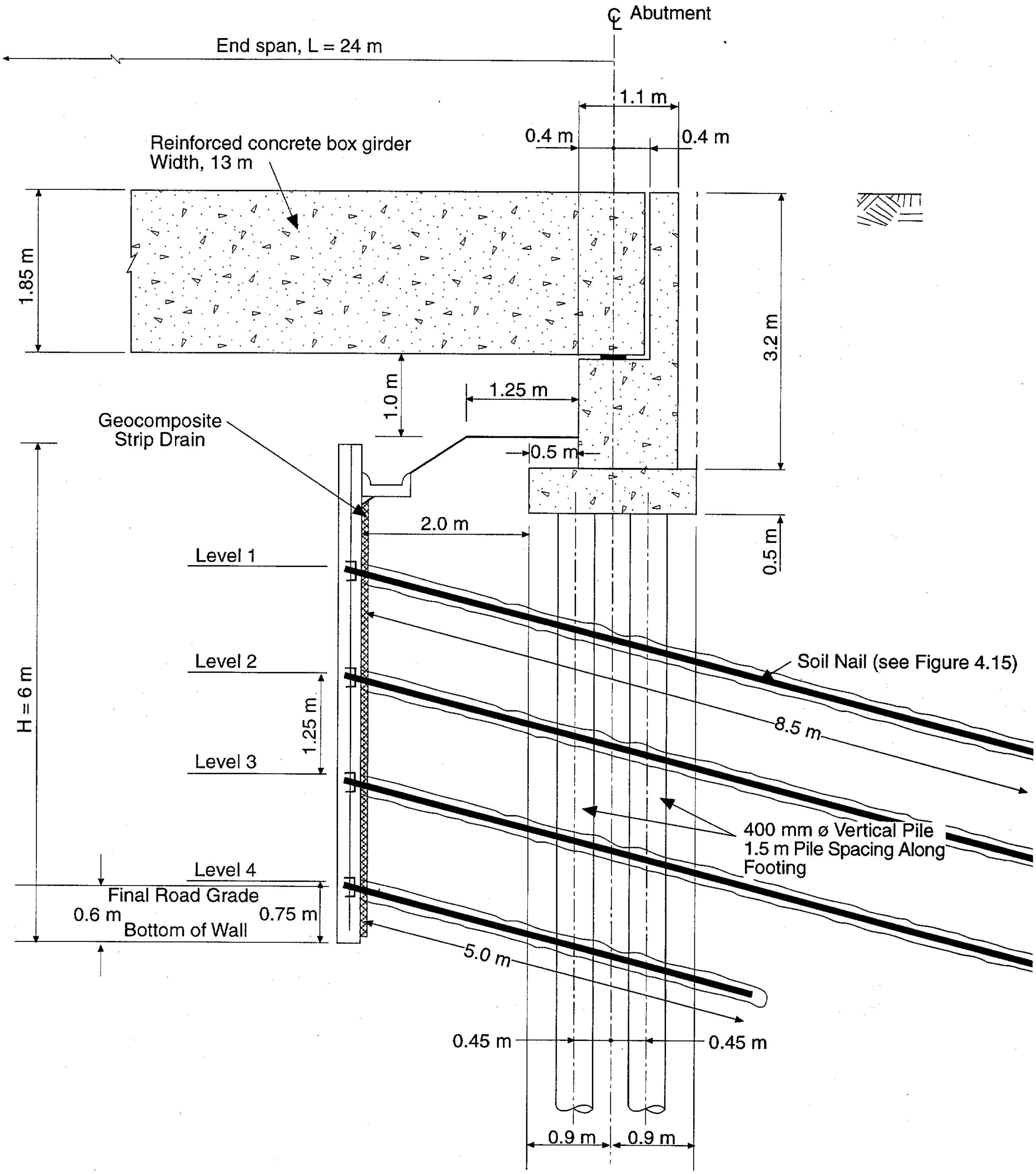


Figure 5.22 Bridge Abutment Wall Final Design Section (SLD)

Shear Check

From force equilibrium, compute the one-way unit service shear force at the level of the upper row of nails (conservative), as indicated on figure 4.13:

$$v = v_{STATIC} + v_{DYNAMIC}$$
$$v = (2.74 + 5.74)\ \text{kN/m}^2 = 8.48\ \text{kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 8-49 of AASHTO [30]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28\ \text{MPa}}\ (0.1\ \text{m}) = 87.8\ \text{kN/m}$$

From table 4.4, the facing shear strength factor equals 0.89. Therefore, the allowable one-way unit shear is computed to be:

$$V = (0.89)(87.8\ \text{kN/m}) = 78.1\ \text{kN/m}$$

Since $v < V$, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the one-way unit service moment at the level of the upper row of nails (conservative) as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever. The dynamic force is assumed to occur at 0.6H above the base of the cantilever:

$$m_S = [(0.33)(v_{STATIC}) + (0.6)(v_{DYNAMIC})](H)$$
$$m_S = [(0.33)(2.74\ \text{kN/m}) + (0.6)(5.74\ \text{kN/m})](0.75\ \text{m})$$
$$m_S = 3.26\ \text{kN-m/m}$$

Compute the nominal unit moment resistance of the facing. Using the procedure demonstrated in appendix F, the nominal unit moment resistance in the vertical direction over the nail locations, $m_{V,NEG}$, is computed to be 17.4 kN-m/m. From table 4.4, the (seismic loading) strength factor for facing flexure is 0.89. Therefore, the allowable one-way unit moment for the upper cantilever is:

$$M = (0.89)\ m_{V,NEG} = (0.89)(17.4\ \text{kN-m/m}) = 15.5\ \text{kN-m/m}$$

Since $m_S < M$, the design for flexure is adequate. (OK)

<u>Step 10 - Check the Facing Reinforcement Details</u>

The facing reinforcement details, being independent of type of loading, have been previously considered for the static loading condition and need not be repeated for the seismic loading condition.

<u>Step 11 - Serviceability Checks</u>

The serviceability checks are not applicable to seismic loading conditions because of the extreme nature of the limit state.

<u>Design Section</u>

Based on the above analyses, the final design cross-section is shown on figure 5.22. It should be noted that, to simplify construction, the chosen final plan nail lengths are different to the nail length distributions used in the analyses. The upper three rows of nails are of uniform length equal the maximum nail length indicated by the above analyses (i.e., 8.5 m). Overall stability analysis has confirmed that the bottom row of nails can be installed with shorter lengths in the range of 5.0 meters.

5.2.2 Load and Resistance Factor Design (LRFD)

5.2.2.1 Static Loading Condition

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Strength I and IV Limit States (table 4.6) defines the static loading condition for this problem.

<u>Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design</u>

It is required to replace a bridge abutment end slope with a vertical soil nail wall in order to add an additional traffic lane and shoulder. The bridge abutment is pile-supported by two rows of 400 mm diameter 15 m long friction piles, at spacings of 1.5 meters along the length of the footing (see figure 5.23). The vertical wall will be located 2 meters in front of the pile cap, resulting in a wall height above road grade of 4.65 meters. Removal of the 1.5H:1V end slope will therefore provide an additional 7 meters of roadway width and this is sufficient for the additional traffic lane and shoulder. The seismic design condition corresponds to a peak ground acceleration of 0.40 g.

The native soils at the site consist of a silty clayey sand that exploration test pitting demonstrated will stand in 2.5 meter high unsupported cuts for a minimum of several days. The site investigation has confirmed that the ground water table is located well below the base of the wall. The approach fill material behind the abutment backwall is a select granular material. Average in-situ densities are 20.0 kN/m^3, and the soil strength parameters are estimated at a friction angle of 34.0 degrees and a cohesion of 5.0 kN/m^2 for the native soils, and a friction angle of 34 degrees for the overlying approach fill. It is estimated that an ultimate pullout resistance on the order of 60.0 kN/m should be achievable with drillhole diameters on the order of 200 mm, within the native soils. This estimated pullout resistance will have to be demonstrated for the contractor's proposed installation method during the initial stages of the wall construction.

Because the soil nail wall will comprise part of a bridge abutment and is a critical structure, encapsulated nails will be used for corrosion protection. Based on experience in these types of materials, anticipated wall displacements associated with wall construction are in the range of 10-15 mm (see step 11), and these displacements have been determined to be acceptable.

The wall will have a maximum height of 5.25 meters (from ground surface immediately behind the wall to the base of the wall), with a vertical face. The critical design cross section is shown on figure 5.23 and will have the center of the pile cap located 2.9 meters behind the facing and the base of the pile cap at a depth of 3.7 meters below the ground surface. The trial vertical nail spacing will be at 1.25 meters as shown on figure 5.23, with a horizontal nail spacing of 1.5 meters to conform to the pile spacing, and the nails will be installed at 15.0 degrees below horizontal for constructability reasons.

It is anticipated that No. 25 or No. 29 minimum Grade 420 bars (Soft Metric designation - see table F.2), approximately 7 to 8 meters long, will be required for support of the bridge abutment wall. Based on local practice and material availability, the trial design will assume a "standard" temporary shotcrete construction facing (28-day compressive strength of 28 Mpa) will have a nominal thickness of 100 mm, and will be reinforced with a single layer of 152 x 152 MW18.7 x MW18.7 (6 x 6-W2.9 x W2.9) welded-wire mesh, and two No. 13 Grade 420 continuous horizontal waler bars along each row of nails.

The nails will be connected to the construction facing by a 225 mm square, 25 mm thick bearing plate. The nails will be installed in vertical columns on 1.5 meter centers, to conform to the layout of the piles. The permanent facing will be a cast-in-place (CIP) concrete wall (28-day compressive strength of 28 MPa), 200 mm thick, reinforced with No. 13 Grade 420 deformed bars on 300 mm centers vertically and horizontally, and connected to the nail heads with a headed-stud connection system. Seismic loading will be evaluated only for the permanent facing.

Step 2 - Compute the Design Nail Head Strengths

The nominal nail head strengths for all credible failure mechanisms are calculated using equations 4.1 through 4.4 and the methodology demonstrated in appendix F. The nominal strengths are shown below and differ from the values calculated for the cutslope design example because of a) the absence of vertical bearing bars in the shotcrete construction facing and b) the reduced vertical nail spacing for the bridge abutment wall versus the cutslope wall. The design nail head strengths for Strength Limit States are computed from the nominal strengths as indicated in the tables below for both the temporary shotcrete construction facing and the permanent CIP concrete facing.

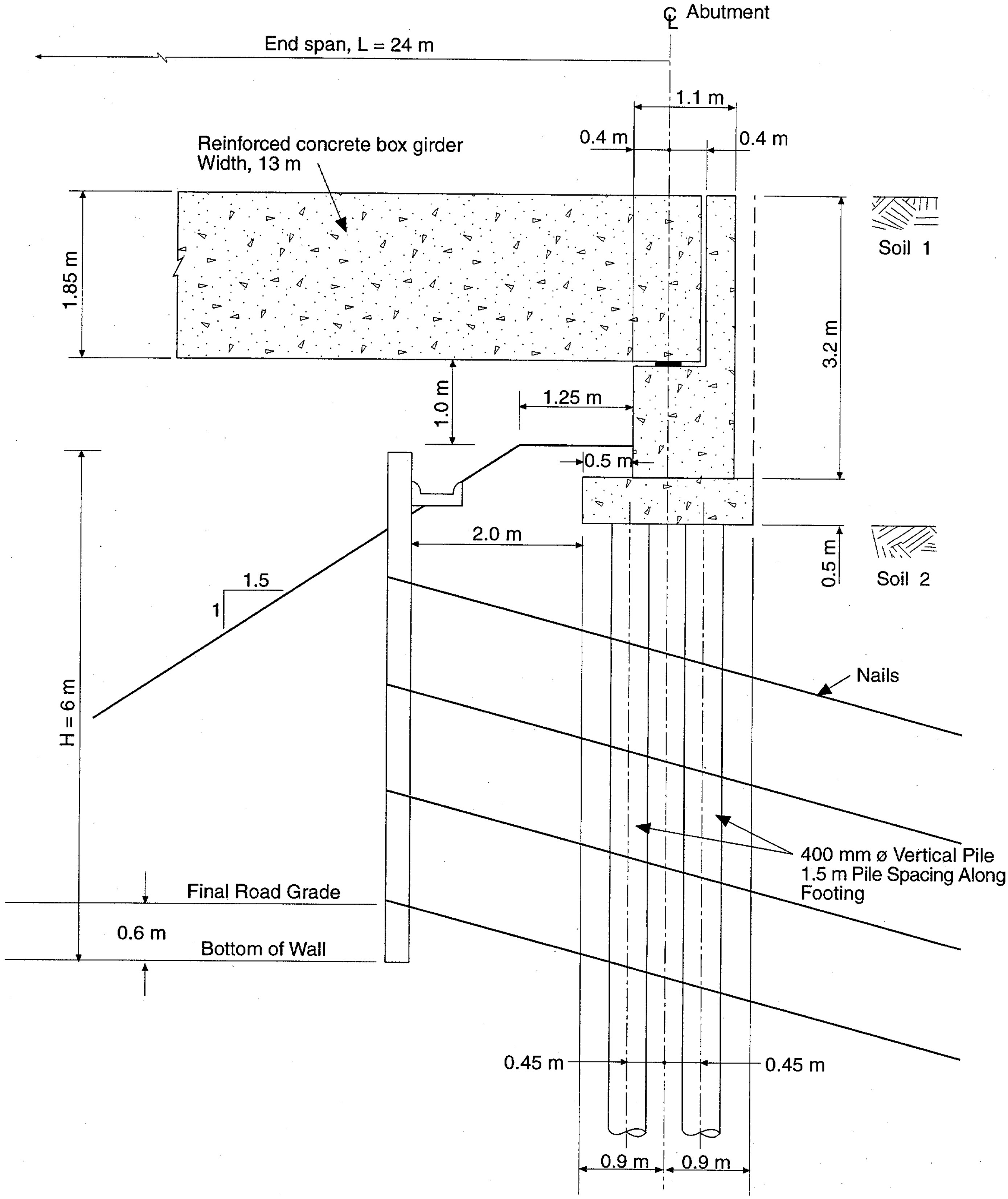


Figure 5.23 Bridge Abutment Wall

SHOTCRETE CONSTRUCTION FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Nail Head Strength T_F (kN)
Facing Flexure	97	$0.90^a(97) = 87$
Facing Punching Shear	210	$0.90^a(210) = 188$

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Nail Head Strength T_F (kN)
Facing Flexure	232	$0.90^a(232) = 209$
Facing Punching Shear	222	$0.90^a(222) = 200$
Headed-Stud Tensile Fracture	639	$0.67^a(639) = 428$

[a] See Table 4.7.

Therefore, the design nail head strength T_F is computed to be 87 kN. That is, facing flexure of the temporary facing is the controlling mode of failure.

Step 3 - Minimum Design Nail Head Strength Check

Since the design problem has a somewhat complex surcharge loading distribution, the estimated nail head service load will be calculated from equation 4.7, with the active earth pressure load P_A determined from a Coulomb-type computer solution i.e., identify the critical active slip surface that results in the highest calculated earth pressure loading. Figure 5.14 shows the design dead and live loadings on the pile-supported bridge abutment, with the unfactored pile loadings corresponding to 301 kN/m vertically and 51.3 kN/m laterally. Because of the twin pile configuration, the pile head is assumed to have rotational fixity. The pile loadings are transmitted to the soil through pile-soil interaction. Vertical (theoretical t-z curves taken from Vijayvergiya [41] and using the method of Coyle and Reese [42]) and lateral (computer program LPILE [43]) soil-pile interaction analyses established that the proportions of the vertical and horizontal pile head loads transmitted to the nailed soil are a function of the depth at which potential slip surfaces intersect the piles. These proportions are shown plotted on figure 5.14. The pile loads were simulated by applying equivalent vertical and horizontal surcharges at the base of the pile cap, as shown on Figure 5.15 (calculated on figure 5.14). The equivalent vertical and horizontal surcharge loadings to be applied at the base of the pile cap are given on figure 5.14 in terms of the fractions of the applied pile loads (f_V vertical and f_H horizontal) transmitted to the soil. The pile load fractions are a function of the depth at which the critical slip surface intersects the piles (as shown on figure 5.14), and this depth must be estimated. For evaluation of the active earth pressure load (figure 5.15), the depth at which the critical slip surface intersects the piles is on the order of 2 meters, giving an f_V value of 0.07 (i.e., a vertical surcharge pressure of 11.7 kN/m^2) and a k_H value of 0.8 (i.e., a horizontal surcharge pressure of 22.8 kN/m^2). For simplicity, the retained earth and live load surcharge behind the bridge abutment are modeled as an equivalent vertical surcharge, as shown on figure 5.15. For a friction angle of 34 degrees, and the Group I surcharge loadings discussed above, the active earth pressure load corresponding to a horizontal, triangular earth pressure distribution is given by P_A equal to 134.6 kN/m (ignoring the cohesive component of soil strength in accordance with the discussion of section 2.4.5). Values for other parameters are as follows:

S_H = 1.50 m
S_V = 1.25 m
H = 5.25 m
F_F = 0.50

Substituting the above values into equation 4.7, to determine the nail head service load:

$$t_F = 2F_F\, P_A\, S_H\, S_V/H$$
$$t_F = (2.0)(0.50)(134.6\text{ kN/m})(1.5\text{ m})(1.25\text{ m})/(5.25\text{ m})$$
$$t_F = 48.1\text{ kN}$$

The factored nail head service load is computed using the load factor from table 4.6 (equal to 1.5 for active earth pressure), and is checked against the design nail head strength as indicated below:

$$\Gamma_{EH}(t_F) = 1.5(48.1\text{ kN}) = 72.2\text{ kN} < 87\text{ kN}$$

(OK - the design nail head strength exceeds the estimated factored nail head service load)

Step 4 - Define the Design Nail Strength Diagrams

Develop the design nail strength diagram for each nail by determining the design pullout resistance, the design nail head strength and the design nail tendon tensile strength.

Design Nail Head Strength

Per step 2, the design nail head strength is 87 kN.

Design Pullout Resistance (Ground-Grout Bond)

$Q = \Phi_Q Q_U$
Φ_Q = 0.70 (table 4.8)
Q_U = 60.0 kN/m
Q = (0.70)(60.0 kN/m) = 42.0 kN/m

Design Nail Tendon Tensile Strength

$T_N = \Phi_N T_{NN}$
$\Phi_N = 0.90$ (table 4.8)
$T_{NN} = A_N F_Y = (510\ mm^2)(0.420\ kN/mm^2) = 214\ kN$
$T_N = (0.90)(214\ kN) = 193\ kN$

Step 5 - Select Trial Nail Spacings and Lengths

In accordance with section 4.7.2 and figure 4.11, the nail length distribution for design purposes (Strength Limit State I) is as shown on figure 5.24. The maximum nail length has been calculated iteratively to be 6.9 meters (see step 7 below). The trial nail length distribution is:

Nail No.	Length (m)
1	6.9
2	6.9
3	5.9
4	3.8

The above nail length distribution for Strength Limit State I is obtained from figure 4.11, as follows:

- The dimensionless nail pullout resistance, Q_D, is calculated:

  $Q_D = \Phi_Q Q_U/(\Gamma_W \gamma S_V S_H) = (0.70)(60.0\ kN/m)/[(1.35)(20\ kN/m^3)(1.25\ m)(1.50\ m)]$
  $= 0.83$

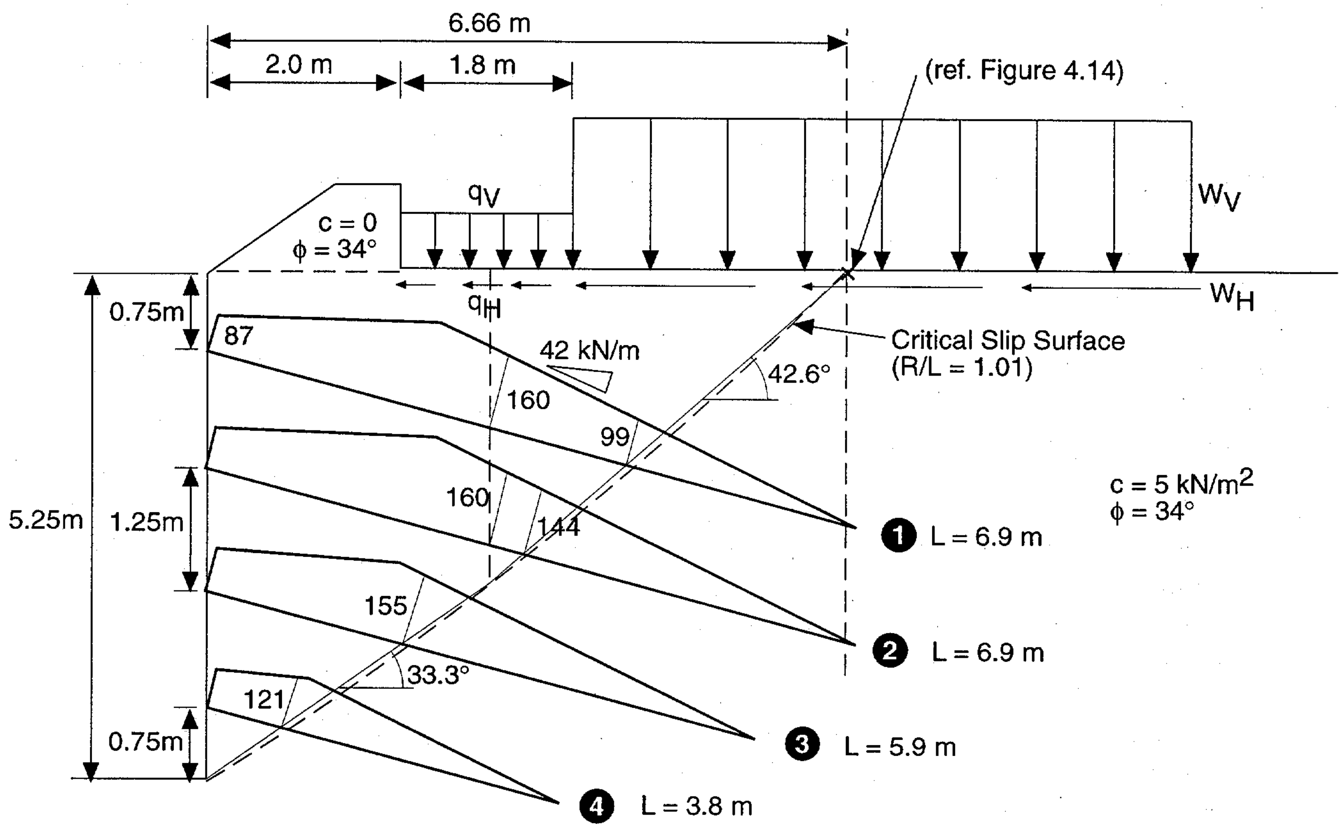
- The dimensionless nail length is:

  $L/H = (6.9\ m)/(5.25\ m)$
  $= 1.31$

- $Q_D / (L/H) = 0.83/1.31 = 0.63$, giving an "R" factor of 0.38.

- Relative nail lengths are calculated from figure 4.11 for the nail head elevations shown on figure 5.23 and an "R" value of 0.38.

The design nail strength diagrams are shown graphically on figure 5.24, prepared in accordance with the procedure previously presented on figure 4.3.



Factored Surcharge:

$q_V = 46.0$ kN/m^2

$q_H = 58.2$ kN/m^2

$W_V = 120.9$ kN/m^2

$W_H = 0$ kN/m^2

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

Figure 5.24 Bridge Abutment Wall Design Example Critical Cross-Section Static Loading - Strength Limit State I (LRFD)

Step 6 - Define the Design Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0^o
Ultimate Cohesion, c_U = 5.0 kN/m^2

From table 4.8, the soil resistance factors for this critical structure are as follows:

Φ_ϕ = 0.65
Φ_C = 0.90

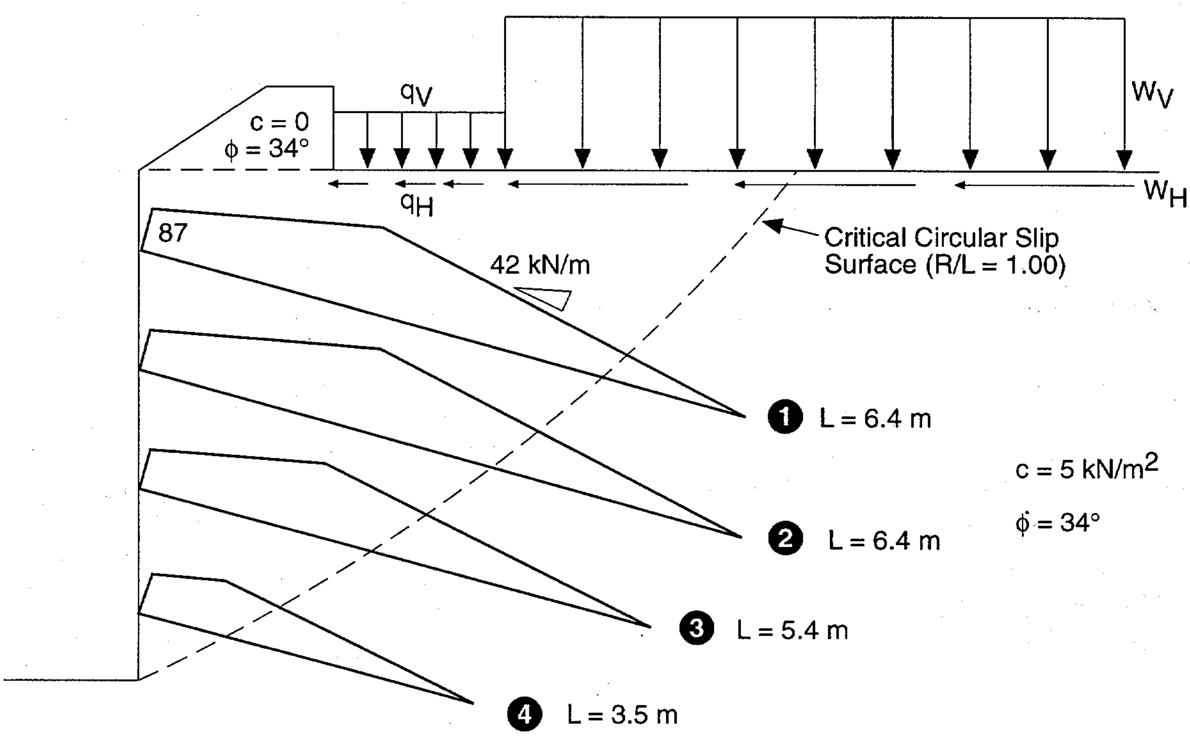
Design Friction Angle, $\phi = \tan^{-1}(\Phi_\phi \tan(\phi_U)) = \tan^{-1}(0.65[\tan(34.0^o)]) = 23.7^o$
Design Cohesion, $c = \Phi_C c_U = 0.90(5.0 \text{ kN/m}^2) = 4.5 \text{ kN/m}^2$

Step 7 - Calculate the Resistance/Load Ratio

For the trial design cross-section and nail pattern shown on figure 5.24, the above design soil strengths and design nail strength diagrams, the factored surcharge and soil self weight loads, a calculated minimum resistance/load (R/L) ratio (equivalent to a factor of safety using factored loads and design soil and nail strengths) of 1.01 is obtained, based on a computer solution (i.e., factored resistances equal or exceed factored loads). **For conciseness of presentation of this design example, the initially chosen trial design is in fact the design that meets the minimum required resistance/load ratio of 1.0. In a real world production design, some design re-runs and iterations would be required to arrive at an acceptable design.** For slip surfaces passing through the toe of the wall, the critical slip surface is conservatively estimated to intersect the piles at a depth of about 4 meters, giving an f_V value of 0.2 (factored vertical surcharge of 46.0 kN/m^2) and an f_H value of 1.15 (factored horizontal or lateral surcharge of 58.2 kN/m^2) - see figures 5.14 and 5.24. The computer solution critical slip surface is shown on figure 5.24. For this application, Strength Limit State IV loading is less critical than Strength Limit State I loading. Figure 5.25 shows the design nail strength diagrams for Strength Limit State IV loading for a nail pattern that provides a calculated minimum resistance/load (R/L) ratio of 1.00. As can be seen from a comparison of figures 5.24 and 5.25, Strength Limit State I requires longer nail lengths and is more critical in this case.

Hand Calculation Check

The following provides an approximate method for performing a hand calculation check, if required, for the Strength Limit State I loading problem described on figure 5.24. The method is approximate in that it is based on force balance only and more reliable results will generally be obtained using methods that address both force and moment equilibrium. In order to facilitate the hand calculation check, the critical circular slip surface may be approximated by a bilinear wedge, as shown on figure 5.24. The forces on the 'active' Block A and the 'resisting' Block B are illustrated on figure 5.26 for the bilinear wedge and are computed below:



Factored Surcharge:

q_V = 37.7 kN/m^2

q_H = 44.1 kN/m^2

W_V = 99.9 kN/m^2

W_H = 0 kN/m^2

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

1 Nail number

Figure 5.25 Bridge Abutment Wall Design Example Critical Cross-Section Static Loading - Strength Limit State IV (LRFD)

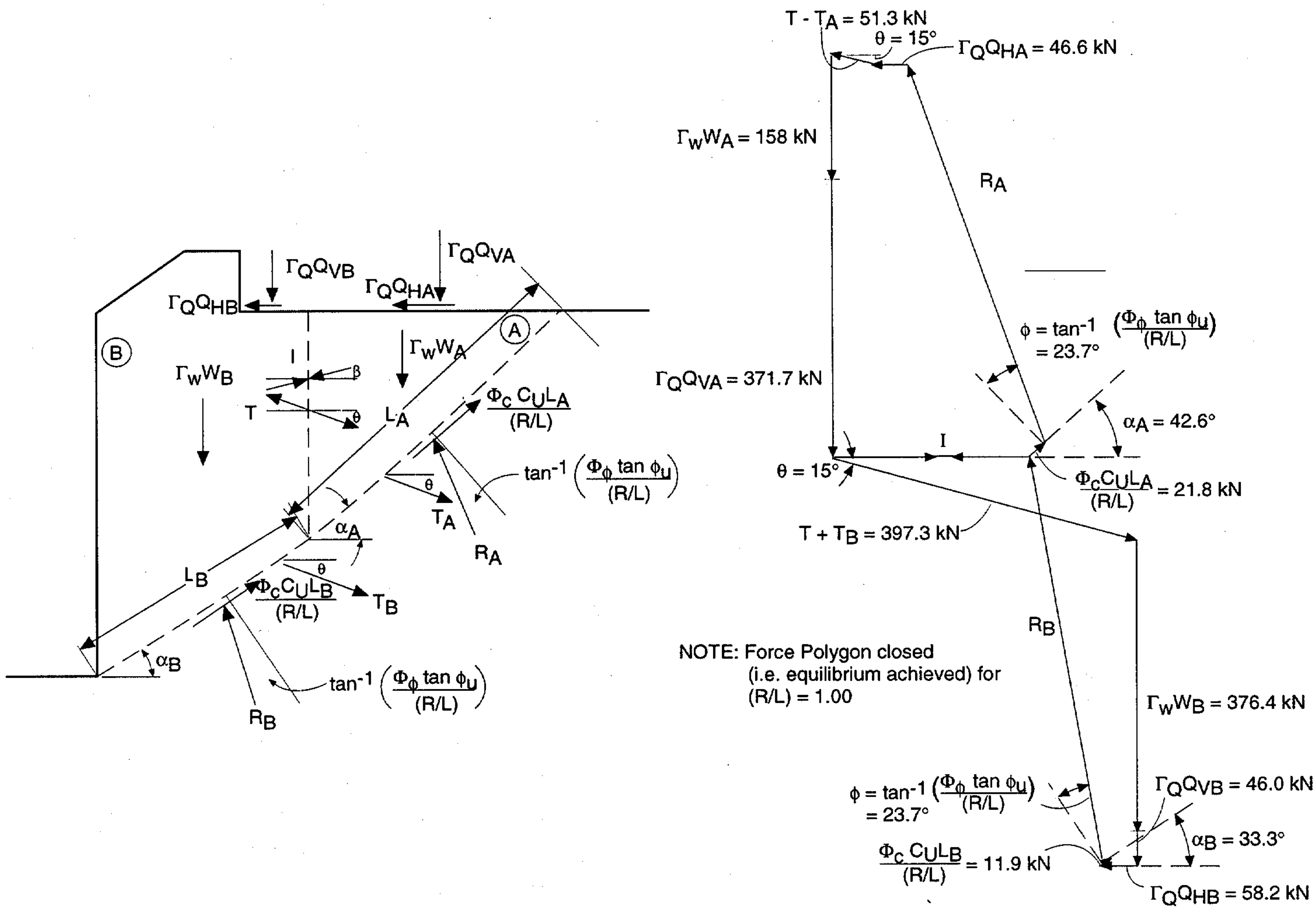


Figure 5.26 Bridge Abutment Design Example Force Diagram and Polygon (LRFD) Hand Calculation Check

Block A

Base Slope Angle, α_A = 42.6°
Base Length, L_A = 4.85 m
Block Weight, W_A = $(5.85\ m^2)(20.0\ kN/m^3)$ = 117.0 kN/m
Soil Weight Load Factor, Γ_W =1.35 (table 4.6)
Factored Soil Block Weight, $\Gamma_W W_A$ = 1.35(117.0 kN/m) = 158.0 kN/m
Factored Vertical Surcharge $\Gamma_Q Q_{VA}$ = $(0.8m)\ (46.0\ kN/m^2) + (2.77m)\ (120.9\ kN/m^2)$
= 371.7 kN/m
Factored Horizontal Surcharge $\Gamma_Q Q_{HA}$ = $(0.8m)\ (58.2\ kN/m^2) + (2.77m)\ (0\ kN/m^2)$
= 46.6 kN/m
Design Nail Force, T_A = $[(99.0\ kN)^{(1)} + (144.0\ kN)^{(2)}]/(1.50\ m)$
= 162.0 kN/m

$^{(n)}$ Nail Number

Block B

Base Slope Angle, α_B = 33.3°
Base Length, L_B = 3.59 m
Block Weight, W_B = $(13.94\ m^2)(20.0\ kN/m^3)$ = 278.8 kN/m
Soil Weight Load Factor, Γ_W =1.35 (table 4.6)
Factored Soil Block Weight, $\Gamma_W W_B$ = 1.35(278.8 kN/m) = 376.4 kN/m
Factored Vertical Surcharge $\Gamma_Q Q_{VB}$ = $(1.0m)\ (46.0\ kN/m^2)$ = 46.0 kN/m
Factored Horizontal Surcharge $\Gamma_Q Q_{HB}$ = $(1.0m)\ (58.2\ kN/m^2)$ = 58.2 kN/m
Design Nail Force, T_B = $[(155.0\ kN)^{(3)} + (121.0\ kN)^{(4)}]/(1.5\ m)$
= 184.0 kN/m

Interslice

Design Nail Force, T = $[(160.0\ kN)^{(1)} + (160.0\ kN)^{(2)}]/(1.5\ m)$
= 213.3 kN/m

Assume that the interslice soil force, I, is inclined at an angle 'β' to the horizontal (figure 5.26). The resistance/load ratio (R/L) is then applied to the design soil strengths, as indicated below.

Equilibrium

Define $\phi = \tan^{-1}[\Phi_\phi \tan(\phi_U)/(R/L)]$

Block A

Vertical

$\Gamma_W W_A + \Gamma_Q Q_{VA} + (T_A\text{-}T)\sin(\theta)\text{-}(I)\sin(\beta)\text{-}(\Phi_C)(c_U)(L_A)\sin(\alpha_A)/(R/L)\text{-}(R_A)\cos(\alpha_A\text{-}\phi)=0$

Horizontal

$(I)\cos(\beta)+(T_A\text{-}T)\cos(\theta)+(\Phi_C)(c_U)(L_A)\cos(\alpha_A)/(R/L)\text{-}(R_A)\sin(\alpha_A\text{-}\phi) - \Gamma_Q Q_{HA} = 0$

Block B

Vertical

$\Gamma_W W_B + \Gamma_Q Q_{VB} +(T_B+T)\sin(\theta)+(I)\sin(\beta)\text{-}(\Phi_C)(c_U)(L_B)\sin(\alpha_B)/(R/L)\text{-}(R_B)\cos(\alpha_B\text{-}\phi)=0$

Horizontal

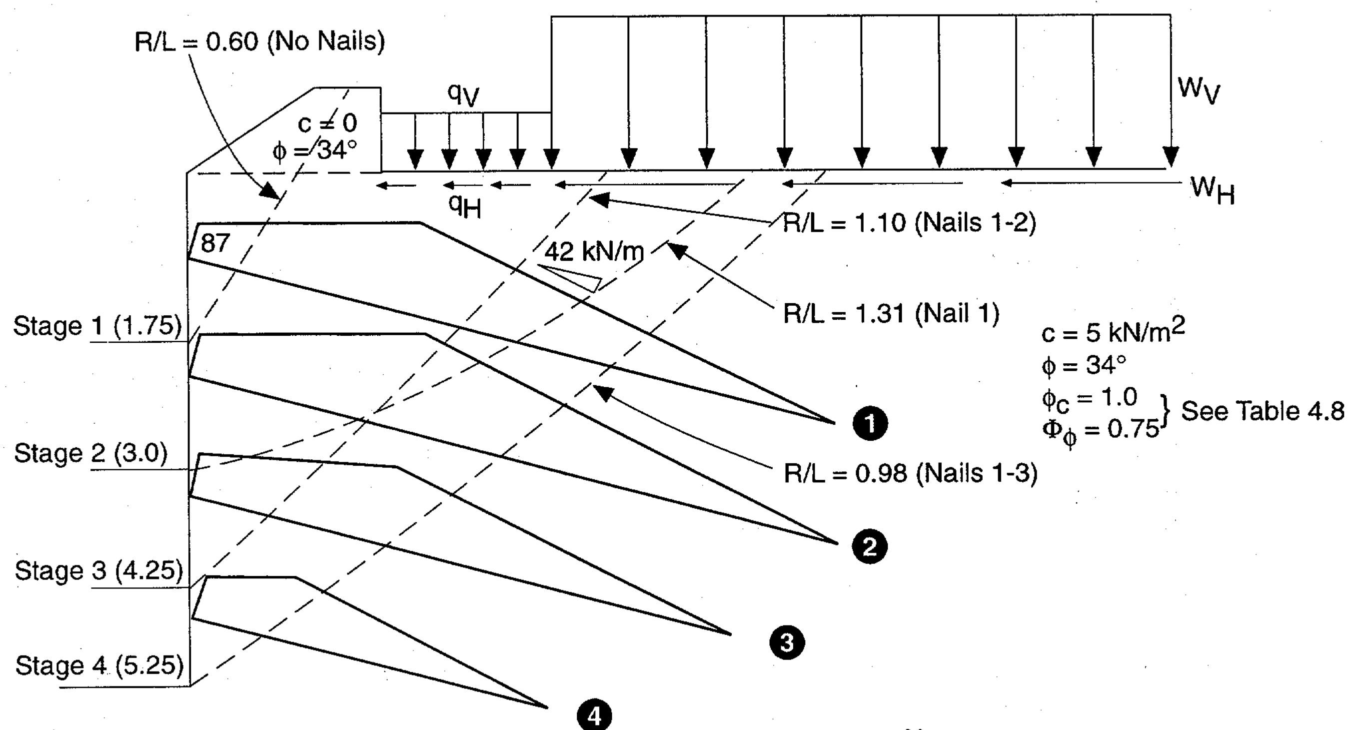
$(I)\cos(\beta)\text{-}(T_B+T)\cos(\theta)\text{-}(\Phi_C)(c_U)(L_B)\cos(\alpha_B)/(R/L)+(R_B)\sin(\alpha_B\text{-}\phi) + \Gamma_Q Q_{HB} = 0$

The previous equations can be reduced to the following:

$$[1+\tan(\beta)\tan(\alpha_A-\phi)]\ [\{-\Gamma_W W_B - \Gamma_Q Q_{VB} - (T_B + T)\sin(\theta)+ \Phi_C c_U(L_B)\sin(\alpha_B)/(R/L)\}\tan(\alpha_B-\phi)+ (T_B + T)\cos(\theta)+ \Phi_C c_U(L_B)\cos(\alpha_B)/(R/L) - \Gamma_Q Q_{HB}] = [1+\tan(\beta)\tan(\alpha_B-\phi)]\ [\{\Gamma_W W_A + \Gamma_Q Q_{VA} + (T_A - T)\sin(\theta)- \Phi_C c_U(L_A)\sin(\alpha_A)/(R/L)\}\tan(\alpha_A-\phi)-(T_A - T)\cos(\theta)- \Phi_C c_U(L_A)\cos(\alpha_A)/(R/L) + \Gamma_Q Q_{HA}]$$

The above equation can be solved iteratively for resistance/load ratio (R/L) (e.g., by use of a spreadsheet), if the interslice force angle β is assumed. For example, assuming $\beta = 0$, the calculated resistance/load ratio (R/L) = 1.00, which is similar to the computer-generated solution. The force polygon for the hand calculation is shown on figure 5.26.

The results of stability assessments during construction are shown on figure 5.27. Calculated resistance/load ratios for each stage of construction, following excavation of each lift and prior to installation of the associated row of nails, are shown together with the corresponding critical slip surface in each case. As noted on figure 5.27, the "equivalent" surface surcharge representing the load transmitted by the piles to the reinforced soil zone is modified as the depth of the critical slip surface increases, in accordance with the information presented on figure 5.14. The estimated "equivalent" surface surcharges representing the pile loads transmitted to the soil are shown on figure 5.27, for each stage of construction. These estimated "equivalent" surcharges were determined by estimating the average depth at which the critical slip surface intersected the piles in each case, and applying the appropriate



Notes:
All dimensions in meters
All allowable nail loads in kN
Horizontal nail spacing = 1.5 m
❶ Nail number

Stage	q_V (kN/m^2)	q_H (kN/m^2)	W_V (kN/m^2)	W_H (kN/m^2)
1	–	–	120.9	0
2	11.5 (f_V = 0.05)*	30.4(f_H = 0.60)	120.9	0
3	18.4(f_V = 0.08)	48.1(f_H = 0.95)	120.9	0
4	29.9(f_V = 0.13)	58.2(f_H = 1.15)	120.9	0

* See Figure 5.14

Figure 5.27 Bridge Abutment Wall Design Example Construction Stability - Strength Limit State I (LRFD)

fractions of the total pile vertical and lateral loads, as determined from figure 5.14. Figure 5.27 indicates that prior to the installation of the first row of nails, the calculated resistance/load ratio is 0.60, which indicates that for the specified load factors of table 4.6 and the resistance factors of tables 4.7 and 4.8, the factored resistances are less than the factored loads. However, as noted previously, this initial lift condition has been evaluated in the field by design phase test pitting. If extra precaution is considered desirable, the first row of nails can be installed in stages with a limited length of open unsupported face at any time. In addition, the nails can be installed through a protective stabilization berm (see Example Plans, appendix A).

It can also be seen that for this problem, while the stage 2 and stage 3 excavation conditions satisfy the minimum construction phase resistance/load ratio requirements, the stage 4 condition (i.e., following the final cut but before installation of the final row of nails) has a calculated resistance/load ratio of 0.98 which is slightly less than the required value of 1.00. In general, this situation could be addressed by a) limiting the length of open excavation in the final cut before nail installation or b) increasing the nail lengths slightly (about 2 percent in this case) to ensure that the factored resistances equal or exceed the factored loads. In this case, however, as will be seen in the following discussion, seismic loading conditions require that longer nails of greater capacity be used and the final nail pattern will therefore satisfy the required construction phase resistance/load ratio value of unity.

Step 8 - External Stability Check of Nailed Block

Defer the external stability check until the final soil nail lengths are defined for the seismic loading condition.

Step 9 - Check the Upper Cantilever

The height of the upper cantilever from the top row of nails to the top of retained soil is identical (0.75 meters) for both the temporary shotcrete and permanent CIP wall facings. Therefore, the static loading is identical in the two cases. Because both the facing thickness and steel content are increased in the permanent facing, the permanent facing is less critical by inspection. Therefore, for the static loading condition, only the construction facing upper cantilever needs to be evaluated.

For the method of construction, the appropriate earth pressure coefficient for the upper cantilever design is an active earth pressure coefficient. For a soil friction angle of 34°, a soil-to-wall interface friction angle of (2/3)(34°) = 22°, a vertical wall, and a soil profile behind the wall as shown on figure 5.22, the active earth pressure load P_A determined from a Coulomb-type computer solution is 2.95 kN/m length of wall. The load component normal to the wall has value of (2.95 kN/m)cos(22°) = 2.74 kN/m, applied at the lower third point of the cantilever.

Shear Check

The factored one-way unit service shear force at the level of the upper row of nails (conservative), is calculated below:

$v = 2.74$ kN/m (see above)
$\Gamma_{EH} = 1.50$ (table 4.6)
$\Gamma_{EH}\, v = (1.5)(2.74 \text{ kN/m}) = 4.11$ kN/m

Compute the nominal one-way unit shear strength of the facing based on equation 5.8.3.3-3 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28 \text{ MPa}}\,(0.05 \text{ m}) = 43.9 \text{ kN/m}$$

From table 4.7, the facing shear resistance factor, Φ_F, equals 0.90. Therefore, the design one-way unit shear strength is:

$$V = \Phi_F V_{NS} = 0.90(43.9 \text{ kN/m}) = 39.5 \text{ kN/m}$$

Since $\Gamma_{EH}\, v = 4.11$ kN/m $< V$, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the factored one-way unit service moment at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever:

$m_S = (0.33)(H)(v)$
$m_S = (0.33)(0.75 \text{ m})(2.74 \text{ kN/m}) = 0.678$ kN-m/m
$\Gamma_{EH} = 1.50$ (table 4.6)
$\Gamma_{EH}\, m_S = (1.5)(0.678 \text{ kN-m/m}) = 1.017$ kN-m/m

Compute the nominal unit moment resistance of the facing. Using the procedure demonstrated in appendix F, the nominal unit moment resistance in the vertical direction over the nail locations $m_{V,NEG}$ is computed to be 2.53 kN-m/m. From table 4.7, the resistance factor Φ_F for facing flexure is 0.90. Therefore, the design one-way unit moment resistance for the upper cantilever is:

$$M = \Phi_F\, m_{V,NEG} = (0.90)(2.53 \text{ kN-m/m}) = 2.28 \text{ kN-m/m}$$

Since $\Gamma_{EH}\, m_S = 1.017 < M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

Shotcrete Construction Facing

Check 1 - Waler Reinforcement

Two No. 13 horizontal waler bars attached beneath the bearing plate will be placed continuously between nail heads in each nail row.

Check 2 - Minimum/Maximum Reinforcement Ratios

Minimum Reinforcement Ratio Requirements

The minimum reinforcement ratio requirements in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] are waived for the temporary shotcrete construction facing per the discussion presented in section 4.7.2.

Maximum Reinforcement Ratio Requirements

Section 5.7.3.3 of AASHTO [29] contains a provision for the maximum amount of reinforcement allowed in a flexural member. Per the discussion presented in section 4.7.2, this provision is not applicable in the case of a soil nail wall.

Check 3 - Minimum Cover Requirements

For the temporary shotcrete construction facing, the reinforcement is placed at the center of the facing (see figure F.1).

Check 4 - Development and Splices of Reinforcement

Splicing of No. 13 Waler Bars

Per section 5.11.5.3 of AASHTO [29], the splice length, L_S, for a Class C splice must equal or exceed the greater of 0.30 m or $L_D = 1.7L_{DB}$, where L_{DB} is computed from section 5.11.2:

$$L_{DB} = 0.019(A_B\,(\text{mm}^2))(F_Y\,(\text{MPa}))/\sqrt{f'_C\,(\text{MPa})}$$

$$L_{DB} = 0.019(129\ \text{mm}^2)(420\ \text{MPa})/\sqrt{28.0\ \text{MPa}} = 195\ \text{mm}$$

$$L_D = 1.7L_{DB} = 1.7(195\ \text{mm}) = 332\ \text{mm}$$

Therefore, provide the minimum 350 mm of splice length.

Splicing of MW 18.7 Mesh

Per section 5.11.6.2 of AASHTO [29], the splice length between outermost crosswires must equal or exceed the greater of (S_{WIRE} + 50 mm), $1.5L_{DB}$, or 150 mm. L_{DB} is computed from AASHTO equation 5.11.2.5.2-1:

$$L_{DB} = 3.25(A_{WIRE}\ (mm^2))(F_Y\ (MPa))/(((S_{WIRE}\ (mm))\sqrt{f'_C\ (MPa)}\ ))$$
$$L_{DB} = 3.25(18.7\ mm^2)(420\ MPa)/((152\ mm)\sqrt{28.0\ MPa}\ )$$
$$L_{DB} = 31.7\ mm$$
$$L_D = 1.5L_{DB} = 1.5(31.7\ mm) = 47.6\ mm$$
$$S_{WIRE} + 50.0\ mm = 152\ mm + 50.0\ mm = 202\ mm$$

Therefore, use a minimum of 200-mm splices for the wire mesh.

Permanent Facing

Check 1 - Waler Reinforcement

There are no applicable requirements.

Check 2 - Minimum and Maximum Reinforcement Ratios

Minimum Reinforcement Ratio Requirements

Per section 5.10.8 of AASHTO [29], the minimum required amount of shrinkage and temperature reinforcement near exposed surfaces of walls and slabs is given by equation 5.10.8.2-1, and must be greater than:

$$0.76A_G/F_Y$$

For one nail spacing:

$$0.76A_G/F_Y = 0.76(1500\ mm)(200\ mm)/(420\ MPa) = 543\ mm^2$$

A_{TOTAL} = 645 mm^2, which is adequate.

Maximum Reinforcement Ratio Requirements

Section 5.7.3.3 of AASHTO [29] contains a provision for the maximum amount of reinforcement allowed in a flexural member. Per the discussion presented in section 4.7.2, this provision is not applicable in the case of a soil nail wall.

Check 3 - Minimum Cover Requirements

Per section 5.12.3 of AASHTO [29], the minimum cover on the front side of the facing is specified at 50 mm. Based on the design arrangement illustrated on figure F.1, the cover to the headed studs is calculated below:

t_C = 200 mm - t_{PL} - L_{HS} = 200 mm - 25 mm - 125 mm = 50 mm

50 mm $\geq$ 50 mm (OK)

The minimum required cover between the permanent facing reinforcing steel and the CIP concrete/temporary shotcrete interface is 38 mm. Based on figure F.1, this cover is:

t_c = 100 mm - 12.7 mm = 87.3 mm > 38 mm

For corrosion protection purposes, there must also be a minimum 75 mm of cover between the facing steel and the soil. Based on figure F.1, the 100 mm thick temporary shotcrete provides adequate cover for the permanent facing steel.

Check 4 - Development and Splices of Reinforcement

The permanent facing reinforcement consists entirely of No. 13 deformed bars. Therefore, the development lengths and splice lengths are identical to those computed for the temporary facing.

Step 11 - Serviceability Checks

Shotcrete Construction Facing

Because of the temporary nature of the wall, the serviceability requirements are waived for the construction facing.

Permanent Facing

Upper Cantilever Serviceability Check - Reinforcement Distribution

Per section 5.7.3.4 of AASHTO [29], the reinforcement must be distributed such that the steel stress does not exceed that given by the following equation:

$$F_S = \frac{z}{(d_C A_E)^{1/3}} \leq 0.6F_Y$$

$$z = 130 \text{ k/in} = 2.28 \times 10^4 \text{ kN/m}$$

$$d_C = 0.05 \text{ m}$$

$$A_E = (100 \text{ mm})(S_H)/(A_{TOTAL}/A_{LB})$$

$$A_E = (100 \text{ mm})(1500 \text{ mm})/((645 \text{ mm}^2)/(129 \text{ mm}^2))$$

$$A_E = 0.03 \text{ m}^2$$

$$0.6F_Y = 0.6(420 \text{ MPa}) = 252 \text{ MPa}$$

$$F_S = \frac{2.28 \times 10^4 \text{ kN/m}}{((0.05 \text{ m})(0.03 \text{ m}^2))^{1/3}} = 199 \text{ MPa}$$

From step 9, the service moment is 0.678 kN-m/m. The corresponding service steel stress is determined from straight-line theory of reinforced concrete:

$k = \sqrt{2\rho n + (\rho n)^2} - \rho n$
$\rho = A_S/(bd)$
$n = E_S/E_C$
$j = 1 - k/3$
$f_S = m_S\, b\,/(A_S j d)$
$E_C = 4734\sqrt{f'_c\,(\text{MPa})} = 4734\sqrt{28\ \text{MPa}}$
$E_C = 2.50 \times 10^4$ MPa
$E_S = (29{,}000{,}000\ \text{psi})(1\ \text{MPa})/(145\ \text{psi}) = 2.00 \times 10^5$ MPa

Substituting the correct values into the above expressions, the service steel stress is computed to be:

$n = (2.00 \times 10^5\ \text{MPa})/(2.50 \times 10^4\ \text{MPa}) = 8.00$
$\rho = A_S/(bd) = (645\ \text{mm}^2)/((1500\ \text{mm})(100\ \text{mm})) = 0.004$
$\rho n = (0.0043)(8.00) = 0.0344$
$k = \sqrt{2(0.0344) + (0.0344)^2} - 0.0344 = 0.230$
$j = 1 - 0.230/3 = 0.923$
$f_S = (0.678\ \text{kN-m/m})(1.50\ \text{m})/((645\ \text{mm}^2)(0.923)(0.10\ \text{m}))$
$f_S = 17.1$ MPa

Since $f_S \leq F_S$, the steel distribution is adequate. (OK)

Overall Displacements of the Wall

Per section 2.4.6, the construction-induced vertical and horizontal permanent displacements at the top of the wall can be expected to be on the order of 0.2 percent of the height of the wall, or about 10 to 15 mm, for the given site soil conditions (medium dense to dense silty clayey sands). Displacements can be anticipated to decrease back from the wall in general accordance with the recommendations given in figure 2.7.

5.2.2.2 Seismic Loading Condition

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Extreme Events I Limit State (table 4.6) considers seismic loading. Because of the temporary nature of the shotcrete construction facing, only the permanent facing is considered for the seismic loading condition.

Step 1 - Set Up Critical Design Cross-Section and Select a Trial Design

Step 1 is identical to that presented for the static loading condition.

Step 2 - Compute the Design Nail Head Strengths

The nominal nail head strengths for all credible failure modes are calculated using equations 4.1 through 4.4 and the methodology demonstrated in appendix F. The design nail head strengths for the permanent CIP concrete facing are computed from the nominal strengths in the following table.

PERMANENT FACING

Failure Mode	Nominal Nail Head Strength T_{FN} (kN)	Design Strength T_F (kN)
Facing Flexure	232	$(1.00)^a(232) = 232$
Facing Punching Shear	222	$(1.00)^a(222) = 222$
Headed-Stud Tensile Fracture	639	$(1.00)^a(639) = 639$

[a] Per section 1.3.2.1 of AASHTO [29].
See table 4.7 .

Therefore, the design nail head strength T_F is computed to be 222 kN. That is, punching shear of the permanent facing is the controlling mode of failure.

Step 3 - Minimum Design Nail Head Strength Check

This check is not applied to extreme event loading combinations.

Step 4 - Define the Design Nail Strength Diagrams

Develop the design nail strength diagram for each nail, by determining the design pullout resistance, the design nail head strength, and the design nail tendon tensile strength.

Design Nail Head Strength

Per step 2, the design nail head strength is 222 kN.

Design Pullout Resistance (Ground-Grout Bond)

$Q = \Phi_Q Q_U$
$\Phi_Q = 0.80$ (table 4.8)
$Q_U = 60.0$ kN/m
$Q = (0.80)(60.0 \text{ kN/m}) = 48.0$ kN/m

Design Nail Tendon Tensile Strength

$T_N = \Phi_N T_{NN}$
$\Phi_N =$ 1.00 (table 4.8)
$T_{NN} = A_N F_Y = (645\ mm^2)(0.42\ kN/mm^2) =$ 271 kN (for No. 29 nail tendon)
$T_N = (1.00)(271\ kN) = 271\ kN$

Step 5 - Select Trial Nail Lengths

In accordance with section 4.7.2 and figure 4.11, the nail length distribution for design purposes is as shown on figure 5.28. The maximum nail length has been calculated iteratively to be 8.2 meters (see step 7 below). The nail length distribution is:

Nail No.	Length (m)
1	8.2
2	8.2
3	6.9
4	4.3

The above nail length distribution is obtained from figure 4.11, as follows:

- The dimensionless nail pullout resistance, Q_D, is calculated:

$Q_D = \phi_Q Q_U/(\Gamma_W\, \gamma S_V S_H) = (0.8)(60.0\ kN/m)/[(1.0^a)(20\ kN/m^3)(1.25\ m)(1.50\ m)]$
$= 1.28$

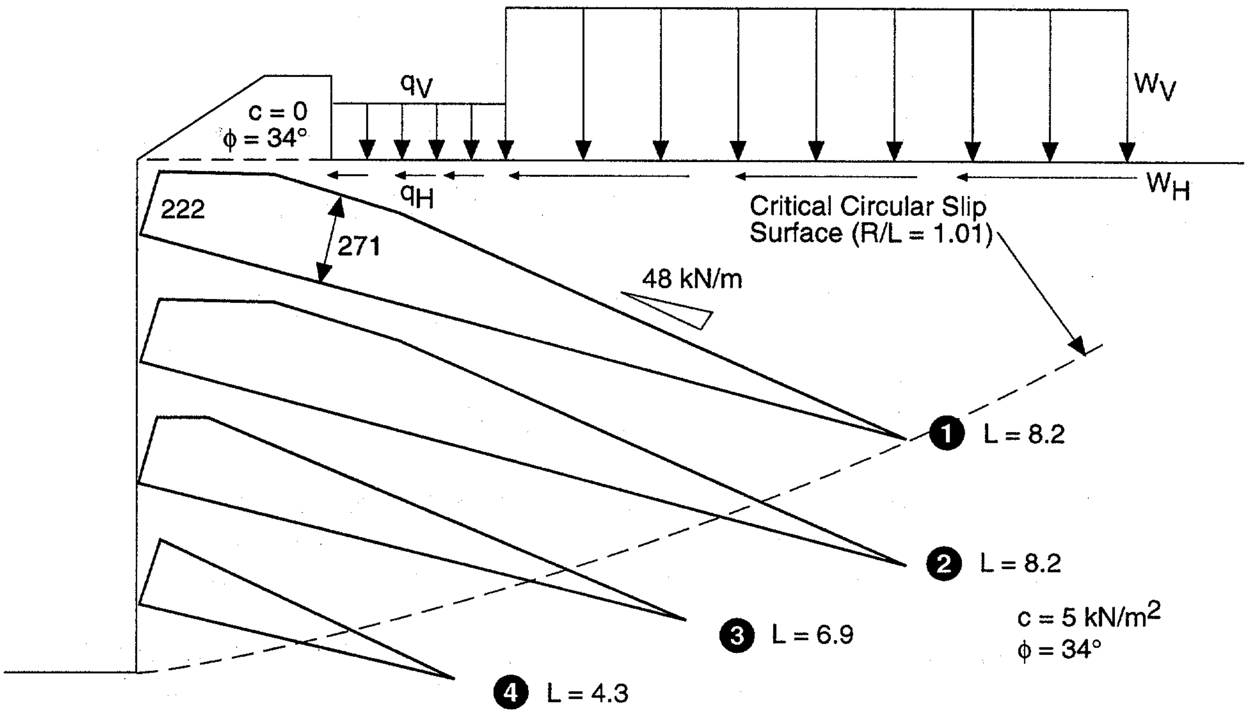
[a] Note that the minimum load factor of 1.0 (table 4.6) is applied, as this is more critical for nail length determination in this case for the seismic loading condition than is the maximum load factor of 1.35.

- The dimensionless nail length is:

$L/H = (8.2\ m)/(5.25\ m)$
$= 1.56$

- Q_D / (L/H) = 1.28/1.56 = 0.82, giving an "R" factor of 0.33.

- Relative nail lengths are calculated from figure 4.11, for the nail head elevations shown on figure 5.28 and an "R" value of 0.33.



Factored Surcharge:

q_{Vmax} = 31.4 kN/m^2

q_{Vmin} = 22.6 kN/m^2

* q_H (0.42g) = 108.2 kN/m^2

q_H (0.32g) = 92.8 kN/m^2

W_{Vmax} = 99.9 kN/m^2

W_{Vmin} = 74.0 kN/m^2

* W_H (0.42g) = 31.1 kN/m^2

W_H (0.32g) = 23.7 kN/m^2

Notes:

All dimensions in meters

All allowable nail loads in kN

Horizontal nail spacing = 1.5 m

❶ Nail number

* Slip surfaces for seismic co-efficient of 0.42g limited to exit depth of 6.66 m (per figures 5.24 and 4.14)

"Max" and "Min" refer to application of maximum and minimum load factors respectively to vertical loads. Horizontal loads have maximum load factors applied.

Figure 5.28 Bridge Abutment Wall Design Example Critical Cross Section Seismic Loading (LRFD)

The design nail strength diagrams are shown graphically on figure 5.28, prepared in accordance with the procedure previously presented on figure 4.3.

Step 6 - Define the Design Soil Strengths

Ultimate Friction Angle, ϕ_U = 34.0°
Ultimate Cohesion, c_U = 5.0 kN/m²

From table 4.8, the soil resistance factors are as follows:

Φ_ϕ = 0.90
Φ_C = 1.00

Design Friction Angle, $\phi = \tan^{-1}(\Phi_\phi \tan(\phi_U)) = \tan^{-1}(0.90[\tan(34.0°)]) = 31.3°$
Design Cohesion, $c = \Phi_C c_U = 1.00(5.0 \text{ kN/m}^2) = 5 \text{ kN/m}^2$

Step 7 - Calculate the Seismic Resistance/Load Ratio

For the trial design cross-section and nail pattern shown on figure 5.28, the above design soil strengths, and design nail strength diagrams, a calculated minimum resistance/load ratio of 1.01 is obtained, based on a computer solution, for the following seismic loading conditions:

- For slip surfaces exiting the reinforced slope within a horizontal distance of 6.66 meters from the top of wall facing (determined from figure 5.24 in accordance with figure 4.14), the applied pseudo-static seismic coefficient for a 0.40 g peak ground acceleration was taken as:

  $A = (1.45 - A_{PK})A_{PK}$
  $A = (1.45 - 0.40)(0.40\text{ g}) = 0.42\text{ g}$

- For more deep-seated slip surfaces, an applied pseudo-static seismic coefficient of A = (0.8)(0.40 g) = 0.32 g (i.e., 80 percent of peak ground acceleration) was chosen, which is consistent with allowable overall displacements of up to 15 mm for the retaining wall system under design seismic loading (in accordance with Section 4.7.2, Seismic Design, Item 3). The 15 mm permanent seismically-induced displacement for the bridge abutment is indicated by the structural engineers as being tolerable.

For the seismic loading condition, the equivalent vertical and horizontal surcharge loads, representative of the pile loads transmitted to the reinforced soil, are computed using the same f_V and f_H factors used for the Strength I Limit State loading condition i.e., f_V value of 0.2 and f_H value of 1.15.

Limiting-equilibrium analysis for seismic loading corresponding to a peak ground acceleration of 0.40g has indicated that the nail tendon size should be increased from No. 25 (for static) to No. 29 (for seismic), and the nail lengths should be increased from 6.9 meters (for static loading) to 8.2 meters (for seismic loading), i.e., about a 20 percent increase in length. For this design example, seismic loading controls the design. Note that table 4.6 requires that the most adverse combination of loads and associated load factors should be applied. Consequently, the following cases were examined:

- Maximum load factors on both vertical and horizontal loads. This condition required the use of No. 29 nail tendons for the seismic loading condition.
- Maximum load factors on horizontal loads and minimum load factors on vertical loads. This condition determined the maximum nail lengths of 8.2 m under seismic loading, and is shown on figure 5.28, together with the critical slip surface giving the lowest calculated resistance to load ratio.

Step 8 - External Stability Check of Nailed Block (Static and Seismic)

For the design nail length of 8.2 m, determined in step 7, the static and seismic bearing capacity checks are performed for the soil nail reinforced block of ground acting as a gravity wall structure (figure 4.12). Pertinent parameters for the analysis include the following:

Soil Unit Weight, γ = 20.0 kN/m^3
Ultimate Soil Friction Angle, ϕ_U = 34°
Ultimate Soil Cohesion, c_U = 5.0 kN/m^2
Ultimate Fill Friction Angle, ϕ_U = 34°
Ultimate Fill Cohesion, c_U = 0 kN/m^2
Seismic Coefficient, A = 0.2g (=0.5A_{PK}, per Section 4.7.2, Seismic Design, Item 4)

In accordance with appendix A, section 11 of the AASHTO [29], the lateral forces due to the active block behind the reinforced soil are computed using the methods of Coulomb and Mononobe and Okabe (including the cohesive component of soil strength). Figure 5.29 shows the static and seismic earth pressure loads, inclined at an angle of 19.4° above horizontal in accordance with the recommendations of figure 4.12, and these are computed to be:

Static Active Earth Load, P_E	=	178.7 kN/m		
Horizontal component	=	$P_E \cos(19.4°)$	=	168.6 kN/m
Vertical component	=	$P_E \sin(19.4°)$	=	59.4 kN/m
Live Earth Pressure Load, P_L	=	26.7 kN/m		
Horizontal component	=	$P_L \cos(19.4°)$	=	25.2 kN/m
Vertical component	=	$P_L \sin(19.4°)$	=	8.9 kN/m
Seismic Earth Load, P_{EQ}	=	108.6 kN/m		
Horizontal component	=	$P_{EQ} \cos(19.4°)$	=	102.4 kN/m
Vertical component	=	$P_{EQ} \sin(19.4°)$	=	36.1 kN/m

For moment equilibrium calculations, the assumed points of application of P_E, P_L and P_{EQ} are at the back of the reinforced block of ground at 0.33H, 0.5H and 0.6H above the base of the soil block, respectively:

Point of Application of P_E = 0.33H = 0.33(8.95 m) = 2.98 m

Point of Application of P_L = 0.5H = 0.50(8.95 m) = 4.48 m

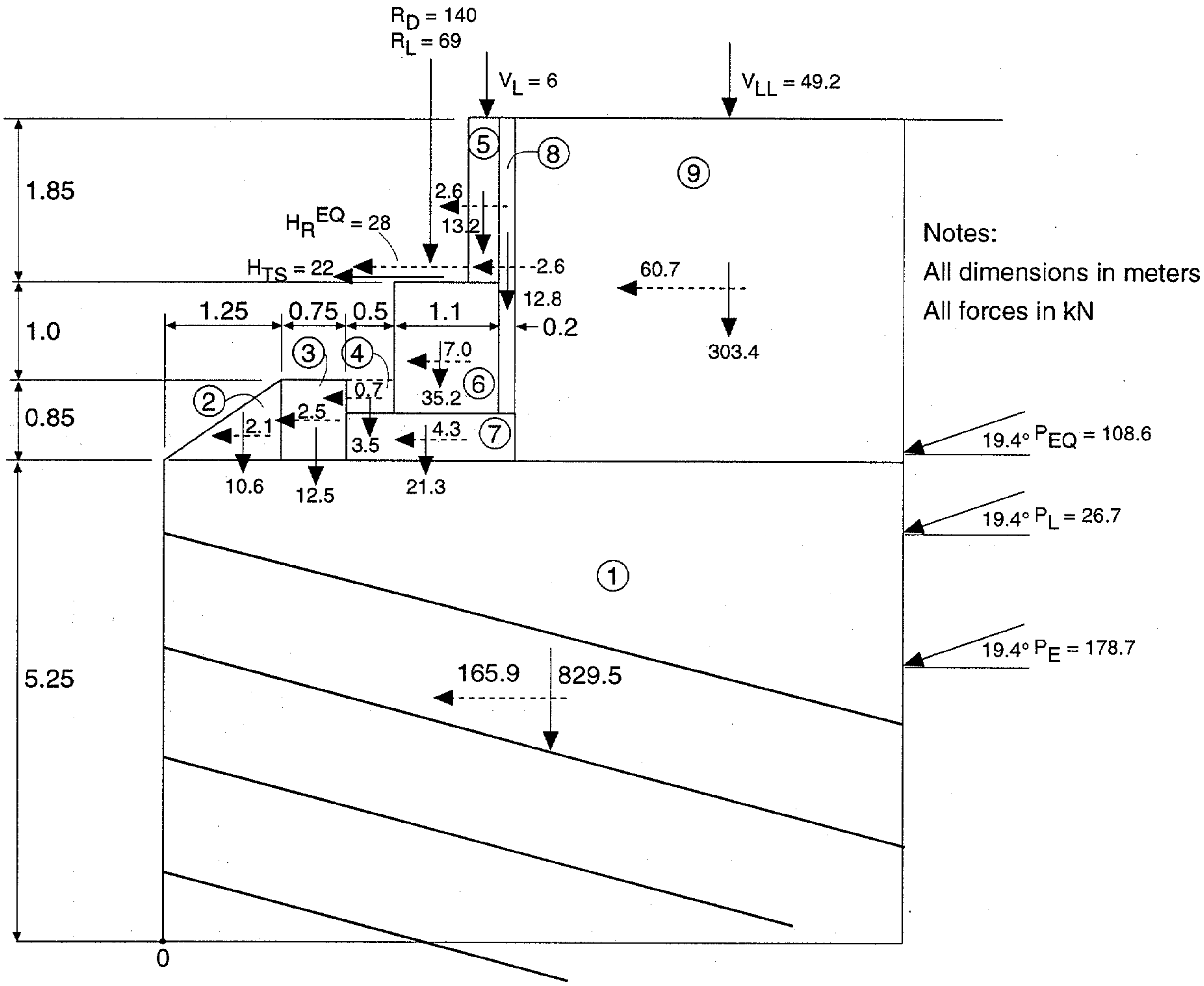
Point of Application of P_{EQ} = 0.6H = 0.60(8.95 m) = 5.37 m

Static Loading (Strength Limit State I)

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Strength I Limit State (table 4.6) is considered. Referring to figure 5.29, forces and moments are computed below:

Vertical Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
$P_E(EH)^a$	$1.5^b(59.4)=89.1$	7.90	703.9
$P_L(LL)^a$	$1.75^c(8.9)=15.6$	7.90	123.0
$R_D(DC)^{a,g}$	$1.25^d(42)=52.5$	2.90	152.3
$R_L(LL)^{a,g}$	$1.75^c(20.7)=36.2$	2.90	105.1
$V_L(LL)^{a,g}$	$1.75^c(1.8)=3.2$	3.45	10.9
$V_{LL}(LL)^a$	$1.75^c(49.2)=86.1$	5.85	503.7
$1\ (EV)^a$	$1.35^e(829.5)=1119.8$	3.95	4423.3
$2\ (EV)^a$	$1.35^e(10.6)=14.3$	0.83	11.9
$3\ (EV)^a$	$1.35^e(12.5)=16.9$	1.62	27.3
$4(EV)^{a,g}$	$1.35^e(1.1)=1.4$	2.25	3.2
$5(DC)^{a,g}$	$1.25^d(4.0)=5.0$	3.45	17.1
$6(DC)^{a,g}$	$1.25^d(10.6)=13.2$	3.05	40.3
$7(DC)^{a,g}$	$1.25^d(6.4)=8.0$	2.90	23.2
$8(EV)^{a,g}$	$1.35^e(3.8)=5.2$	3.70	19.2
$9(EV)^a$	$1.35^e(303.4)=409.6$	5.85	2396.1
Sum	1876.0 (N)	N/A	8560.3



Load	Vertical Force (kN)	Horizontal Movement Arm (m)	Horizontal Force (kN)	Vertical Movement Arm (m)
P_E	59.4	7.9	168.6	2.98
P_L	8.9	7.9	25.2	4.48
P_{EQ}	36.1	7.9	102.4	5.37
R_D	42.0	2.9	–	–
R_L	20.7	2.9	–	–
V_L	1.8	3.45	–	–
V_{LL}	49.2	5.85	–	–
1	829.5	3.95	165.9	2.62
2	10.6	0.83	2.1	5.53
3	12.5	1.62	2.5	5.67
4	1.1	2.25	0.7	5.92
5	4.0	3.45	2.6	8.02
6	10.6	3.05	7.0	6.42
7	6.4	2.90	4.3	5.50
8	3.8	3.70	2.6	7.35
9	303.4	5.85	60.7	7.10
H_{TS}	–	–	22	7.10
H_R^{EQ}	–	–	28	7.10

Figure 5.29 Bridge Abutment Wall Design Example Bearing Capacity Stability Check (LRFD)

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
P_E (EH)[a]	$1.5^e(168.6)=252.9$	2.98	-753.6
P_L(LL)[a]	$1.75^c(25.2)=44.1$	4.48	-197.6
H_{TS}(SH)[a, g]	$0.5^f(22)=11$	7.1	-78.1
Sum	308.0 (H)	N/A	-1029.3

[a] Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
[b] Load factor for lateral earth pressure (EH)
[c] Load factor for live load (LL)
[d] Load factor for structural dead load (DC)
[e] Load factor for earthfill dead load (EV)
[f] Load factor for temperature/shrinkage (TU/SH)
[g] For the nail reinforced block shown on figure 5.29, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

$$N = 1876.0 \text{ kN}$$
$$X_O = (8560.3 \text{ kN-m/m} - 1029.3 \text{ kN-m/m})/(1876.0 \text{ kN/m}) = 4.01 \text{ m}$$

Since $2X_o > B$

$$q_{MAX} = N/(B) = (1876.0 \text{ kN/m})/(7.90 \text{ m}) = 237 \text{ kN/m}^2$$

In accordance with section 10 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] (and by reference to equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30]), the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, B′ = B (see above) = 7.90 m. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$$q_{ULT} = (5.0 \text{ kN/m}^2)(42.0)(1.18)(0.74) + 0.5(20.0 \text{ kN/m}^3)(7.90 \text{ m})(41.0)(0.89)(0.63)$$
$$q_{ULT} = 1{,}999 \text{ kN/m}^2$$

It should be noted that unfactored loads are used in computing the load inclination factor "i." For a bearing capacity resistance factor Φ_q of 0.45 (section 10.5.4 of AASHTO [29], the design bearing capacity is:

$$\Phi_q q_{ULT} = 0.45(1{,}999 \text{ kN/m}^2) = 890 \text{ kN/m}^2$$

Since $q_{MAX} < \Phi_q q_{ULT}$, the design bearing capacity is adequate. (OK)

As noted above, the eccentricity of the vertical resultant force is negative so the eccentricity check is satisfied.

The above analyses may be repeated for Strength Limit State I loading, but with minimum load factors of 0.90 and 1.00 applied to structural component dead loads (DC) and earthfill dead loads (EV) respectively (see table 4.6). For this loading condition q_{MAX} = 184 kN/m^2, $\Phi_q q_{ULT}$ = 903 kN/m^2, and the eccentricity of the vertical resultant force is 0.03 m, which is less than B/4 = 1.98 m.

Static Loading (Strength Limit State IV)

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Strength IV Limit State (table 4.6) is considered. Referring to figure 5.29, forces and moments are computed below:

Vertical Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
P_E(EH)[a]	1.5[b](59.4)=89.1	7.90	703.9
R_D(DC)[a, g]	1.5[d](42)=63.0	2.90	182.7
1 (EV)[a]	1.35[e](829.5)=1119.8	3.95	4423.3
2 (EV)[a]	1.35[e](10.6)=14.3	0.83	11.9
3 (EV)[a]	1.35[e](12.5)=16.9	1.62	27.3
4(EV)[a, g]	1.35[e](1.1)=1.4	2.25	3.2
5(DC)[a, g]	1.5[d](4.0)=5.9	3.45	20.5
6(DC)[a, g]	1.5[d](10.6)=15.8	3.05	48.3
7(DC)[a, g]	1.5[d](6.4)=9.6	2.90	27.8
8(EV)[a, g]	1.35[e](3.8)=5.2	3.70	19.2
9(EV)[a]	1.35[e](303.4)=409.6	5.85	2396.1
Sum	1750.7 (N)	N/A	7864.2

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
P_E (EH)[a]	$1.5^e(168.6)=252.9$	2.98	-753.6
H_{TS}(SH)[a, g]	$0.5^f(22)=11$	7.1	-78.1
Sum	263.9 (H)	N/A	-831.7

[a] Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
[b] Load factor for lateral earth pressure (EH)
[c] Load factor for live load (LL)
[d] Load factor for structural dead load (DC)
[e] Load factor for earthfill dead load (EV)
[f] Load factor for temperature/shrinkage (TU/SH)
[g] For the nail reinforced block shown on figure 5.29, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

$N = 1750.7$ kN

$X_O = (7864.2 \text{ kN-m/m} - 831.7 \text{ kN-m/m})/(1750.7 \text{ kN/m}) = 4.02$ m

Since $2X_o > B$

$q_{MAX} = N/(B) = (1750.7 \text{ kN/m})/(7.90 \text{ m}) = 222 \text{ kN/m}^2$

In accordance with section 10 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] (and by reference to equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30]), the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B$ (see above) = 7.90 m. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$q_{ULT} = (5.0 \text{ kN/m}^2)(42.0)(1.18)(0.75) + 0.5(20.0 \text{ kN/m}^3)(7.90 \text{ m})(41.0)(0.89)(0.65)$

$q_{ULT} = 2{,}060 \text{ kN/m}^2$

It should be noted that unfactored loads are used in computing the load inclination factor "i." For a bearing capacity resistance factor Φ_q of 0.45 (section 10.5.4 of AASHTO [29], the design bearing capacity is:

$$\Phi_q q_{ULT} = 0.45(2{,}060 \text{ kN/m}^2) = 927 \text{ kN/m}^2$$

Since $q_{MAX} < \Phi_q q_{ULT}$, the design bearing capacity is adequate. (OK)

As noted above, the eccentricity of the vertical resultant force is negative so the eccentricity check is satisfied.

The above analyses may be repeated for Strength Limit State IV loading, but with a minimum load factor of 1.00 applied to earthfill dead loads (EV) (see table 4.6). For this loading condition $q_{MAX} = 172$ kN/m^2, $\Phi_q q_{ULT} = 926$ kN/m^2, and eccentricity of the vertical resultant force is 0.05 m, which is less than B/4 = 1.98 m.

Seismic Loading

In accordance with the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], the Extreme Event I Limit State (table 4.6) is considered. Referring to figure 5.29, forces and moments are computed below:

Vertical Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
P_E(EH)[a]	1.5[b](59.4)=89.1	7.90	703.9
P_{EQ}(EQ)[a]	1.0[h](36.1)=36.1	7.90	285.0
R_D(DC)[a, g]	1.25[d](42)=52.5	2.90	152.3
1 (EV)[a]	1.35[e](829.5)=1119.8	3.95	4423.3
2 (EV)[a]	1.35[e](10.6)=14.3	0.83	11.9
3 (EV)[a]	1.35[e](12.5)=16.9	1.62	27.3
4(EV)[a, g]	1.35[e](1.1)=1.4	2.25	3.2
5(DC)[a, g]	1.25[d](4.0)=5.0	3.45	17.1
6(DC)[a, g]	1.25[d](10.6)=13.2	3.05	40.3
7(DC)[a, g]	1.25[d](6.4)=8.0	2.90	23.2
8(EV)[a, g]	1.35[e](3.8)=5.2	3.70	19.2
9(EV)[a]	1.35[e](303.4)=409.6	5.85	2396.1
Sum	1771.0 (N)	N/A	8102.6

Horizontal Loads and Moments

Item	Factored Force (kN/m)	Moment Arm (m)	Factored Moment About Point O (kN-m/m)
P_E (EH)[a]	$1.5^e(168.6)=252.9$	2.98	-753.6
P_{EQ}(EQ)[a]	$1.0^h(102.4)=102.4$	5.37	-550.1
H_R^{EQ}(EQ)[a, g]	$1.0^h(28.0)=28.0$	7.10	-198.8
1 (EQ)[a]	$1.0^h(165.9)=165.9$	2.62	-434.7
2 (EQ)[a]	$1.0^h(2.1)=2.1$	5.53	-11.7
3 (EQ)[a]	$1.0^h(2.5)=2.5$	5.67	-14.2
4 (EQ)[a, g]	$1.0^h(0.7)=0.7$	5.92	-4.1
5 (EQ)[a, g]	$1.0^h(2.6)=2.6$	8.02	-21.2
6 (EQ)[a, g]	$1.0^h(7.0)=7.0$	6.42	-45.2
7 (EQ)[a, g]	$1.0^h(4.3)=4.3$	5.50	-23.4
8 (EQ)[a, g]	$1.0^h(2.6)=2.6$	7.35	-18.8
9 (EQ)[a]	$1.0^h(60.7)=60.7$	7.1	-431.0
Sum	631.8 (H)	N/A	-2506.8

[a] Notations used in the AASHTO LRFD Bridge Design Specifications, 1st Edition [29].
[b] Load factor for lateral earth pressure (EH)
[c] Load factor for live load (LL)
[d] Load factor for structural dead load (DC)
[e] Load factor for earthfill dead load (EV)
[f] Load factor for temperature/shrinkage (TU/SH)
[g] For the nail reinforced block shown on figure 5.29, it is estimated that 30% of the pile head vertical load and 100% of the pile head lateral load are transferred to the soil by pile-soil interaction (see figure 5.14)
[h] Load factor for earthquake load (EQ)

Stability Criteria

Referring to figure 4.12, the maximum applied bearing pressure and the location of the resultant force are checked as follows:

$$N = 1771.0 \text{ kN}$$
$$X_O = (8102.6 \text{ kN-m/m} - 2506.8 \text{ kN-m/m})/(1771.0 \text{ kN/m}) = 3.16 \text{ m}$$
$$q_{MAX} = N/(2X_o) = (1771.0 \text{ kN/m})/[2(3.16 \text{ m})] = 280 \text{ kN/m}^2$$

In accordance with section 10 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29] (and by reference to equation 4.4.7.1.1-1, section 4, division I of the Standard Specifications for Highway Bridges, 15th Edition [30]), the ultimate bearing capacity is given by:

$$q_{ULT} = c_U N_c s_c i_c + 0.5\gamma B' N_\gamma s_\gamma i_\gamma$$

The effective width, $B' = B - 2(e) = 2X_O$ (see above) = 2(3.16 m) = 6.32 m to account for eccentric loading. The factors "s" and "i" account for the shape of the loaded area and the inclination of the applied loading, respectively, per AASHTO [30]. The factors N_c and N_γ are the conventional bearing capacity factors. The ultimate bearing capacity is computed to be:

$$q_{ULT} = (5.0\ kN/m^2)(42.0)(1.15)(0.38) + 0.5(20.0\ kN/m^3)(6.32\ m)(41.0)(0.92)(0.24)$$
$$q_{ULT} = 664\ kN/m^2$$

It should be noted that unfactored loads are used in computing the load inclination factor "i." For a bearing capacity resistance factor Φ_q of 1.0 (section 1.3.2.1 of AASHTO [29]), the design bearing capacity is:

$$\Phi_q q_{ULT} = 1.0(664\ kN/m^2) = 664\ kN/m^2$$

Since $q_{MAX} < \Phi_q q_{ULT}$, the design bearing capacity is adequate. (OK)

Check that the location of the resultant is within the middle half of the block of reinforced soil, per section 10.6.3.1.5 of AASHTO [29].

$$e = B/2 - X_o = (7.90\ m)/2 - 3.16\ m = 0.79\ m$$
$$B/4 = (7.90\ m)/4 = 1.98\ m$$

Since $e < B/4$, the resultant is within the middle half. (OK)

The above analyses may be repeated for Extreme Event I Limit State loading, but with minimum load factors of 0.90 and 1.00 applied to structural component dead loads (DC) and earthfill dead loads (EV) respectively (see table 4.6). For this loading condition $q_{MAX} = 241\ kN/m^2$, $\Phi_q q_{ULT} = 591\ kN/m^2$, and eccentricity e = 1.16 m.

Overall Stability

For the uniform nail length pattern selected (see figure 5.30), the deep ground water table, and the overall geometry of the slope, deep seated failure surfaces passing beneath the toe of the wall will not be critical.

Step 9 - Check the Upper Cantilever

In accordance with appendix A, section 11 of the AASHTO LRFD Bridge Design Specifications, 1st Edition [29], equation A11.1.1.1-2, the approach developed by Mononobe and Okabe for a free-standing retaining structure is used to develop the active seismic loading on the upper cantilever. For a soil friction angle of 34°, zero cohesion (ignore

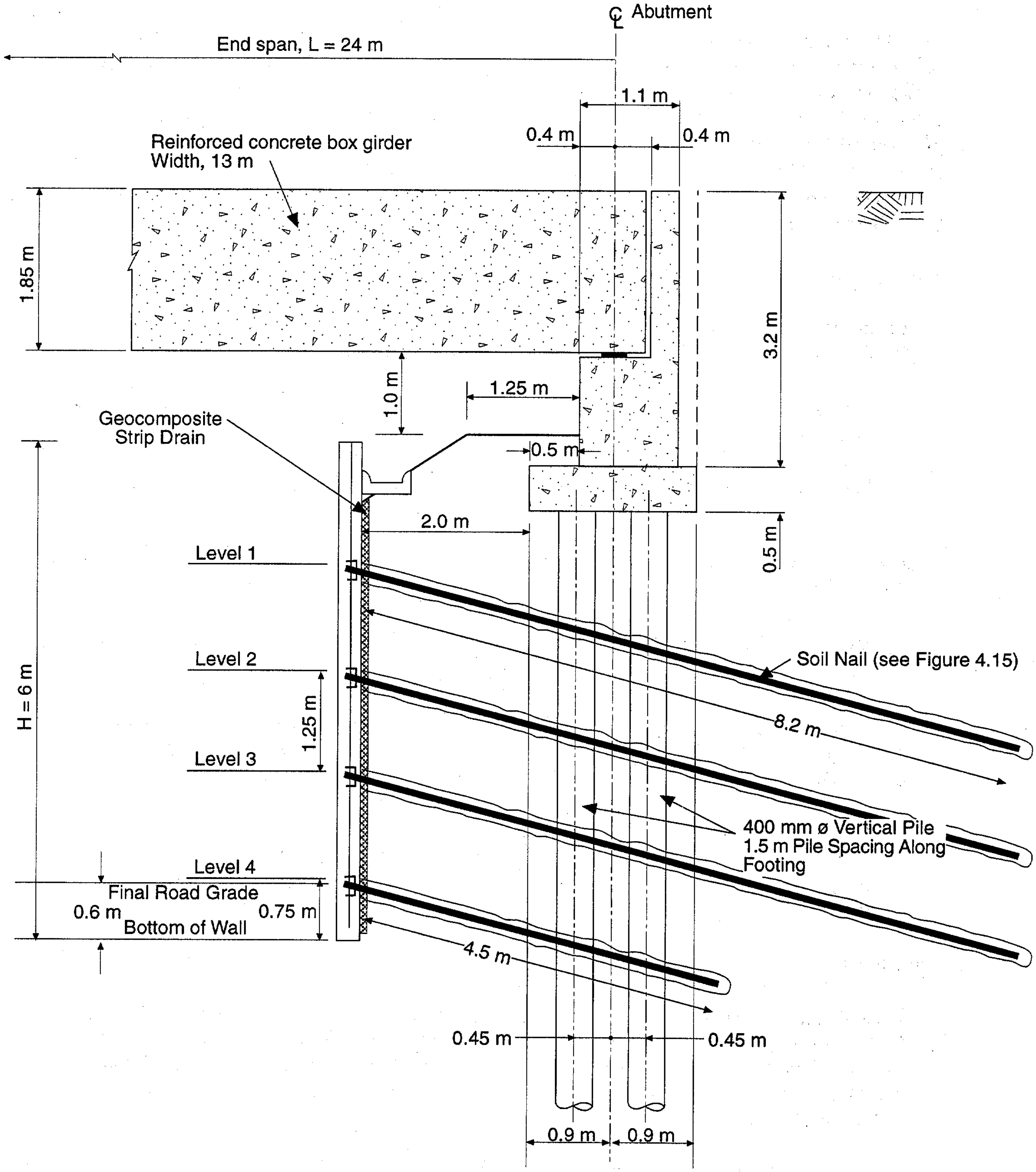


Figure 5.30 Bridge Abutment Wall Final Design Section (LRFD)

it), a soil-to-wall interface friction angle of (2/3)34° = 22°, a vertical wall, a soil profile behind the wall as shown on figure 5.23, and a horizontal pseudo-static seismic coefficient of 0.32 g (i.e., $0.8A_{PK}$), the combined static and dynamic active earth pressure load P_{AE} determined from a Mononobe-Okabe type computer solution is 9.14 kN/m length of wall. This can be considered to consist of a static earth pressure load of 2.95 kN/m (see step 9 for the static loading condition) and a dynamic earth pressure load of 6.19 kN/m. The load components normal to the wall are (2.95)cos(22°) = 2.74 kN/m (static) and (6.19)cos(22°) = 5.74 kN/m (dynamic).

Shear Check

From force equilibrium, compute the factored one-way unit service shear force at the level of the upper row of nails (conservative), as indicated on figure 4.13:

$$v_{STATIC} = = 2.74 \text{ kN/m}$$
$$v_{DYNAMIC} = = 5.74 \text{ kN/m}$$
$$\Gamma_{EH} = 1.50; \quad \Gamma_{EQ} = 1.00 \quad \text{(table 4.6)}$$
$$\Gamma_{EH} v_{STATIC} + \Gamma_{EQ} v_{DYNAMIC} = 1.50(2.74 \text{ kN/m}) + 1.00(5.74 \text{ kN/m})$$
$$= 9.85 \text{ kN/m}$$

Compute the nominal one-way unit shear strength of the facing based on equation 5.8.3.3-3 of AASHTO [29]:

$$V_{NS} = 0.166\sqrt{f'_c(\text{MPa})}(d)$$
$$V_{NS} = 0.166\sqrt{28 \text{ MPa}}\,(0.1 \text{ m}) = 87.8 \text{ kN/m}$$

From table 4.7 and section 1.3.2.1 of AASHTO [29], the facing shear resistance factor, Φ_F, equals 1.0. Therefore, the design one-way unit shear strength of the facing is computed to be:

$$V = \Phi_F V_{NS} = 1.00(87.8 \text{ kN/m}) = 87.8 \text{ kN/m}$$

Since $\Gamma_{EH} v_{STATIC} + \Gamma_{EQ} v_{DYNAMIC} = 9.85$ kN/m < V, the design for shear is adequate. (OK)

Flexure Check

From moment equilibrium, compute the factored one-way unit service moment at the level of the upper row of nails (conservative), as indicated on figure 4.13. Note that for moment determination, the point of application of the static force is taken as 0.33H above the base of the cantilever. The dynamic force is assumed to occur at 0.6H above the base of the cantilever:

$$m_{STATIC} = (0.33)(v_{STATIC})(H) = (0.33)(2.74 \text{ kN/m})(0.75 \text{ m})$$
$$= 0.678 \text{ kN-m/m}$$
$$m_{DYNAMIC} = (0.60)(v_{DYNAMIC})(H) = (0.60)(5.74 \text{ kN/m})(0.75 \text{ m})$$

$= 2.583$ kN-m/m

$\Gamma_{EH} = 1.50; \Gamma_{EQ} = 1.00$ (table 4.6)

$\Gamma_{EH} m_{STATIC} + \Gamma_{EQ} m_{DYNAMIC} = 1.50(0.678 \text{ kN-m/m}) + 1.00(2.583 \text{ kN-m/m})$
$= 3.6$ kN-m/m

Compute the nominal unit moment resistance of the facing. Using the procedure demonstrated in appendix F, the nominal unit moment resistance in the vertical direction over the nail locations $m_{V,NEG}$ is computed to be 17.4 kN-m/m. Per section 1.3.2.1 of AASHTO [29], the resistance factor Φ_F for facing flexure is 1.00. Therefore, the design one-way unit moment resistance for the upper cantilever is:

$$M = \Phi_F\, m_{V,NEG} = (1.00)(17.4 \text{ kN-m/m}) = 17.4 \text{ kN-m/m}$$

Since $\Gamma_{EH} m_{STATIC} + \Gamma_{EQ} m_{DYNAMIC} = 3.6$ kN-m/m $< M$, the design for flexure is adequate. (OK)

Step 10 - Check the Facing Reinforcement Details

The facing reinforcement details, being independent of type of loading, have been previously considered for the static loading condition and need not be repeated for the seismic loading condition.

Step 11 - Serviceability Checks

The serviceability checks are not applicable to seismic loading conditions because of the extreme nature of the limit state.

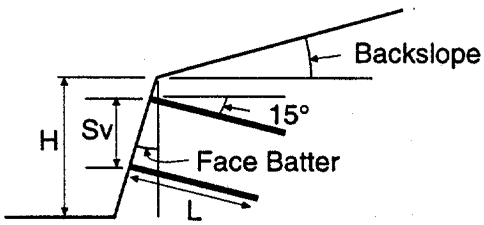
Design Section

Based on the above analyses, the final design cross-section is shown on figure 5.30. It should be noted that, to simplify construction, the chosen final plan nail lengths are different to the nail length distributions used in the analyses. The upper three rows of nails are of uniform length equal to the maximum nail length indicated by the above analyses (i.e., 8.2 m). Overall stability analysis has confirmed that the bottom row of nails can be installed with shorter lengths of about 4.5 meters.

5.3 Simplified Design Charts for Preliminary Design of Cutslope Walls

5.3.1 Design Variables

For simple cutslope geometric configurations in relatively homogeneous soil conditions, the design charts shown on figures 5.31 through 5.34 can be used to obtain a preliminary soil nail wall design suitable for scoping or cost estimating purposes. The charts were developed using the design methodology presented in chapter 4. The design charts have been prepared for the



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$

$$= \Phi_c c_u/(\Gamma_w \gamma H)\text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$

$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{ (LRFD)}$$

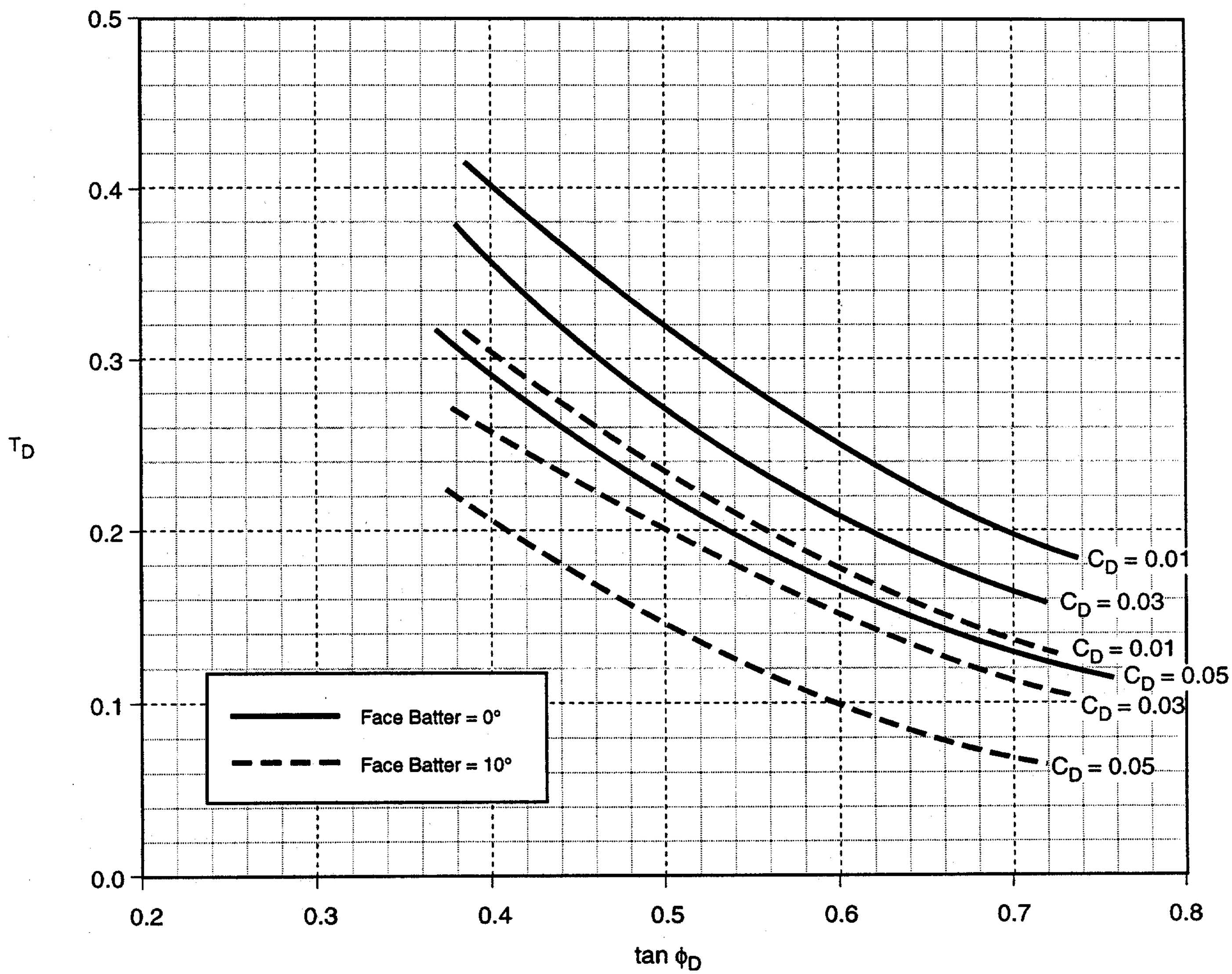
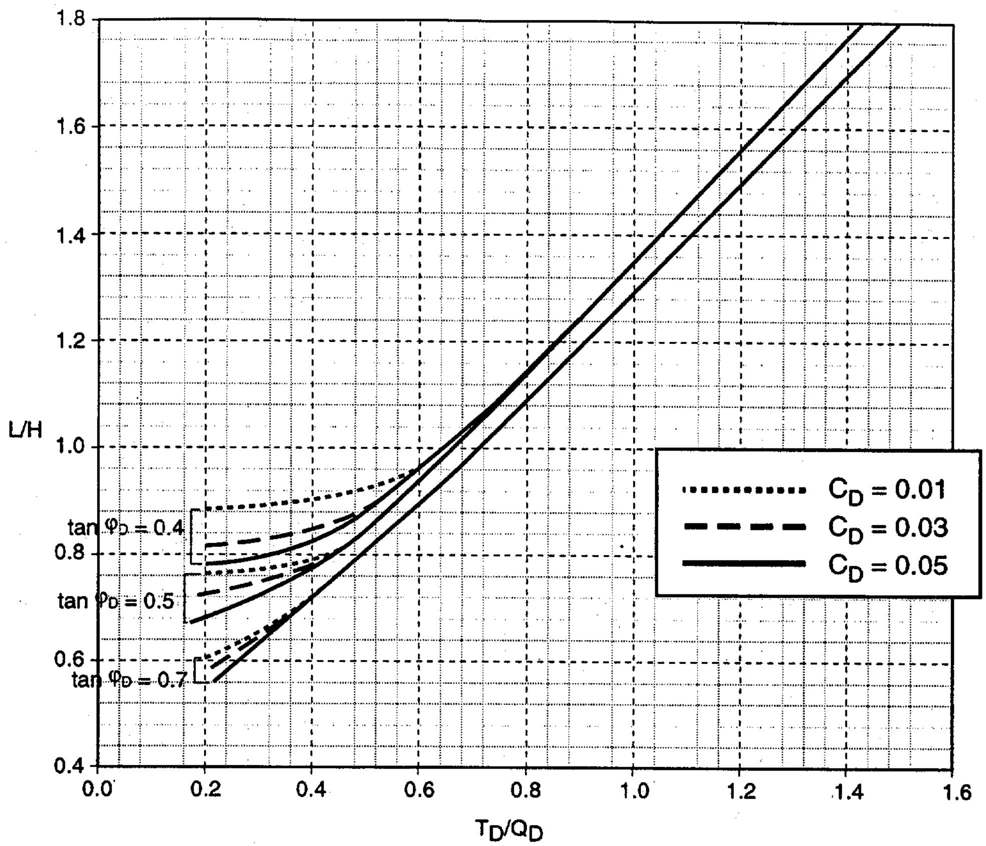
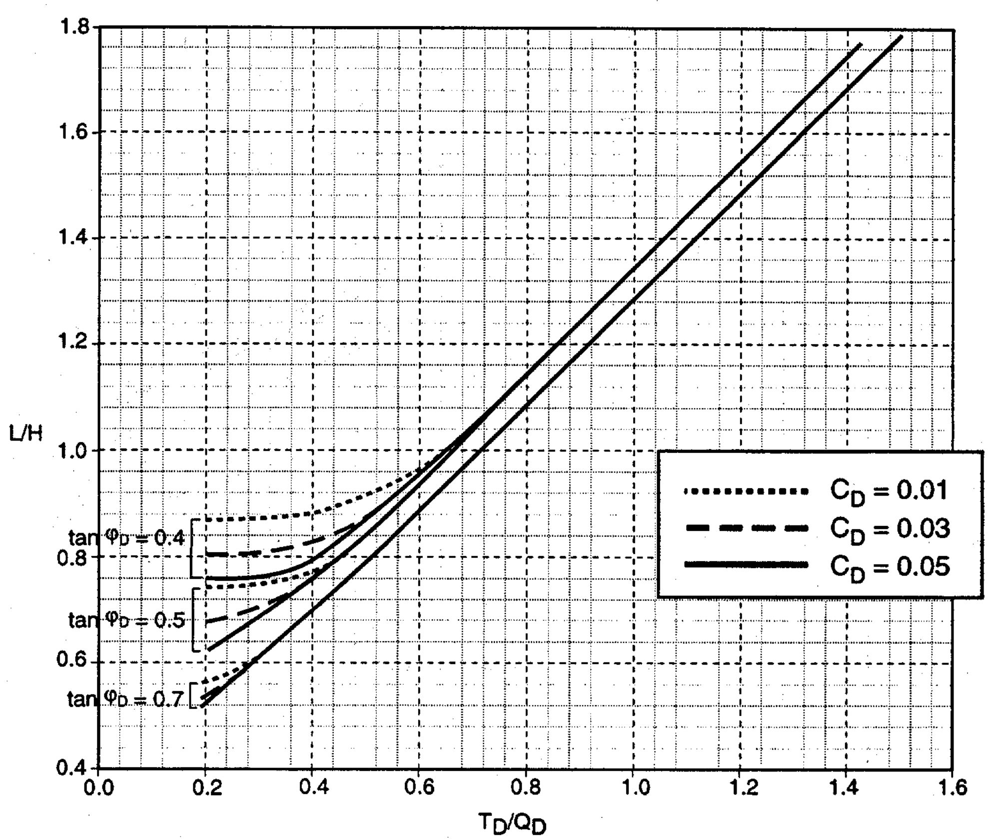


Figure 5.31A Preliminary Design Chart 1A
Backslope = 0°

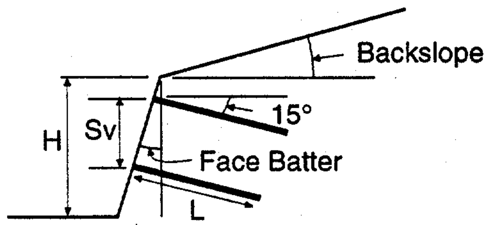


Backslope = 0° Face Batter = 0° (Chart 1B)



Backslope = 0° Face Batter = 10° (Chart 1C)

Figure 5.31B and C Preliminary Design Charts 1B and 1C



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$

$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$

$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$

$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{ (LRFD)}$$

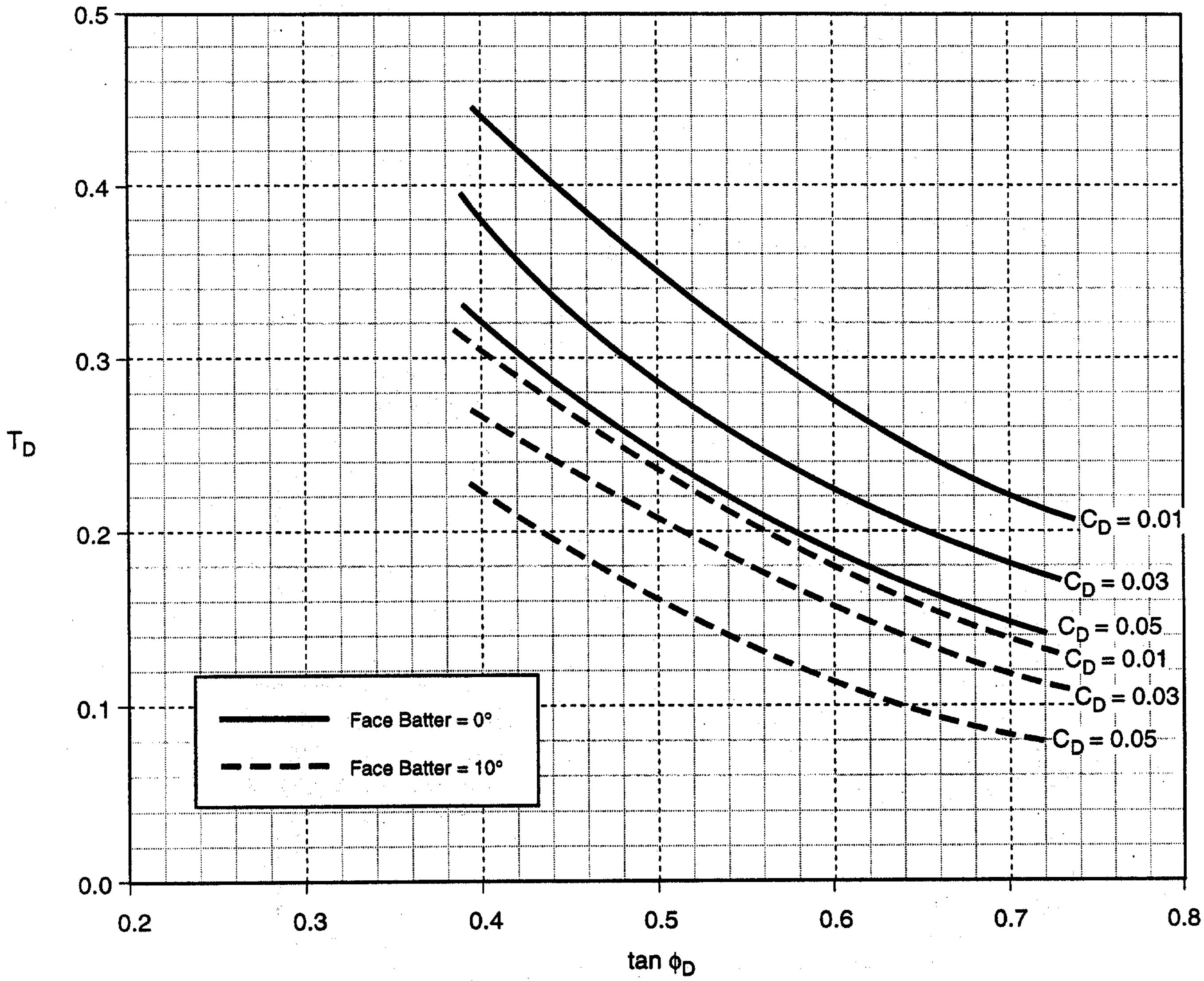
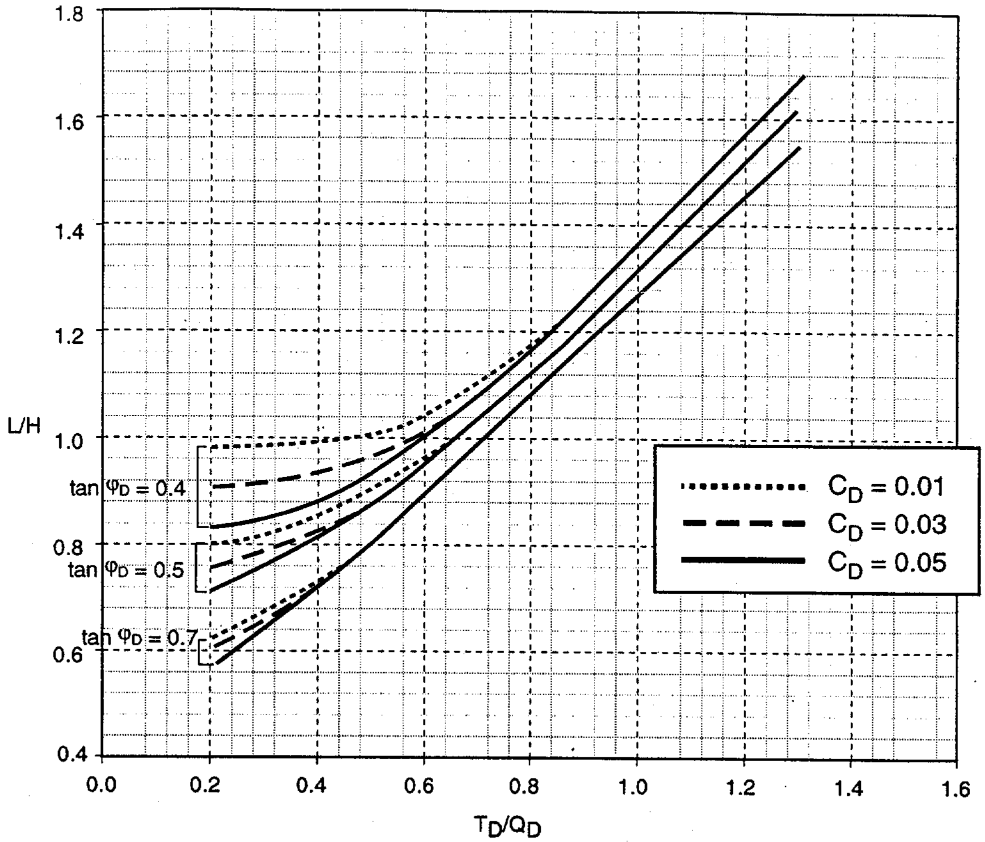
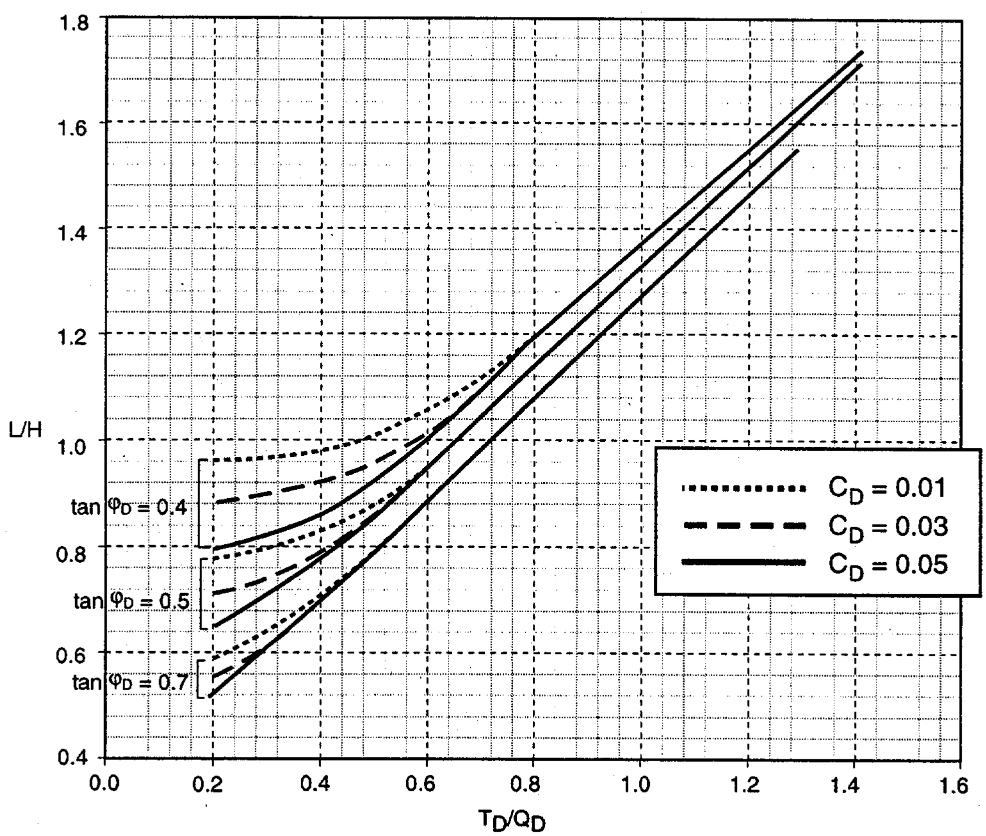


Figure 5.32A Preliminary Design Chart 2A
Backslope = 10°

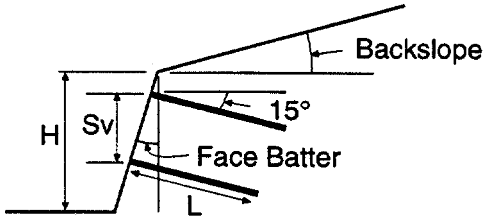


Backslope = 10° Face Batter = 0° (Chart 2B)



Backslope = 10° Face Batter = 10° (Chart 2C)

Figure 5.32B and C Preliminary Design Charts 2B and 2C



$C_D = c_u/(F_c \gamma H)$ (SLD)

$= \Phi_c c_u/(\Gamma_w \gamma H)$ (LRFD)

$Q_D = \alpha_Q Q_u/(\gamma S_V S_H)$ (SLD)

$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H)$ (LRFD)

$\tan \phi_D = \dfrac{\tan \phi_u}{F\phi}$ (SLD)

$= \Phi_\phi \tan\phi_u$ (LRFD)

$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H)$ (SLD)

$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H)$ (LRFD)

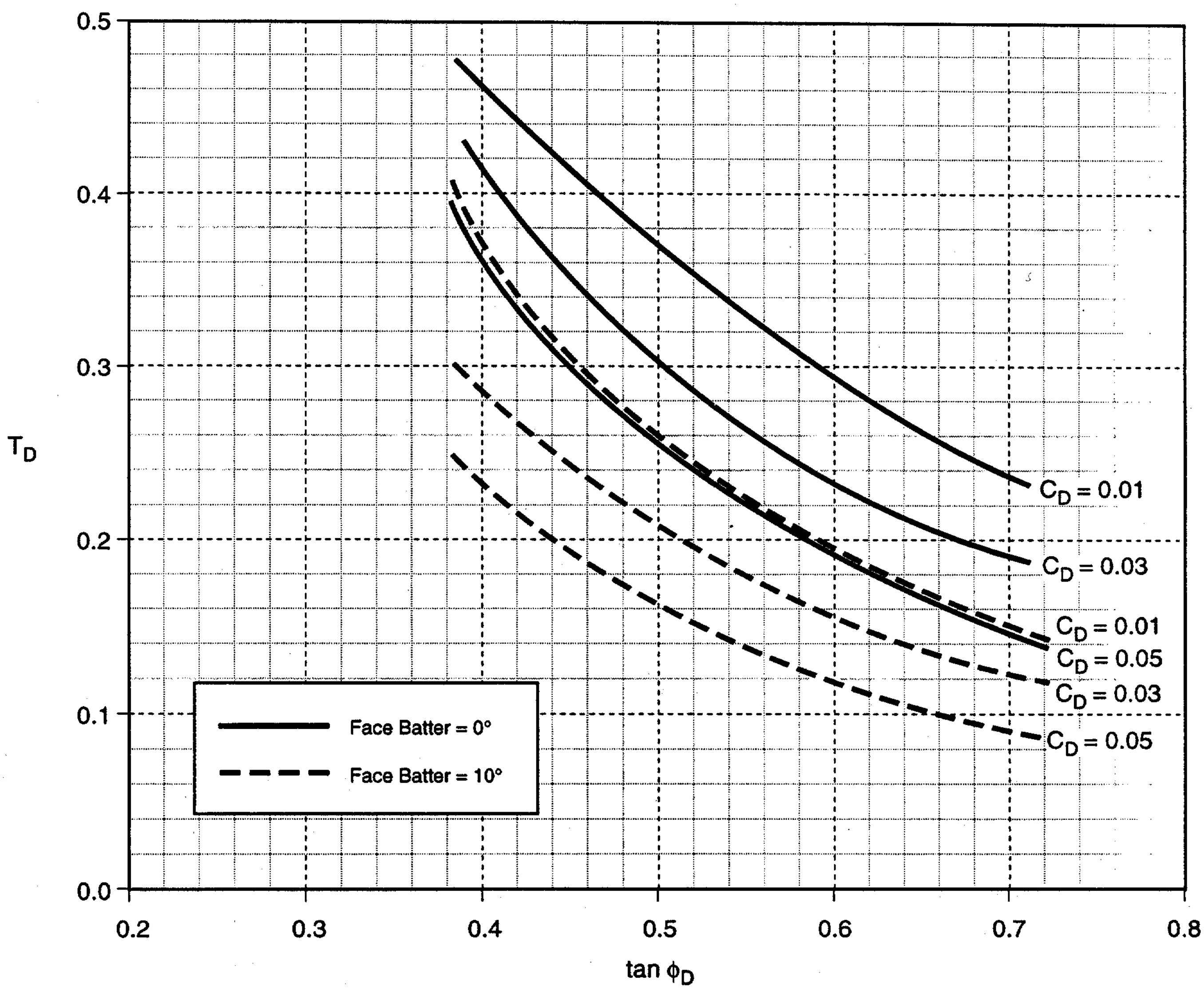
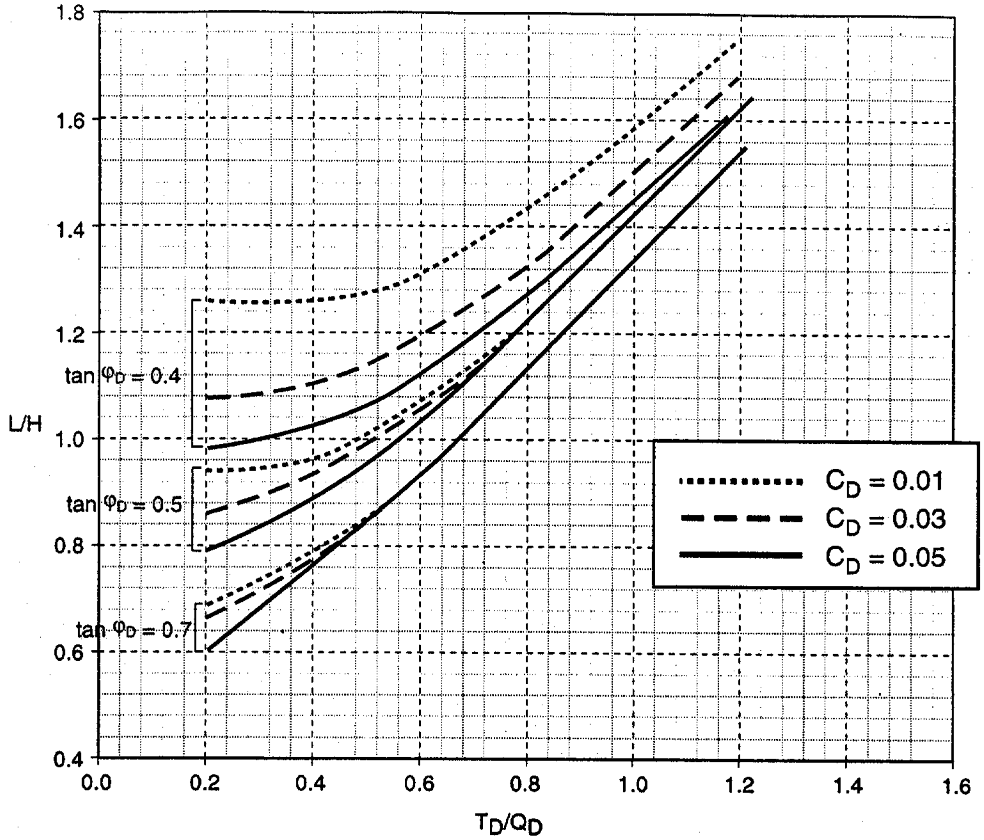
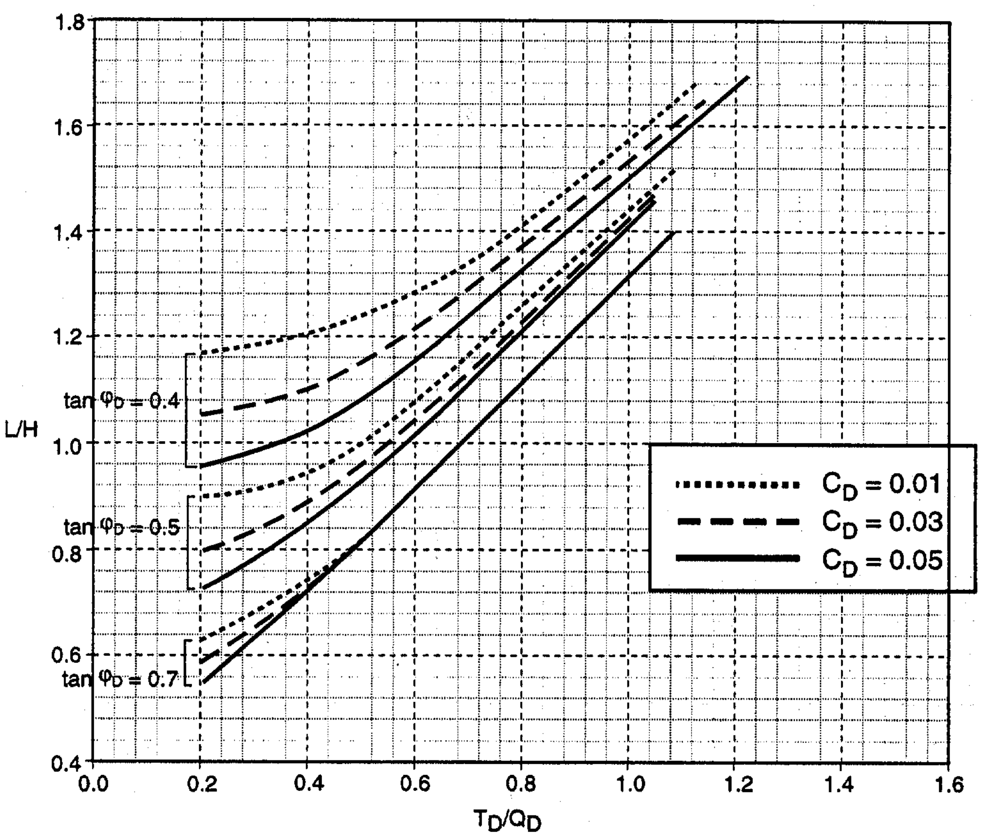


Figure 5.33A Preliminary Design Chart 3A
Backslope = 20°

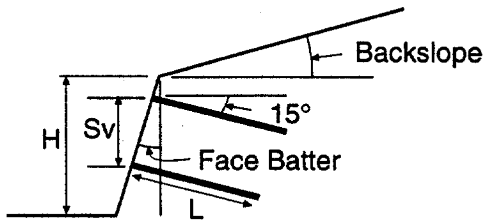


Backslope = 20° Face Batter = 0° (Chart 3B)



Backslope = 20° Face Batter = 10° (Chart 3C)

Figure 5.33B and C Preliminary Design Charts 3B and 3C



$$C_D = c_u/(F_c \gamma H) \quad \text{(SLD)}$$
$$= \Phi_c c_u/(\Gamma_w \gamma H) \text{(LRFD)}$$

$$Q_D = \alpha_Q Q_u/(\gamma S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_Q Q_u/(\Gamma_w \gamma S_V S_H) \quad \text{(LRFD)}$$

$$\tan \phi_D = \frac{\tan \phi_u}{F\phi} \quad \text{(SLD)}$$
$$= \Phi_\phi \tan\phi_u \quad \text{(LRFD)}$$

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad \text{(SLD)}$$
$$= \Phi_N T_{NN}/(\Gamma_w \gamma H S_V S_H) \text{(LRFD)}$$

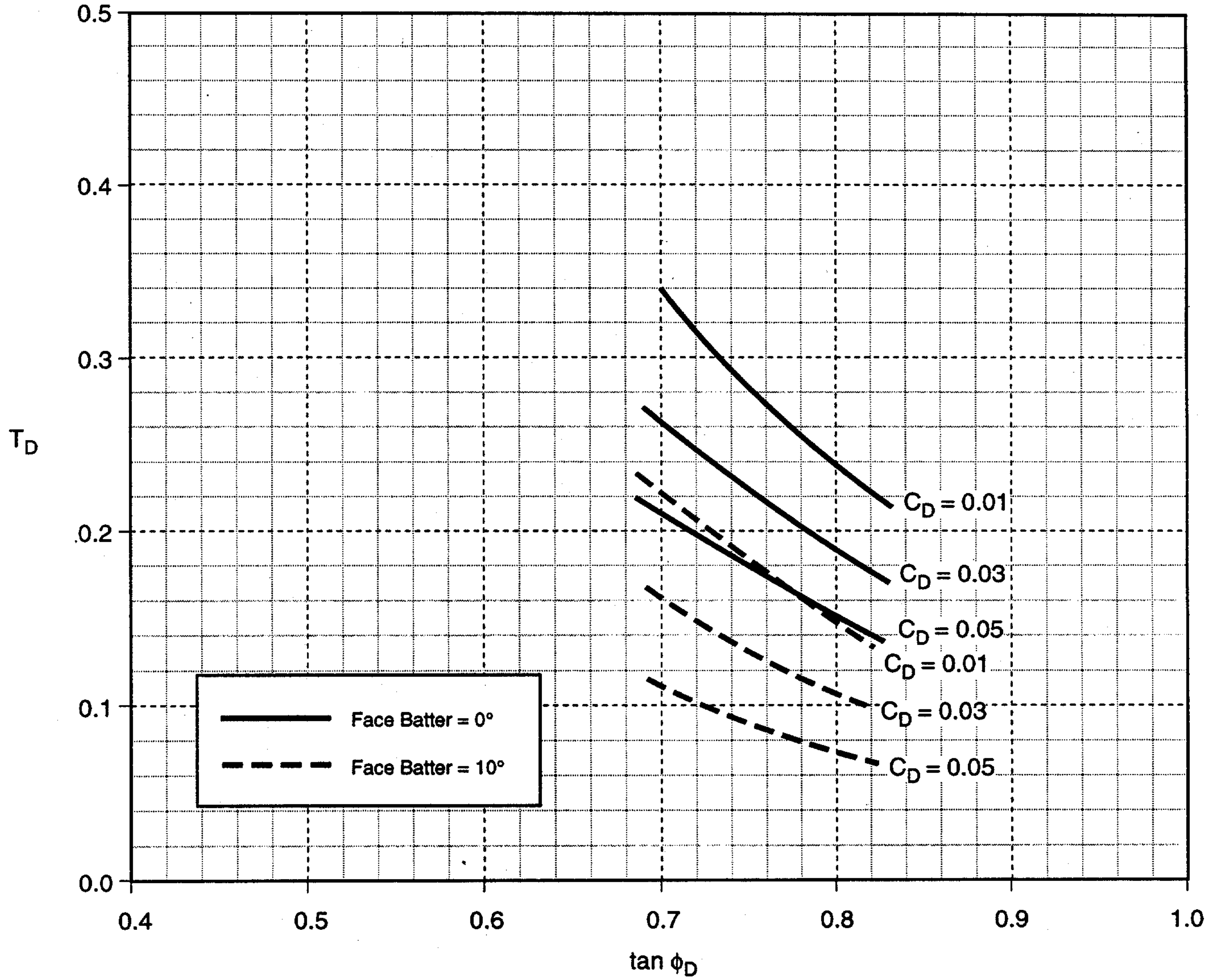
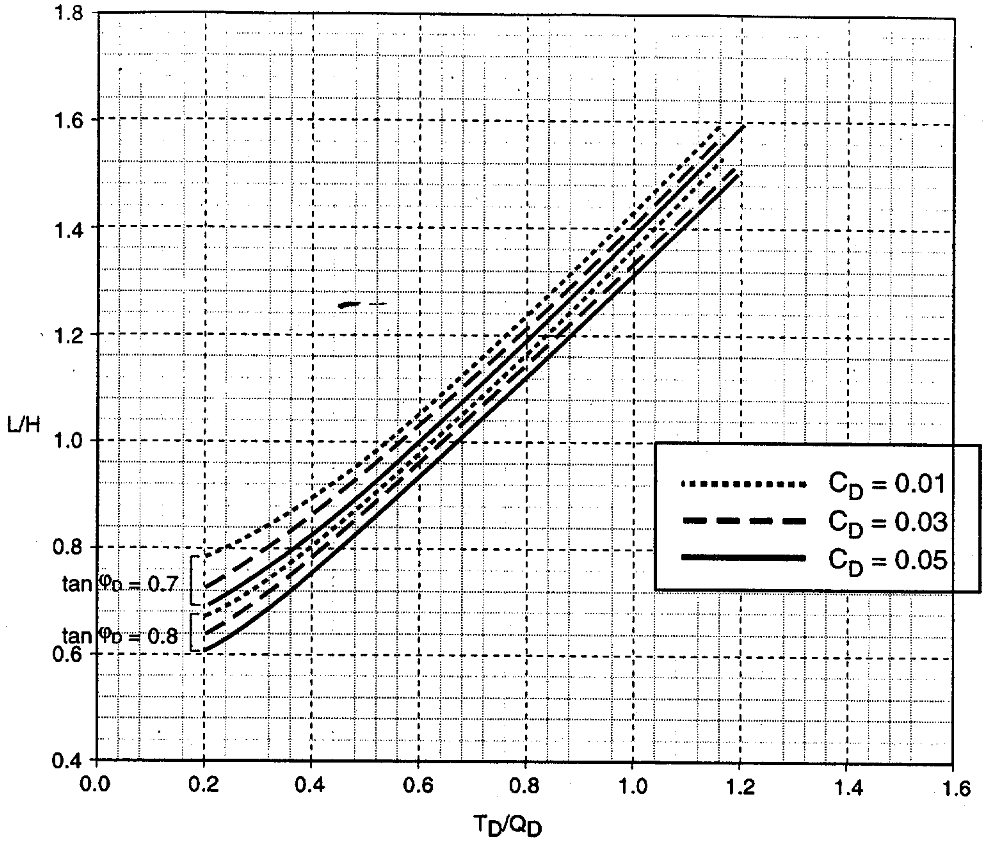
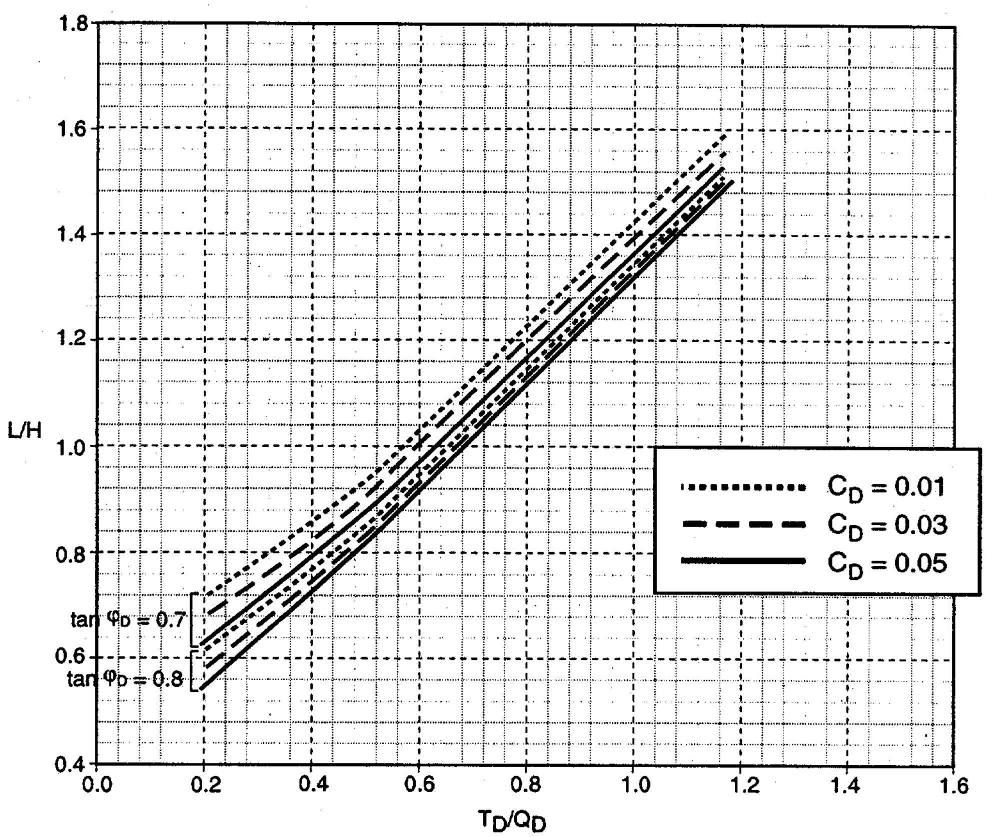


Figure 5.34A Preliminary Design Chart 4A
Backslope = 34°



Backslope = 34° Face Batter = 0° (Chart 4B)



Backslope = 34° Face Batter = 10° (Chart 4C)

Figure 5.34B and C Preliminary Design Charts 4B and 4C

common nail declination of 15°. The design charts are presented in dimensionless format, with the following variables:

Geometric Variables

Backslope Angle, β

Four sets of design charts are presented (three charts per set), with each set of charts corresponding to a single backslope angle of 0, 10, 20 or 34 degrees. For intermediate backslope angles, interpolate between the charts.

Face or Batter Angle, δ

For each backslope angle, design information is presented for two face or batter angles of 0 and 10 degrees from the vertical. For intermediate face or batter angles, interpolate between the charts.

Strength Variables

Factored Friction Angle, ϕ_D

The factored friction angle of the soil is defined by the following relationship:

SLD

$$\phi_D = \tan^{-1}[\tan(\phi_U)/F_\phi] \qquad (5.1a)$$

LRFD

$$\phi_D = \tan^{-1}[\Phi_\phi \tan(\phi_U)] \qquad (5.1b)$$

The factored friction angle is shown on the horizontal axis of chart A of each chart set.

Dimensionless Cohesion c_D

c_D is the soil cohesion normalized with respect to the soil unit weight and the vertical height of the cut:

SLD

$$c_D = c_U/(F_C \gamma H) \qquad (5.2a)$$

LRFD

$$c_D = \Phi_C \, c_U/(\Gamma_W \gamma H) \qquad (5.2b)$$

The dimensionless cohesion is shown as a parameter for each slope geometry for three values of 0.01, 0.03 and 0.05. Interpolate for intermediate values of the dimensionless cohesion.

Dimensionless Nail Tensile Capacity T_D

The dimensionless nail tensile capacity is the factored nominal nail tensile strength normalized with respect to the soil unit weight, the vertical height of the slope, and the nail spacings:

SLD

$$T_D = \alpha_N T_{NN}/(\gamma H S_V S_H) \quad (5.3a)$$

LRFD

$$T_D = \Phi_N T_{NN}/(\Gamma_W \gamma H S_V S_H) \quad (5.3b)$$

The dimensionless nail tensile capacity is shown on the vertical axis of chart A of each chart set.

Dimensionless Pullout Resistance Q_D

The dimensionless pullout resistance is the factored ultimate pullout resistance (expressed as a force per unit length of nail), normalized with respect to the soil unit weight and the nail spacings:

SLD

$$Q_D = \alpha_Q Q_U/(\gamma S_V S_H) \quad (5.4a)$$

LRFD

$$Q_D = \Phi_Q Q_U/(\Gamma_W \gamma S_V S_H) \quad (5.4b)$$

The dimensionless pullout resistance is shown as being incorporated into the ratio (T_D/Q_D) on the horizontal axis of charts B and C of each chart set.

5.3.2 Design Chart Procedure

The procedure for using the design charts in conjunction with the dimensionless variables discussed above consists of the following:

Step 1

Select the design chart set corresponding to the appropriate backslope angle. If necessary, interpolate results for intermediate backslope angles to those given in the charts.

Step 2

Compute the factored soil friction angle ϕ_D and the dimensionless factored soil cohesion c_D as defined above (equations 5.1 and 5.2). From the appropriate chart A, determine the dimensionless nail tensile capacity T_D.

Step 3

The required nominal nail tensile strength T_{NN} can then be determined from the relations presented above (equations 5.3), and from knowledge of the dimensionless nail tensile capacity T_D (from chart A), the soil unit weight, the vertical height of the slope, the vertical and horizontal nail spacings, and the nail tendon strength factor (SLD) or the nail tendon resistance factor and unit weight load factor (LRFD).

Step 4

Compute the dimensionless nail pullout resistance Q_D (equations 5.4). Divide the calculated dimensionless nail tensile capacity T_D by the computed dimensionless nail pullout resistance Q_D, and determine the required nail length from the appropriate Charts B/C, depending on the batter of the face of the wall.

5.3.3 Example Application of Design Charts

The use of the preliminary design charts is demonstrated below for both SLD and LRFD using the cutslope example of section 5.1. The parameter values and units are given in section 5.1.

Service Load Design (SLD)

Step 1

Figure 5.1 shows that the design section has face batter of 10° and a backslope angle of 20°. Therefore the design chart set is that presented on figures 5.33A and 5.33C.

Step 2

$$\phi_D = \tan^{-1}[\tan(\phi_U)/F_\phi] = \tan^{-1}[\tan(34°)/1.35] = 26.5°$$

$$\tan(\phi_D) = \tan(26.5°) = 0.5$$

c_D = $c_U/(F_C\gamma H)$ = $(5.0\ kN/m^2)/[1.35(18.0\ kN/m^3)(9.50\ m)]$ = 0.022

From Chart A (figure 5.33A), T_D = 0.23

Step 3

$T_D = \alpha_N T_{NN} / (\gamma H S_V S_H)$
$T_{NN} = \gamma H S_V S_H T_D / \alpha_N = (18.0\ kN/m^3)(9.50\ m)(1.50\ m)(1.50\ m)(0.23)/(0.55)$
T_{NN} = 161 kN (Required nominal nail strength.)

Step 4

$Q_D = \alpha_Q Q_U / (\gamma S_V S_H) = (0.50)(60.0\ kN/m)/[(18\ kN/m3)(1.50\ m)(1.50\ m)]$
Q_D = 0.74

T_D/Q_D = 0.23/0.74 = 0.31

From Chart C (figure 5.33C), L/H = 0.87

L = 0.87(9.50 m) = 8.3 m

In summary, the design charts indicate a required bar yield strength of about 161 kN (use No. 25, Grade 420 bars), and a nail length of about 8.3 meters. This nail length could be slightly conservative since the effective backslope angle of the design section shown on figure 5.1 may be something less than 20° (i.e., the design charts are prepared for constant backslope angles only, whereas the design example backslope angle is variable). For comparison purposes, a backslope angle of 10° would indicate a required nail length of about 7.3 meters.

Load and Resistance Factor Design (LRFD)

Step 1

Figure 5.7 shows that the design section has face batter of 10° and a backslope angle of 20°. Therefore the design chart set is that presented on figures 5.33A and 5.33C.

Step 2

$\phi_D = \tan^{-1}[\Phi_\phi \tan(\phi_U)] = \tan^{-1}[0.75\tan(34°)] = 26.8°$

$\tan(\phi_D) = \tan(26.8°) = 0.51$

$c_D = \Phi_c c_U / (\Gamma_W \gamma H) = (0.9)(5.0\ kN/m^2)/[1.35(18.0\ kN/m^3)(9.50\ m)] = 0.019$

From Chart A (figure 5.33), T_D = 0.23

Step 3

$T_D = \Phi_N T_{NN} / (\Gamma_W \gamma H S_V S_H)$
$T_{NN} = \Gamma_W \gamma H S_V S_H T_D / \Phi_N$
$T_{NN} = (1.35)(18.0\ kN/m^3)(9.50\ m)(1.50\ m)(1.50\ m)(0.23)/(0.90)$
$T_{NN} = 133\ kN$ (Required nominal nail strength.)

<u>Step 4</u>

$Q_D = \Phi_Q Q_U/(\Gamma_W \gamma S_V S_H) = (0.70)(60.0\ kN/m)/[(1.35)(18\ kN/m3)(1.50\ m)(1.50\ m)]$
$Q_D = 0.77$

$T_D/Q_D = 0.23/0.77 = 0.30$

From Chart C (figure 5.33C), $L/H = 0.88$

$L = 0.88(9.50\ m) = 8.4\ m$

In summary, the design charts indicate a required bar yield strength of about 133 kN (use No. 25, Grade 420 bars, although No. 22, Grade 420 bars could also be used) and a nail length of approximately 8.4 m. This nail length could be slightly conservative since the effective backslope angle of the design section shown on figure 5.7 may be something less than 20° (i.e., the design charts are prepared for constant backslope angles only, whereas the design example backslope angle is variable). For comparison purposes, a backslope angle of 10° would indicate a required nail length of about 7.3 meters.

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