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NCHRP REPORT 701

Proposed Specifications for LRFD Soil-Nailing Design and Construction

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Subscriber Categories
Bridges and Other Structures • Geotechnology • Highways

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in cooperation with the Federal Highway Administration

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FOREWORD

By David A. Reynaud
Staff Officer
Transportation Research Board

This report contains proposed specifications for the design and construction of soil-nailed retaining structures. Despite their advantages in cut applications, these structures are not available to some state DOTs, due to the lack of guidance for their use in AASHTO's standard specifications based on load and resistance factor design (LRFD). This report will be of interest to geotechnical engineers and construction managers, who would like to promote a more common utilization of soil nailing.

The soil-nailing method of earth retention is the preferred retaining wall option for many cut applications, because their advantages may include cost, speed of construction, construction flexibility, and aesthetics. Federal Highway Administration (FHWA) Demonstration Project No. 103 developed comprehensive design and construction manuals for temporary and permanent soil-nailed structures. These FHWA soil-nailing manuals contained a detailed design protocol for allowable stress design (ASD) and a preliminary load and resistance factor design (LRFD) approach.

The *AASHTO Standard Bridge Specifications,* the *AASHTO LRFD Bridge Design Specifications,* and the *AASHTO LRFD Bridge Construction Specifications* do not include guidance for soil-nailed structures. In the absence of AASHTO LRFD specifications, some state departments of transportation will not use soil-nailed retaining structures. Given the advantages of soil-nailed structures, there is a need to develop proposed standard design and construction specifications for soil-nailed structures for incorporation into the AASHTO LRFD Bridge Design and Construction Specifications.

The objective of NCHRP Project 24-21 was to develop these proposed LRFD design and construction specifications for soil-nailed retaining structures. To accomplish the project objective, the research agency, Geosyntec Consultants, used the existing FHWA guidelines on soil nailing, conducted a comprehensive review of current soil-nailing design and construction guidance for both ASD and LRFD specifications, and drafted proposed LRFD design and construction specifications. The research team subsequently identified, evaluated, and calibrated a range of resistance factors, based on the level of detail and confidence in the accuracy of the site investigations for multiple soil nail wall (SNW) project scenarios. These resistance factors were used with current AASHTO load factors to design SNWs using LRFD methodology and compared to SNWs designed using ASD methodology for the same project scenarios to demonstrate equivalence.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

SUMMARY

Proposed Specifications for LRFD Soil-Nailing Design and Construction

NCHRP Project 24-21 was conducted to develop procedures based on the load and resistance factor design (LRFD) method for the design of soil nail walls (SNWs) according to the most common U.S. practice in this technology. The work consisted of several tasks, including (i) a review of procedures and specifications for the design and construction of SNWs in both the LRFD and the allowable stress design (ASD) methods, (ii) compilation of soil nail load-test data and load data from instrumented walls, (iii) development of databases for pullout resistance and loads in SNWs, (iv) development of resistance factors based on the databases using reliability methods, and (v) comparison of designs using the LRFD and ASD methods and establishment of differences. The review of procedures for the design and construction of SNWs was focused on U.S. practice, although international references were also consulted. The task also comprised the review of current/interim editions of the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* (AASHTO, 2007).

A significant volume of soil nail load-test data was collected from several sources. After several results were eliminated due to lack of information or inconsistencies, a database of nail pullout resistance was compiled to support the calibration of pullout resistance factors. The volume of pullout resistance data was sufficient to create data subsets for three subsurface conditions, namely predominantly sandy soils, clayey soils, and weathered rock. More data points were available from projects of SNWs constructed in sandy soils than in clayey soils and weathered rock. To reduce the scatter due to variable levels of workmanship and equipment among different contractors, data was selected, as much as possible, from the same contractor using the same equipment at the same project.

Statistical parameters were obtained for four soil/rock types for the pullout capacity. In addition, soil nail load data allowed an estimation of the statistical parameters for the bias of loads. Load and resistance were considered as lognormal random variables. Resistance factors for elements that are common to other retaining systems (e.g., factor for the nominal tensile resistance of steel bars) were adopted from the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2007) for consistency. Current values were found to be acceptable for the design of SNWs. The calibration of the resistance factor for soil nail pullout was conducted using reliability methods as suggested by Allen et al. (2005) for the development of load and resistance factors in geotechnical and structural design. The target reliability index was selected based on a comparison of SNWs with other substructures that have comparable levels of structural redundancy and for which target reliability indices have been proposed. The reliability selected for SNWs was 2.33, which is consistent with the value used for the calibration of resistance parameters for pullout in mechanically stabilized earth (MSE) walls. The calibration used a Monte Carlo simulation using statistical parameters for load and

resistances selected earlier and up to 10,000 random simulations for each of the load and resistance variables.

To be consistent with the AASHTO (2007) specifications, overall stability was adopted to be a service limit state where limit-equilibrium methods are applied. Although load factors are 1.0 for service limit states, a series of pullout resistance factors was obtained for a range of load factors other than 1.0 to show the effect of load factors on the pullout resistance factor for each of the soil/rock types considered. The load factors selected were $\lambda_Q = 1.0$, 1.35, 1.5, 1.6, and 1.75. This range represents the values that can be commonly used for retaining structures that are part of bridge substructures. Calibrated pullout resistance factors based on this range of load factors are presented.

Calibration resistance factors were subsequently used to perform comparative designs for SNWs for a wide variety of conditions. The objective of the comparative designs was to evaluate differences of the required soil nail length, as obtained using computer programs with the ASD method or the LRFD method. Over 30 design cases were considered to assess the effect of several key factors in the design. These factors included wall height, soil friction angle, bond resistance, and surcharge loads. Results of the comparative designs indicate that the required soil nail length calculated using the LRFD method and the proposed resistance factors were quite close to those obtained with the ASD method. For all cases considered, the bar lengths are, on average, approximately only 4% longer in the LRFD method. None of the factors studied in this comparison appear to have a greater influence over other factors on the calculated nail lengths, possibly with the exception of surcharge loads. The largest difference obtained in the comparative analysis was approximately 8%. The comparative designs mentioned previously have shown that the design of SNWs using the LRFD method would result in comparable, although not identical (only slightly higher), quantities to those obtained with the ASD method. There are no essential differences in the requirement of bar diameters, bar lengths, and facing dimensions and quantities using either method. The use of the LRFD method allows SNWs to be designed with a reliability level that is compatible with reliability levels of other elements of a bridge superstructure or other comparable retaining systems.

Proposed specifications for the design and construction of SNWs were also developed and are provided as appendices to this report. The proposed specifications follow the format of AASHTO (2007).

CHAPTER 1

Background

1.1 Introduction

This report presents the results of NCHRP Project 24-21, "LRFD Soil-Nailing Design and Construction Specifications." The report contains the results of a review of the load and resistance factor design (LRFD) method used for geotechnical applications, including soil nail walls (SNWs) and the results of a comprehensive review of soil-nailing design and construction procedures used in current U.S. practice. Subsequently, the report includes the basis for developing a database of soil nail pullout resistance tests, loads, and calibration results of resistance factors applicable to SNWs. A comparison of the designs of SNWs using both the LRFD and the allowable stress design (ASD) methods for identical loads, wall geometry, and material conditions is also presented. A summary of findings and suggested topics for additional research are included. Appendices include potential sections of LRFD specifications for the design and construction of SNWs, a database of soil nail pullout resistance tests, and comparative analyses. The potential LRFD specifications were developed for consideration by the American Association of State Highway and Transportation Officials (AASHTO) for future editions of the *LRFD Bridge Design Specifications.*

1.2 Problem Statement

LRFD-based design methods for steel and reinforced concrete components of bridges and structures have been used for many years in the United States (e.g., Galambos and Ravindra, 1978; AISC, 1994; and ACI, 1995). Before the 1990s, bridge components, including substructure components (e.g., bridge foundations), were designed using the ASD method, as presented in the AASHTO *Standard Specifications for Highway Bridges.* However, this situation changed in the early 1990s, when AASHTO developed design specifications, titled *AASHTO LRFD Bridge Design Specifications* (AASHTO, 1994), for highway bridges. Since the first edition, updated editions [e.g., 4th edition (AASHTO, 2007)] and interim versions of the *LRFD Bridge Design Specifications* have been published every few years.

The main objective of the *LRFD Bridge Design Specifications* is to promote the use of the LRFD method and thereby realize the perceived advantages of this method over the ASD method for the design of highway bridges and substructures. Some bridge substructures components [e.g., shallow foundations, deep foundations, and mechanically stabilized earth (MSE) walls] were addressed in the first edition of the *LRFD Bridge Design Specifications,* and other bridge substructures have been only progressively added to more recent editions. However, other substructure components, including SNWs, have not been included through the latest edition (i.e., 2007) of the *LRFD Bridge Design Specifications.*

Introduced in the United States in the mid-1970s, the use of SNWs in this country has increased in the last two decades or so due, in part, to the advantages of SNWs over comparable retaining systems, including anchored walls, for certain subsurface and project conditions. Some of the advantages of SNWs over other systems include lower cost, faster installation, use of smaller equipment, and a larger structural redundancy (e.g., more soil nails are installed per unit area than ground anchors). The use of SNWs as a permanent retaining structure in transportation projects became more common in the late 1980s and early 1990s thanks largely to the sponsorship of the Federal Highway Administration (FHWA). FHWA has financed the preparation of seminal documents for the design and construction of SNWs that have helped promote this technology. In fact, nowadays, the analysis, design, and construction of SNWs in the United States are commonly performed using procedures contained in documents developed on behalf of FHWA.

For example, FHWA commissioned the first comprehensive document for the design and construction of SNWs (Elias and Juran, 1991). In 1993, FHWA sponsored a tour to Europe for FHWA engineers and U.S.-based professors and consultants to gather information on SNWs in those European countries that were at that time leading the use of this technology. Findings of the tour were summarized in a publication (FHWA, 1993a). In 1993, FHWA also commissioned the English translation of the French national manual on soil nail technology (FHWA, 1993b), which was then one of the most advanced documents in this field. In 1994, FHWA initiated Project Demonstration 103 to disseminate the use of SNWs among state departments of transportation (DOTs). As part of this effort, FHWA published "Soil Nailing Field Inspectors Manual, Project Demonstration 103" (Porterfield et al., 1994). Project Demonstration 103, whose initial contributors were engineering consulting firms and research institutions, evolved into a manual for the design and construction of SNWs a few years later (Byrne et al., 1998). The 1998 FHWA manual presented both ASD- and LRFD-based methodologies for the design of SNWs. More recently, FHWA published an updated manual on the design and construction of SNWs in the series titled "Geotechnical Engineering Circulars" (GECs) as GEC No. 7 (Lazarte et al., 2003).

The 1998 FHWA manual on SNW design (Byrne et al., 1998) provided uncalibrated resistance factors for pullout resistance that had been developed simply by relating them to safety factors used in common SNW practice, as contained in the 16th edition of the ASD-based AASHTO *Standard Specifications* (AASHTO, 1996). GEC No. 7 (Lazarte et al., 2003) addressed only the ASD method. Therefore, a fully calibrated LRFD methodology for SNWs was lacking and hence was not included in the initial versions of the *LRFD Bridge Design Specifications.* To allow SNWs to be included in the *LRFD Bridge Design Specifications* and to further promote the use of SNWs by all state DOTs, particularly among those that have not applied this technology (in part because of the absence of SNWs in AASHTO design specifications), AASHTO funded this research through NCHRP.

1.3 Research Objectives

NCHRP established the following objectives for this research:

- Review existing procedures and specifications in current U.S. and international practice for the design and construction of SNWs;
- Examine existing LRFD-based guidance for the design of SNWs used in U.S. practice; and
- Obtain the necessary information from soil nail load tests to develop statistically based load and resistance factors for SNWs.

1.4 Report Organization

The remainder of this report is organized as follows:

- Chapter 2, Research Approach, provides a description of the methodology followed to meet the research objectives;
- Chapter 3, Findings and Applications, presents:
  - A summary of a review of the current use of the LRFD method in geotechnical design;
  - A summary of a review of current soil-nailing practice, focused on the U.S. practice;
  - An introductory discussion of load and resistance factors to be used for SNW design;
  - A brief description of a database of soil nail load tests developed for this research;
  - Statistics of predicted and measured loads and resistances for SNW limit states; and
  - Calibration results of resistance factors for soil nail pullout.
- Chapter 4, Conclusions and Suggested Research, provides a summary of research findings and suggestions for future research.
- Lists of references, abbreviations, and symbols are provided.

Additional information is presented in the following appendices:

- Appendix A: Proposed LRFD Design Specifications for Soil Nail Walls;
- Appendix B: Proposed LRFD Construction Specifications for Soil Nail Walls;
- Appendix C: Soil Nail Test Pullout Resistance and Load Database; and
- Appendix D: Comparison of ASD- and LRFD-Based Designs of Soil Nail Walls.

CHAPTER 2

Research Approach

2.1 Introduction

To achieve the objectives established for this project, the following research approach and research tasks were established:

- Review existing procedures for the design and construction of SNWs according to the ASD and LRFD methodologies;
- Compile load-test data from several sources;
- Develop resistance factors through calibration of load-test data using appropriate reliability-based methods;
- Compare designs of SNWs prepared with LRFD and ASD methods; and
- Prepare LRFD design and construction specifications for SNWs to be considered by AASHTO for future editions of the AASHTO *LRFD Bridge Design Specifications.*

Each of these tasks is described in further detail in the following paragraphs.

2.2 Review of Design and Construction Procedures for Soil Nailing

First a review of the LRFD method as applied to geotechnical applications and retaining structures was made. As part of this review, an evaluation was performed of LRFD methodologies developed for other bridge substructure components that shared some common aspects with SNWs. Subsequently, a review was conducted of existing procedures for the design and construction of SNWs focused on U.S. practice. Relevant state-of-the-art publications related to the design of SNWs, including Byrne et al. (1998), Lazarte et al. (2003), and other recent national and international references (e.g., Clouterre, 2000) addressing SNW design were reviewed.

2.3 Compilation of Soil Nail Load-Test Data

A database of soil nail load-test results was compiled to provide data for the calibration of resistance factors for soil nail pullout. Sources of information included load-test results from (i) files owned by the research team, (ii) members of ADSC: The International Association of Foundation Drilling, (iii) other SNW contractors, (iv) DOTs, (v) research institutions, and (vi) published journals and reports. Chapter 3 provides a description of the data and contains a discussion of data adequacy for calibration purposes.

2.4 Development of Resistance Factors through Calibration of Load-Test Data

Resistance factors for the design of SNWs were developed and calibrated applying reliability methods and using the values contained in the soil nail test database. The calibration was conducted using the procedures presented in the publication "Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD Foundation Strength Limit State Design" (Allen, 2005). Chapter 3 provides the results of the calibration.

2.5 Comparisons of Designs Based on the LRFD and ASD Methods

Calibrated resistance factors were used in LRFD-based designs of various SNWs. These designs were compared with designs obtained using the ASD method for the same SNWs and load conditions. Differences of key design parameters in SNWs design were assessed and potential advantages of the LRFD-based methodology were quantified. Proposed changes to be considered in future editions of the *LRFD Bridge Design Specifications* were identified.

2.6 Proposed LRFD Design and Construction Specifications

LRFD-based specifications for the design and construction of SNWs were developed as part of this research. Appendices A and B, respectively, contain the proposed design and construction specifications, which are formatted according to the latest *LRFD Bridge Design Specifications* (AASHTO, 2007).

CHAPTER 3

Findings and Applications

3.1 Overview

This chapter first presents the results of a review of current LRFD practice in geotechnical design, introduces the basis for LRFD-based methods for retaining structures, and provides the results of a review of current U.S. practice of soil nailing. Subsequently, the chapter provides discussions of LRFD limit states in the design of SNWs and a synthesis of approaches used to calibrate resistance and load factors. Finally, calibrations of resistant factors are presented.

3.2 Review of Current LRFD Practice

3.2.1 Historical Development of LRFD

3.2.1.1 Structural Design

The early use of concepts of probability and reliability, as used to quantify uncertainties in the design of structures (Freudenthal, 1947, 1951; Freudenthal and Gumbel, 1956), set the basis for the subsequent development of the LRFD framework. In the 1970s and 1980s, the development of LRFD methods for structural applications advanced substantially when various structural codes started to incorporate reliability concepts. For example, reliability was used in the American National Standards Institute code (ANSI) for design loads for buildings (as summarized by Ellingwood et al., 1980; Ellingwood and Galambos, 1982; Ellingwood et al., 1982a and 1982b). Other design codes incorporating LRFD concepts included those for steel construction [American Institute of Steel Construction (AISC), 1994; Galambos and Ravindra, 1978], concrete construction [American Concrete Institute (ACI), 1995], and offshore platforms [American Petroleum Institute (API), 1989; Moses, 1985, 1986]. International building codes containing reliability or LRFD methods included the National Building Code of Canada (Siu et al., 1975; National Research Council of Canada, 1977) and Report 63 developed by the United Kingdom's Construction Industry Research and Information Association (CIRIA, 1977).

3.2.1.2 Geotechnical Design

In an early effort to distinguish different sources of uncertainty in geotechnical design, Taylor (1948) proposed the use of separate and independent factors of safety for the cohesion and frictional components of soil resistance. However, the concept of a load factor, which incorporates the uncertainty related to loads, was not used in geotechnical design at that time. All uncertainty in geotechnical design was concentrated in the resistance. The use of both load and resistance factors in geotechnical engineering was initiated by Brinch-Hansen in Denmark (Brinch-Hansen 1953, 1956, 1966). Later publications related to the use of LRFD concepts in geotechnical design include Barker et al. (1991) for foundations and retaining structures, Fellenius (1994) and Meyerhof (1994) for shallow foundations, O'Neill (1995) for deep foundations, Hamilton and Murff (1992) and Tang (1993) for foundations of offshore platforms, Kulhawy and Phoon (1996) for foundations of transmission towers, Withiam et al. (1991, 1995) and D'Appolonia (1999) for retaining structures, Allen et al. (2001) and Chen (2000a, 2000b) for MSE walls, and Paikowsky et al. (2004, NCHRP Project 24-17) for deep foundations.

3.2.2 Overview of Uncertainty in Design of Structures

This section provides an overview of common approaches in dealing with uncertainty in structural design. In the design of structures, a number of uncertainty sources must be addressed. These sources may include the following:

- Material dimensions and location/extension;
- Material properties, including unit weight/density and strength;
- Long-term material performance;
- Possible failure modes;
- Methods used to analyze loads and evaluate load distribution;

- Methods used to predict transient loads;
- Methods used to predict the structural response; and
- Potential changes over time associated with the structural function.

Besides the sources listed above, in geotechnical design, uncertainties also arise from the variability of subsurface conditions, the intrinsic errors made in the estimation of material properties, and the divergences that occur due to the differences between the estimated and actual properties of the structure. The variability of subsurface conditions arises as a result of the spatial variability of soil and rock properties. Spatial variability of soil/rock properties may be caused by differences in geology across a site; in contrast, local variability of soil/rock properties commonly results from the inherent heterogeneities of most natural materials. Intrinsic errors in the estimation of material properties (i.e., usually referred to as bias) arise from (i) sampling methods used to obtain soil/rock specimens [e.g., a standard penetration test (SPT)]; (ii) field or laboratory testing techniques used to evaluate soil/rock properties (e.g., SPT blow count or triaxial tests); and (iii) models used to interpret and predict soil/rock properties (e.g., Mohr-Coulomb model). Measurements of soil/rock properties in the field and laboratory produce random errors that are typical of all measurements. Finally, uncertainty in geotechnical design may also occur due to differences between the assumed or estimated properties and the actual properties of the constructed structure as a result of differing construction methods or insufficient construction quality control and assurance.

3.2.3 Overview of the ASD Method

Uncertainty in engineering design has traditionally been addressed with factors of safety (*FS*) in the allowable stress design (ASD). In the ASD method, allowable "stresses" (or, more generally, resistances) of structural components are obtained by dividing the values of ultimate strengths of those structural components by *FS*. The general design condition in the ASD method can be expressed as:

$$\Sigma Q_i \leq R_{all} = \frac{R_n}{FS} \tag{3-1}$$

where

ΣQ_i = the effect of all combined loads on a given structural component for a given failure mode,
R_{all} = the allowable stress of that structural component,
R_n = the ultimate or maximum strength of that structural component, and
FS = the factor of safety applied to that ultimate resistance.

Allowable stresses represent normal working conditions of a structural element and are therefore selected lower than the ultimate capacity of the structural element. Structures have various components that may be subjected to numerous loading conditions, possibly involving different potential failure modes. As a result, numerous equations, similar in format to Equation 3-1, must be considered to achieve a safe design of each structural component and of the entire system for all expected conditions.

In Equation 3-1, all uncertainty is concentrated in *FS* that appears on only one side of the design equation. *FS* is typically adopted based on experience, engineering judgment, and common practice. It is not usually based on uncertainty quantifications (i.e., by establishing the probability of failure of a selected failure mode or structural component). Minimum values of *FS* recommended for design of certain structures are selected generally by agencies with jurisdiction or interest on those structures. For example, for the design of bridge structures and substructures, AASHTO has developed a set of *FS* values that is contained in the ASD-based AASHTO Standard Specifications (AASHTO, 1996).

In general, *FS* values that are selected based on experience tend to provide safe and reasonably economical designs after years of practice. However, the selection of new *FS* values for new problems (i.e., use of materials, construction methods, or consideration of infrequent loading) may be more challenging than simply selecting values based on existing ranges. In deriving *FS* values for new problems, different design practitioners may select different *FS* values if only engineering judgment is used. The ASD method may occasionally provide inconsistent levels of safety for structures involving various components with multiple factors of safety (each possibly involving different probabilities of failure). To overcome some of these limitations of the ASD method, the LRFD has been developed.

3.2.4 Overview of the LRFD Method

3.2.4.1 Objectives and Basic Description of the LRFD Method

To address design uncertainty in a more systematic manner than in the ASD method, the LRFD method was developed with the following objectives: (i) to account for uncertainty in loads and resistances separately with the use of factors for load and resistance; (ii) to provide reliability-based load and resistance factors based on accepted levels of structural reliability; and (iii) to provide consistent levels of safety across a structure when several components are present. This approach is used in the current AASHTO *LRFD Bridge Design Specifications.*

In the LRFD method, two parameters account for uncertainty: load factor for load uncertainty and resistance factor for material uncertainty. The use of separate parameters is justified because the nature, variability, and hence level of uncertainty associated with loads are different than the uncertainty related to resistance. In principle, the LRFD method can result in more consistent levels of safety across the entire structure because the relationship between the levels of safety of different structural members is accounted for in this method. Resistance and load factors are selected using probability-based techniques so that these factors are related to acceptable levels of structural reliability, which is equivalent to a tolerable probability of failure. Unlike the *FS,* the LRFD-based parameters are calibrated with respect to actual load and resistance data.

Load and resistance factors are related to each other through limit states. A limit state is a condition in which the structure as a whole, or one of its components, has achieved a level of stress, deformation, or displacement that may affect its performance.

In the LRFD method included in the AASHTO *LRFD Bridge Design Specifications,* four types of limit states are defined:

(i) Strength limit states,
(ii) Extreme-event limit states,
(iii) Service limit states, and
(iv) Fatigue limit states.

Therefore, the design objectives in the LRFD methodology are to demonstrate that (i) the available resistance (i.e., for strength and extreme-event limit states) is sufficient; (ii) other structural conditions (e.g., tolerable deformations in service limit states) are within tolerable limits; and (iii) the structural performance is adequate for all foreseeable load conditions arising during the design life of the structure.

In general, all of these limit states must be considered in the design of structural elements, although not all limit states are directly applicable for geotechnical design. These limit states are described in more detail in the following subsections.

3.2.4.2 Strength Limit States

Strength limit states are those related to the strength (i.e., generally referred to as nominal resistance in the LRFD convention, as defined subsequently) and the stability of structural components during the design life of the structure. For each strength limit state, a design equation can be generically expressed as:

$$\phi R_n \geq \sum_{i=1}^{N} \gamma_i \eta_i Q_i \qquad (3\text{-}2)$$

Where

R_n = the nominal resistance of a given structural component for the strength limit state being considered;
ϕ = a non-dimensional resistance factor related to R_n;
Q_i = the *i*-th load type that participates in this limit state;
γ_i = a non-dimensional load factor associated with Q_i;
η_i = a load-modification factor; and
N = the number of load types considered in the limit state.

These quantities are described in the following paragraphs.

Nominal resistance is the resistance of an entire structure (or of one of its components), which is established based on stresses or deformations or is a specified strength of the materials involved in the structure. In general, nominal resistances of structural components are derived from the specified materials and dimensions. For example, the specified tensile yield strength of a steel bar is typically a nominal strength. However, the nominal resistance of soils and other natural materials is obtained differently. The nominal resistance of soils is derived using suitable field/laboratory methods or other acceptable means (e.g., correlations between field test results and soil strength parameters). The nominal resistance of soils commonly represents an ultimate strength of the soils. For example, the internal friction angle of granular soils, which is routinely estimated from field/laboratory tests or correlations, is an ultimate strength to be used in establishing the nominal resistance of soils.

Resistance factors commonly reduce nominal resistances; therefore, they are typically ≤ 1.0. Section 10, Foundations, and Section 11, Abutments, Piers, and Walls, of the *LRFD Bridge Design Specifications* (AASHTO, 2007) present prescribed values of resistance factors for geotechnical design of bridge substructure components.

Load factors (γ_i) are statistically based multipliers that are used in the LRFD method to account for load variability sources (e.g., frequency of loads, inaccuracies in load estimation, and likelihood of simultaneous load occurrences). While the resistance factors remain the same once they are selected, different γ_i are selected for different load combinations. For strength limit states, load factors are typically ≥ 1.0 if the acting load is destabilizing. Conversely, load factors are ≤ 1.0 if the acting load component tends to stabilize the structure. An example of stabilizing loads is the horizontal force that arises from soil passive pressures that resist the lateral movement of an embedded foundation. Guidance for selecting load factors for different load combinations in bridge substructure components are contained in Table 3.4.1-1, Load Combinations and Load Factors, and Table 3.4.1-2, Load Factors for Permanent Loads, of Section 3, Loads and Load Factors, of the *LRFD Bridge Design Specifications* (AASHTO, 2007). The number of load components (N) may vary for different load combinations, as presented in AASHTO (2007).

Factor η_i accounts for redundancy, ductility, and importance of the structure and varies between 0.95 and 1.05. Additional guidance for the selection of these factors can be found in Section 1.3, Design Philosophy, of AASHTO (2007).

3.2.4.3 Extreme-Event Limit States

Extreme-event limit states are those related to infrequent but large loads that have return periods exceeding the design life of the structure. Extreme–event limit states in bridges and substructures include loads arising from seismic events, ice formation, and vehicle and vessel collision. The same design equation used for strength limit states is commonly used for extreme-event limit states, although the load factors are different. The load factors that must be considered for different load combinations in extreme-event limit states are contained in Table 3.4.1-1 of AASHTO (2007).

3.2.4.4 Service Limit States

Service limit states are those states related to inadequate conditions that may arise during normal operation of the structure but do not cause a collapse. Inadequate conditions may include excessive deformation, excessive settlements, and cracking. For each service limit state, the following condition must be met:

$$S_{MAX} \leq S_{TOLERABLE} \tag{3-3}$$

Where

S_{MAX} = the maximum calculated value of a quantity S (e.g., deflection or settlement) expected to occur under normal conditions; and

$S_{TOLERABLE}$ = the maximum value of S the structure can sustain before its functionality is affected.

The load factors for different load combinations to be considered in service limit states are contained in Table 3.4.1-1 of AASHTO (2007).

Importantly, due to reasons that will be presented subsequently, overall stability, slope stability, and other stability states are considered service limit states per AASHTO (2007). For these cases, an equation similar to that of strength limit states is used, with the exception that all load factors are selected equal to 1.0 to reflect the assumption that the structure is under normal conditions.

3.2.4.5 Fatigue Limit States

Fatigue limit states are those states in which loads are applied repetitively and may affect the performance of a structure, while the stress levels are significantly below the values used in strength limit states. For example, fatigue limit states are applicable to structures that may be sensitive to fracture as a result of repetitive loads (e.g., vehicular loads and dynamic loads). Additional information on fatigue limit states can be found in Article 3.6.1.4.1 of AASHTO (2007).

3.2.5 Resistances and Loads as Random Variables

In the LRFD method, loads, Q, and resistances, R, are considered random independent variables with probability density functions $f_R(R)$ and $f_Q(Q)$ that are usually normal or lognormal (as shown in Figure 3-1), mean values Q_m and R_m, and standard deviations σ_Q and σ_R, respectively. R and Q are commonly assumed to be probabilistically independent in geotechnical design (Baecher and Christian, 2003). The variability of these random variables can be conveniently expressed through coefficients of variation (COV), which are defined as:

$$COV_Q = \frac{\sigma_Q}{Q_m} \tag{3-4}$$

$$COV_R = \frac{\sigma_R}{R_m} \tag{3-5}$$

COVs, which also can be expressed as a percentage, are useful as they express uncertainty as a fraction (or percentage) of the mean values.

Nominal values of loads and resistances, Q_n and R_n, are defined as:

$$Q_m = \lambda_Q Q_n \tag{3-6}$$

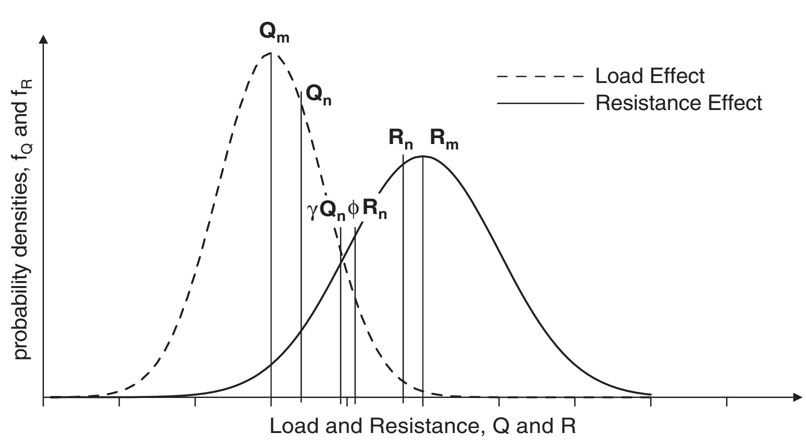


Figure 3-1. Probability density functions for load and resistance.

$$R_m = \lambda_R R_n \tag{3-7}$$

where λ_Q and λ_R are the bias factors for loads and resistances, respectively. Bias factors represent ratios of measured to predicted values of loads or resistance. In obtaining bias factors, predictive formulas used in the common practice or contained in design codes are considered. On the other hand, with a sufficiently representative database of measured loads and resistances of a structure component, statistical analyses can be performed to obtain bias factors and thereby assess the efficiency of design formulas in predicting measured values. In the case of resistance, predicted resistance are on average greater than measured resistances; therefore, $\lambda_R > 1$ and safe predictions are produced. Conversely, predictions are unconservative when $\lambda_R < 1$.

Design values of resistance are obtained by reducing nominal resistances with a resistance factor, ϕ, that is usually ≤ 1.0. Conversely, design values of loads are obtained by increasing nominal load values using a load factor, γ, that is usually ≥ 1.0 (Figure 3-1).

The random variables Q and R are related by the safety margin M, another random variable, which is defined as $M = R - Q$. According to this definition, a combination of Q and R values results in a safe condition when $M \geq 0$. An alternative definition of safety margin is $M' = R/Q$, in which case, the pair Q and R results in a safe condition when $M' \geq 1$. Note that the alternative definition coincides with the traditional ASD format using factors of safety.

A probabilistic density distribution for M, $f_M(M)$, with mean $M_m = R_m - Q_m$ and standard deviation σ_M, can be obtained based on the distributions of R and Q (Figure 3-2). The condition $M = 0$ is the limit state. If the alternative definition of safety margin is used, a distribution $f_{M'}(M')$ for M', with mean $M'_m = R_m/Q_m$ and standard deviation σ'_M, can be obtained. In this case, the condition $M' = 1$ is the limit state. For the alternative definition, an equation format similar to that of M is obtained by calculating log $(R/Q = 1)$, or $\log R - \log Q = 0$.

As illustrated on Figure 3-2, loads can potentially be larger than resistances and the probability that $R < Q$ is non-zero. The area under the probability density distribution $f_M(M)$ in the interval $M \leq 0$ is the probability of failure, P_f, which is defined as $P_f = P_i\,(R < Q) = P\,(R/Q < 1) = P\,(\ln R/Q < 0)$.

Probability of failure is a small number in practice; therefore, the reliability index, β, can be used instead to quantify the likelihood of failure. The reliability index is defined as the number of standard deviations, σ_M, of the probability density distribution $f_M(M)$ that exists between the mean value, M_m, and the limit state (i.e., $M = 0$) (Figure 3-2). In other words, β is the "distance" between points M_m and 0 on the M-axis that is normalized by σ_M.

R and Q are assumed to be probabilistically independent and it follows that the reliability index can be expressed as:

$$\beta = \frac{M_m}{\sigma_M} = \frac{R_m - Q_m}{\sigma_M} \tag{3-8}$$

If the alternate definition of safety margin is used, the reliability index can be expressed as:

$$\beta = \frac{\ln R_m - \ln Q_m}{\sigma_{M'}} \tag{3-9}$$

The reliability index increases when the probability of failure decreases and σ_M (or COV_M) decreases.

For $\beta \leq 2$, the reliability index is computed to be similar for both normal and lognormal probability distributions. For $\beta > 2$, the divergence for β for these distributions tends to increase significantly (Baecher and Christian, 2003).

If R and Q are normally distributed, the probability of failure, P_f, can be expressed as a function of β as follows:

$$P_f = \Phi^{-1}(-\beta) \tag{3-10}$$

where Φ^{-1} is the inverse of the cumulative distribution Φ of a standard normal function.

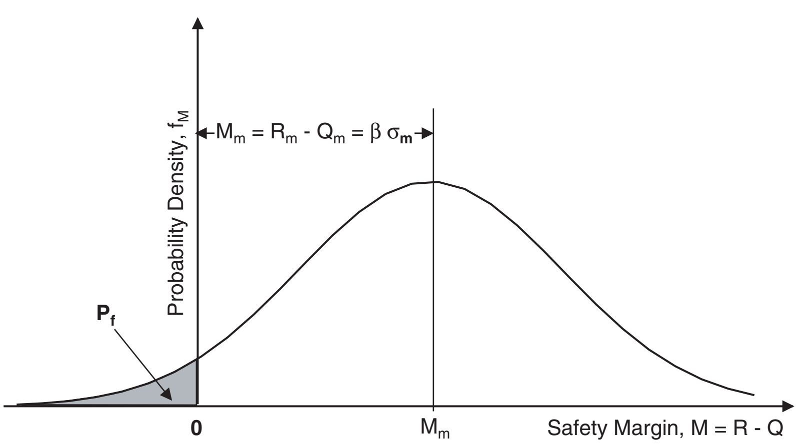


Figure 3-2. Probability density function of safety margin, M.

Values of the cumulative distribution of Φ and/or its inverse can be obtained from various probability and statistics references (e.g., Baecher and Christian, 2003) or can be computed using statistical software.

3.2.6 Approaches for Calibration of Resistance and Load Factors

One of the objectives mentioned for the LRFD method was to provide γ and ϕ factors that relate to acceptable levels of P_f. This relationship is established through a calibration, which is performed by fixing one of the factors (usually the load factor) and calibrating the other. Therefore, one factor cannot be modified without modifying the other. Calibrations can be performed using the following methods, each with an increasing level of complexity (Withiam et al., 1998):

- Method A: Calibration using engineering judgment;
- Method B: Calibration by matching factors to *FS* in ASD-based design codes; and
- Method C: Calibration using reliability-based procedures.

A description of each of these methods is presented in the following paragraphs.

Method A: Calibration Using Engineering Judgment

This method is best suited for situations where a great deal of experience is available among a summoned team of design professionals (for example, a panel of experts). This method can, in theory, be advantageous because it may incorporate proven design practices that have led to safe and cost-efficient projects. This approach may increase the confidence of other design engineers in certain design procedures. Disadvantages of this method include the possibility that the judgment of the panel members may be unintentionally biased.

Method B: Calibration by Matching Factors to Safety Factors Contained in Design Codes

In this method, resistance factors are calibrated by matching or calibrating them to *FS* values used in the ASD format. This approach is appealing because of its mathematical simplicity, consistency with earlier design practice, and transparency to most practicing engineers. This approach is commonly the first to be used until load and resistance statistics are available. However, the approach may not always address all sources of uncertainty in an explicit manner.

In this method, a resistance factor can be calibrated from a *FS* value as follows:

$$\phi \geq \frac{\Sigma\gamma_i Q}{FS\,\Sigma Q_i} \qquad (3\text{-}11)$$

where all variables were previously defined.

If the loads are limited to dead and live loads, therefore:

$$\phi \geq \frac{\gamma_{DC}Q_{DC} + \gamma_{LL}Q_{LL}}{FS(Q_{DC} + Q_{LL})} \qquad (3\text{-}12)$$

where subscripts *DC* and *LL* refer to permanent and live loads, respectively.

Method C: Calibration Using Reliability-Based Procedures

In this method, factors are calibrated according to a reliability analysis and are based on empirical data (e.g., load-test data). In addition, a tolerable level of uncertainty is selected. Tolerable levels of uncertainty are expressed through a target value of the reliability index, β_T, which reflects an accepted, low probability of failure for a given structure type and load scenario.

This method is more complex than Methods A and B and requires that adequate and sufficient empirical information be available. Comparative designs help evaluate the factors obtained in this method and correlate them with factors obtained using other methods. An advantage of this method is that it can provide more explicit insight on the bias of certain predictive design formulas and can help identify and quantify the largest sources of uncertainty arising in design. The method may not be amenable and transparent for engineers unfamiliar with reliability concepts.

Three different levels of calibration complexity can be achieved in Method C [Withiam et al. (1998)]—Levels I, II, and III—each of which is described in the following paragraphs.

Level I. Level I calibration is referred to as a first-order second-moment (FOSM) calibration methodology. At this level, the random variables *R* and *Q* and their mathematical derivatives used to derive β are only approximated. As discussed earlier, *R* and *Q* are assumed to be statistically independent. The key simplification in this method is that only the first-order derivatives of the squared values of *R* and *Q* and/or their derivatives (i.e., known as second moments in probability) are included, while higher-order terms are disregarded. In this method, the reliability index β is expressed as a linear approximation of *R* and *Q* around the mean values. An advantage of this method is that it can provide approximate, closed-form approximations for resistance factors.

If the random variables *Q* and *R* are normally distributed and statistically independent, the resistance factor can be estimated as (Withiam et al., 1998):

$$\phi = \frac{\lambda_R \Sigma\gamma_i Q_i}{Q_m + \beta_T\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad (3\text{-}13)$$

where all variables were defined previously.

If Q_i involves permanent and live loads, the resistance factor can be calculated as:

$$\phi = \frac{\lambda_R(\gamma_{DC}Q_{DC} + \gamma_{LL}Q_{LL})}{(\gamma_{DC}Q_{DC} + \lambda_{LL}Q_{LL}) + \beta_T\sqrt{\sigma_R^2 + \sigma_{DC}^2 + \sigma_{LL}^2}} \tag{3-14}$$

where all variables were defined previously.

If the random variables are lognormal, the resistance factor can be calculated as follows (Barker et al., 1991; Withiam et al., 1998):

$$\phi = \frac{\lambda_R \sum \gamma_i Q_i \sqrt{\dfrac{1+COV_Q^2}{1+COV_R^2}}}{Q_m \exp\left[\beta_T\sqrt{\ln(1+COV_R^2)(1+COV_Q^2)}\right]} \tag{3-15}$$

If Q_i involves permanent and live loads, the resistance factor can be calculated as:

$$\phi = \frac{\lambda_R\left(\dfrac{\lambda_{DC}Q_{DC}}{Q_{LL}} + \gamma_{LL}\right)\sqrt{\dfrac{1+COV_{DC}^2+COV_{LL}^2}{1+COV_R^2}}}{\left(\dfrac{\lambda_{DC}Q_{DC}}{Q_{LL}} + \lambda_{LL}\right)\exp\left[\beta_T\sqrt{\ln(1+COV_R^2)(1+COV_{DC}^2+COV_{LL}^2)}\right]} \tag{3-16}$$

The Level I calibration is computationally simple and the relative contribution of each variable to the load and resistance factors can be readily identified. Occasionally, this calibration procedure may provide erroneous results if higher derivatives of the random variables contribute significantly to uncertainty but are left out in the simplification. However, for most geotechnical design, higher-order derivatives of the random variables are uncommon or are disregarded because the random variables participate in linear or up to quadratic equations.

Level II. The Level II calibration is an advanced first-order second-moment (AFOSM) procedure (Hasofer and Lind, 1974; Baecher and Christian, 2003). In this procedure, the limit state function (e.g., $M = 0$) is first approximated as a linear function, and M is evaluated for a combination of R and Q at a strategically selected "design point" (labeled Point B on Figure 3-3) The design point is chosen to be on the surface of the joint probability distribution $f(R, Q)$ (shown as contour lines on Figure 3-3) and along the plane defined by the limit condition $M = 0$ (straight dotted line labeled on Figure 3-3) that is tangent to the joint probability surface. In this method, design point B is selected because Point B is at the peak of the bell curve that rises and intersects the $f(R, Q)$ surface and the $M = 0$ plane and thereby has the highest probability of occurrence. The most "probable" occurrence of R and Q is Point A, located at the "highest" point on the surface. However, Point A does not represent a limit state because it is off the $M = 0$ plane. On Figure 3-3, the distance between Points A and B is the reliability index, β.

One key step in this method is to numerically locate Point B, or equivalently, the minimum "distance," β. Numerical

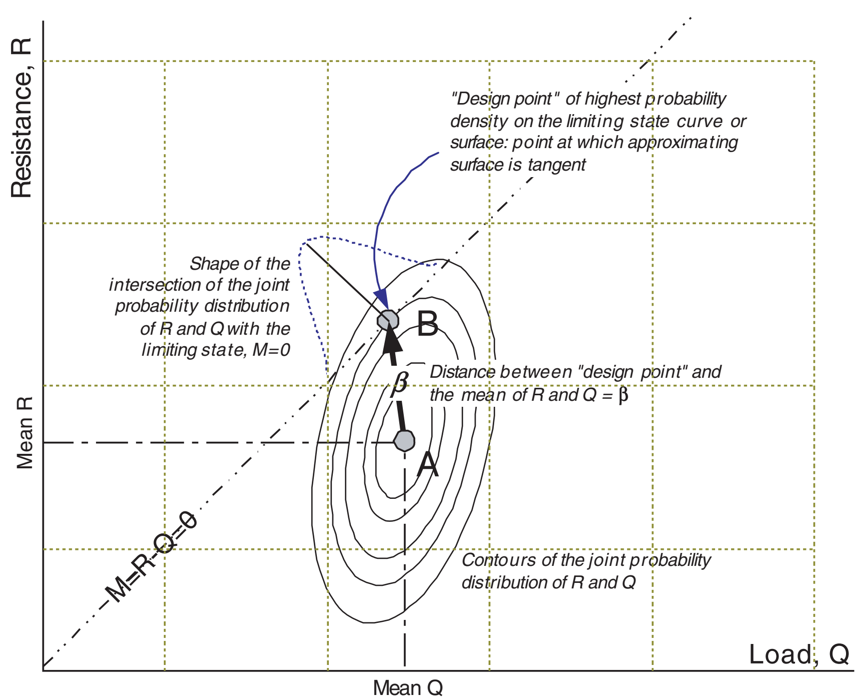


Figure 3-3. Limit state surfaces in the calculation of reliability index.

evaluations that consider iteratively values of the random variables are conducted and the distance β is recalculated until a minimum value of β is found. The iteration starts by assuming an initial value for the distance A-B. A disadvantage of this method is that the computational effort can be significant for certain problems and that a significant volume of data is necessary to develop the joint probability distribution correctly and accurately.

Level III. The Level III calibration represents the highest level of calibration complexity. This level involves formulating the problem with higher-order derivatives of random variables. For most geotechnical applications, however, this method provides relatively small improvements in the accuracy of calculated load and resistance factors when compared to those values provided by Level II calibrations. Therefore, the additional computational effort demanded by this level of analysis generally does not warrant its use.

In this investigation, Method C, Levels I and II, were used.

3.2.7 Steps to Perform the Calibration of Resistance Factors

To perform the calibration of resistance factors, the following steps are taken (Withiam et al., 1997; Allen et al., 2005):

1. Establish the limit state function (i.e., $M = 0$) that explicitly incorporates load and resistance factors, γ and ϕ;
2. Obtain preliminary probability density function (PDF, usually normal or lognormal), cumulative density functions (CDFs), and statistical parameters for random variables *R* and *Q;*
3. Select an acceptable probability of failure, P_f, and a corresponding target reliability index, β_T;
4. Fix load factors in the limit state using statistics or other means;
5. Adjust statistical parameters until there is a best-fit of the CDFs with data points;
6. Perform, in a Monte Carlo simulation, the following steps:
   a. Estimate an initial, trial value for the resistance factor;
   b. Generate random numbers and generate values for *R* and *Q* that extrapolate the existing data; and
   c. Calculate random values of the limit state function, *M;*
7. Using graphical methods or other means, obtain the β value that makes $M = 0$. Compare the calculated β with the target reliability index, β_T; modify the resistance factor and repeat the simulation until the calculated β coincides with β_T. At this point, the final, calibrated resistance factor is obtained.

Each of the previous steps is discussed in the following subsections.

Step 1: Establish a Limit State Function

The limit state function is defined as (Allen et al., 2005):

$$M = R - Q \tag{3-17}$$

where *R* and Q are random variables representing resistance and the maximum load, respectively. A design equation representing Equation 3-17 requires that $\phi_R R_n - \gamma_Q Q_{max} \geq 0$, where ϕ_R is a resistance factor; R_n is a random variable representing the nominal resistance, γ_Q is a load factor, and Q_{max} is a random variable representing the maximum load. When $M = 0$, a non-random value for R_n can be related by the following relation:

$$R_n = \frac{\gamma_Q}{\phi_R} Q_{max} \tag{3-18}$$

Using the previous equation, the general expression (Eq. 3-17) for the limit state function, *M,* can be written as:

$$M = \left(\frac{\gamma_Q}{\phi_R} Q_{max} \right) - Q_{max} \tag{3-19}$$

Note that the two terms in Equation 3-19 that contain Q_{max} are actually two separate random variables, each with different statistical parameters and characterization, and each with both non-random and random components. The quantity Q_{max} as used in the two terms of Equation 3-19 illustrate that the non-random part of the resistance and load random variables can be related. Each of the random variables of Equation 3-19 is generated separately in the Monte Carlo simulation. The simulations are unaffected if the random variables of Equation 3-19 are multiplied or divided by a non-random factor. Therefore, to simplify the calculations, both random variables are normalized by the non-random value Q_{max}, which is equivalent to adopting $Q_{max} = 1$ for the non-random components above (Allen et al., 2005).

Step 2: Develop PDFs and Statistical Parameters for R and Q

In this step, the random variables are assigned a PDF and their statistical parameters are estimated based on existing data. The two most common distributions considered in geotechnical design are normal and lognormal.

If the variable Q_{max} is normally distributed, random values, $Q_{max\,i}$, of this variable can be generated as:

$$Q_{max\,i} = Q_{max\,mean} \left(1 + COV_Q \; z_i \right) \tag{3-20}$$

where

$Q_{max\,i}$ = a randomly generated value of the normal variable Q_{max};

$Q_{max\,mean}$ = mean of Q_{max};

COV_Q = coefficient of variation of the bias of Q_{max};
z_i = standard normal variable, which is the inverse $\Phi^{-1}(u_{ia})$ of the normal function Φ; and
u_{ia} = a random number between 0 and 1 (representing a random probability of occurrence).

In addition, $Q_{max\,mean} = \lambda_Q\, Q_o$, where λ_Q is the normal mean of the bias of Q_{max}, and Q_o is a non-random scaling value.

If the variable Q_{max} is lognormal, random values of this variable can be generated as:

$$Q_{max\,i} = \exp\left(\mu_{ln\,Q} + \sigma_{ln\,Q}\; z_i\right) \tag{3-21}$$

where
$\mu_{ln\,Q}$ = lognormal mean of Q_{max} and
$\sigma_{ln\,Q}$ = lognormal standard deviation of Q_{max}.

The above parameters can be obtained from the normal parameters defined previously as:

$$\mu_{ln\,Q} = \ln\left(Q_{max\,mean}\right) - \frac{\sigma^2_{ln\,Q}}{2} \tag{3-22}$$

and

$$\sigma_{ln\,Q} = \sqrt{\ln\left(COV_Q + 1\right)} \tag{3-23}$$

If the resistance is modeled as a lognormal variable, the first term of Equation 3-19 can be randomly generated as:

$$R_{ni} = \frac{\gamma_Q}{\phi_R}\exp(\mu_{\ln R} + \sigma_{\ln R} z_i) \tag{3-24}$$

where
$\mu_{ln\,R}$ = lognormal mean of R_n;
$\sigma_{ln\,R}$ = lognormal standard deviation of R_n;
z_i = standard normal variable, which is the inverse $\Phi^{-1}(u_{ib})$ of the normal function Φ; and
u_{ib} = a random number between 0 and 1 (representing a random probability of occurrence, and being independently generated from u_{ia}).

The above parameters can be obtained from the normal parameters for R_n as:

$$\mu_{ln\,R} = \ln\left(R_{n\,mean}\right) - \frac{\sigma^2_{ln\,R}}{2} \tag{3-25}$$

and

$$\sigma_{ln\,R} = \sqrt{\ln\left(COV_R + 1\right)} \tag{3-26}$$

where
$R_{n\,mean}$ = mean of R_n and
COV_R = coefficient of variation of the bias of R_n.

In addition, $R_{n\,mean} = \lambda_R\, R_o$, where λ_R is the normal mean of the bias of R_n and R_o is the non-random scaling value defined previously.

Step 3: Select Target Reliability Index

Target reliability indices are selected based on the type of structure, importance of structure (i.e., related to consequences of failure), and the structural redundancy. Structural redundancy refers to the ability of a structure to transfer loads to other members if one of its supporting members fails. Target reliability indices typically range between 2 and 3 for typical geotechnical design (Barker et al., 1991). Allen et al. (2005) recommend selecting β_T close to 2, the lower end of the typical range, when the structural component is not critical or it is redundant, and close to 3, the upper end of the range, when the structural component is critical or it is non-redundant.

Zhang et al. (2001) suggested that it is acceptable to assign to individual structural elements participating in a group a probability of failure that is higher than that of the group. Allen et al. (2005) suggested that an individual element of a substructure can be considered redundant if the reliability index of the entire system is significantly lower (i.e., 0.5 lower) than that of individual components. This situation may occur in geotechnical systems that rely on numerous structural elements (e.g., various layers of geosynthetic or steel reinforcement in a retaining structure or various driven piles in a pile group). Systems with various structural elements tend to have greater structural redundancy and thereby result in a higher overall reliability index than systems with few resisting elements. For example, a pile group is significantly more redundant than a single drilled pile. This concept will be applied to SNWs, as discussed in the following paragraph.

Resistance factors for shallow foundations have been calibrated using $\beta_T = 3.0$ (corresponding to $P_f = 0.14\%$, a relatively low value), as these systems are not highly redundant (Baker et al, 1991). Resistance factors for deep foundations have been calibrated for $\beta_T = 2.33$ (corresponding to $P_f = 1\%$), as driven piles and drilled shafts are typically installed as part of pile/shaft groups (Paikowsky et al., 2004) and thereby carry some structural redundancy. D'Appolonia (1999) used $\beta_T = 2.50$ to calibrate resistance factors for pullout in geogrids, which is a system that tends to be redundant as multiple reinforcement layers are installed with a typical vertical spacing of 1 to 1.5 ft. Allen et al. (2001) adopted $\beta_T = 2.33$ for the calibration of pullout resistance factors in MSE walls.

Step 4: Establish Load Factors

An estimate of the load factor needs to be performed to evaluate whether the load factors [typically those used in AASHTO (2007)] are applicable or whether different load factors need to be proposed.

Allen et al. (2005) present the following equation to estimate the load factor when load statistics are available:

$$\gamma_Q = \lambda_Q \left(1 + n_\sigma \, COV_Q\right) \tag{3-27}$$

where

γ_Q = load factor;
λ_Q = mean of the bias for the load Q;
COV_Q = coefficient of variation of the load bias (i.e., measured-to-predicted ratio for loads); and
n_σ = number of standard deviations from the mean of Q.

This procedure is approximate and is valid for any CDF. The greater the selected value of n_σ is, the lower the probability will be that the measured loads exceed the nominal load. Typically, the number of standard deviations of the load bias is selected at $n_\sigma = 2$, which results in a probability of approximately 2% for the factored load values (Allen et al., 2005) to exceed the nominal load. This procedure is currently used in the AASHTO *LRFD Bridge Design Specifications* and in the Ontario Highway Bridge Design Code (as referenced in Nowak, 1999; Nowak and Collins, 2000). It is recognized that this procedure is based on judgment and not necessarily on a rational procedure (Allen et al., 2005).

Step 5: Best Fit Cumulative Density Functions to Data Points

The selected CDFs for load and resistance must be fitted to the data points to assess the adequacy of the selected CDFs and their statistical parameters. The CDF for loads, which is plotted as variate, must be compared to the lower tail of the load data point distribution. Conversely, the CDF for resistance must be compared to the upper tail of the resistance data point distribution. Finally, both load and resistance approximations should be plotted side by side and compared.

Step 6: Conduct Monte Carlo Simulation

A Monte Carlo simulation is a statistical procedure used to artificially generate many more values of load and resistance than are available from measured data points. Therefore, this technique can be used to extrapolate the data at both ends of the distribution.

In a Monte Carlo simulation, random numbers are generated independently for each of Q_{max} and R_n, assuming that these variables are statistically independent. New sets of u_{ia} and u_{ib} are generated a minimum of 10,000 times to calculate new values for $Q_{max\,i}$ and $R_{n\,i}$ and to develop complete distributions of these random variables. As Q_{max} and R_n are either normal or lognormal, closed form solutions may be obtained for the CDFs of the limit state.

Step 7: Compare Computed and Target Reliability Indices

Following a cyclic calculation scheme, computed and target reliability indices are compared at the end of each Monte Carlo simulation. The iteration is stopped when the difference between the computed and target reliability indices is negligible.

3.3 Review of Current U.S. Soil-Nailing Practice

3.3.1 Introduction

In this section, the results of a review of current U.S. practice of soil nailing are presented. The results of the review are presented as descriptions of the most significant construction steps of SNWs and the main components of an SNW. While this section presents a summary of the review, more detailed information of construction aspects and SNW elements are contained in Appendix B. After the main components of a SNW are identified in this section, a discussion is presented of the limit states to be considered in the design of SNWs based on the LRFD method. For each of these limit states, a description of key variables participating in the limit state equation is provided.

3.3.2 Basic Description of Soil Nail Walls

SNWs are earth-retaining structures constructed using passive reinforcing elements, referred to as soil nails. The term "passive" is used because soil nails are typically not post-tensioned. SNWs are constructed using "top-down" methods, where excavation lifts are created and reinforcing elements are installed after each lift excavation sequence. Soil nails are installed in each excavation lift to provide lateral support to the soil exposed in each excavation level. As each excavation lift is commonly 5 ft deep, nails are installed at a vertical spacing of approximately 5 ft. Soil nails are commonly installed with a horizontal spacing of 5 ft also.

In U.S. soil-nailing practice, after a lift is excavated, holes (commonly known as "drill-holes," regardless of whether they are drilled or driven) are created on the exposed excavation. Drill-holes are created typically by drilling at an inclination of approximately 15 degrees from the horizontal; then, soil nails are inserted into the holes, and the annulus between the drill-hole and nails is filled with grout. Finally, a facing layer of reinforced shotcrete is applied over the protruding nail heads at the face of the excavation. This cycle is repeated for each subsequent lift of excavation. Appendix B presents detailed information of other aspects of SNW construction, including contractor's qualifications, information on suitable methods to store and handle various materials used in SNW construction, nail installation, grouting, and soil nail testing.

A more detailed description of construction aspects related to SNWs is presented in Byrne et al. (1998) and Lazarte et al. (2003).

Soil nails and the facing layer both contribute to excavation stability. While soil nails provide support to the soils retained behind the wall, the facing provides connectivity and structural continuity to nails, thus making the SNW act as a unit.

SNWs have been used successfully in a wide variety of subsurface conditions, including soils and rocks. Although nails are used in soil and weathered rock, the term "soil nail" will be used interchangeably in this document whether the nails are installed in soil or rock. SNWs can be more advantageous than other top-down retaining systems when the construction takes place in granular soils exhibiting some cohesion and/or in weathered, soil-like rock. SNWs are generally unsuitable when they are built below the groundwater table. Additional information related to favorable and unfavorable subsurface conditions for constructing SNWs are presented in Byrne et al. (1998) and Lazarte et al. (2003).

In transportation projects, including those involving bridge substructures, SNWs are routinely used as permanent structures having a minimum design service life of 75 years per AASHTO (2007). SNWs that are built as temporary structures [i.e., service life up to 36 months per AASHTO (2007)] are routinely used in urban settings for shoring up temporary excavations. However, the use of SNWs as temporary earth-retaining systems in bridge substructures is uncommon. This document focuses on SNWs used as permanent structures.

The practice of SNWs varies throughout the United States, particularly in non-public projects. SNW practice differing from that described in this document may include the use of different nail types (e.g., hollow steel bars as opposed to solid bars), different nail materials (e.g., synthetic materials instead of steel), and novel construction procedures. However, none of these variations are discussed in this document.

3.3.3 Main Components of Soil Nail Walls

The main components of SNWs used in typical U.S. practice are identified on Figure 3-4. These components are described in the following paragraphs. Additional information on SNW components is contained in Appendix B. In addition, a more detailed description of typical components of SNWs is presented in Byrne et al. (1998) and Lazarte et al. (2003).

3.3.3.1 Steel Bars

Reinforcing soil nails are solid steel bars. The bars develop tensile stresses in response to the outward deformation of soils that are retained in each excavation lift. Soil movement can occur during excavation or after excavation when external structural loads (e.g., weight of superstructure) are applied. Steel bars used in SNWs are threaded and, as mentioned earlier, are not commonly post-tensioned. In some cases, however, the upper rows of soil nails are post-tensioned as a means to control and limit the outward movement of the wall. Other elements commonly used in connection with the soil nail bars are centralizers and bar couplers (not shown in Figure 3-4, see additional descriptions in Appendix B).

3.3.3.2 Facing System

Facing systems typically consist of temporary and permanent facing. Temporary facing is applied on the exposed soil as each lift is excavated to provide temporary stability. Permanent facing is applied over the temporary facing to provide architectural finish and structural continuity. Temporary facing most commonly consists of reinforced shotcrete. The reinforcement used in the shotcrete usually consists of (i) welded wire mesh (WWM), which is installed over the entire facing; (ii) additional horizontal bars (commonly called "waler bars") that are placed around nail heads; and (iii) additional vertical bearing bars that are also placed around nail heads (see bottom of Figure 3-4). Permanent facing may consist of cast-in-place (CIP) reinforced concrete, reinforced shotcrete, or precast concrete panels.

3.3.3.3 Grout

Grout used in SNWs may consist of a mixture of neat Portland cement mortar or fine aggregate, cement, and water. Grout typically covers all the length of the steel bars, transfers tensile stresses from the bars to the surrounding soil, and provides corrosion protection to the bars. Grout is commonly applied in the drill-holes under gravity using the tremie method.

3.3.3.4 Components at the Soil Nail Head

To provide connection between nails and facing at the protruding soil nail heads, connecting components are installed at this location. These components typically consist of nut, washers, bearing plate, and headed-studs or anchor bolts. The headed-studs or anchor bolts are attached to the bearing plate. Additional descriptions of nail head components are provided in Appendix B.

3.3.3.5 Drainage System

A drainage system is typically installed behind the SNW facing to collect groundwater occurring behind the facing and to convey it away from the wall. The most commonly used drainage system consists of composite, geosynthetic drainage strips, which are also referred to as geocomposite sheet drains

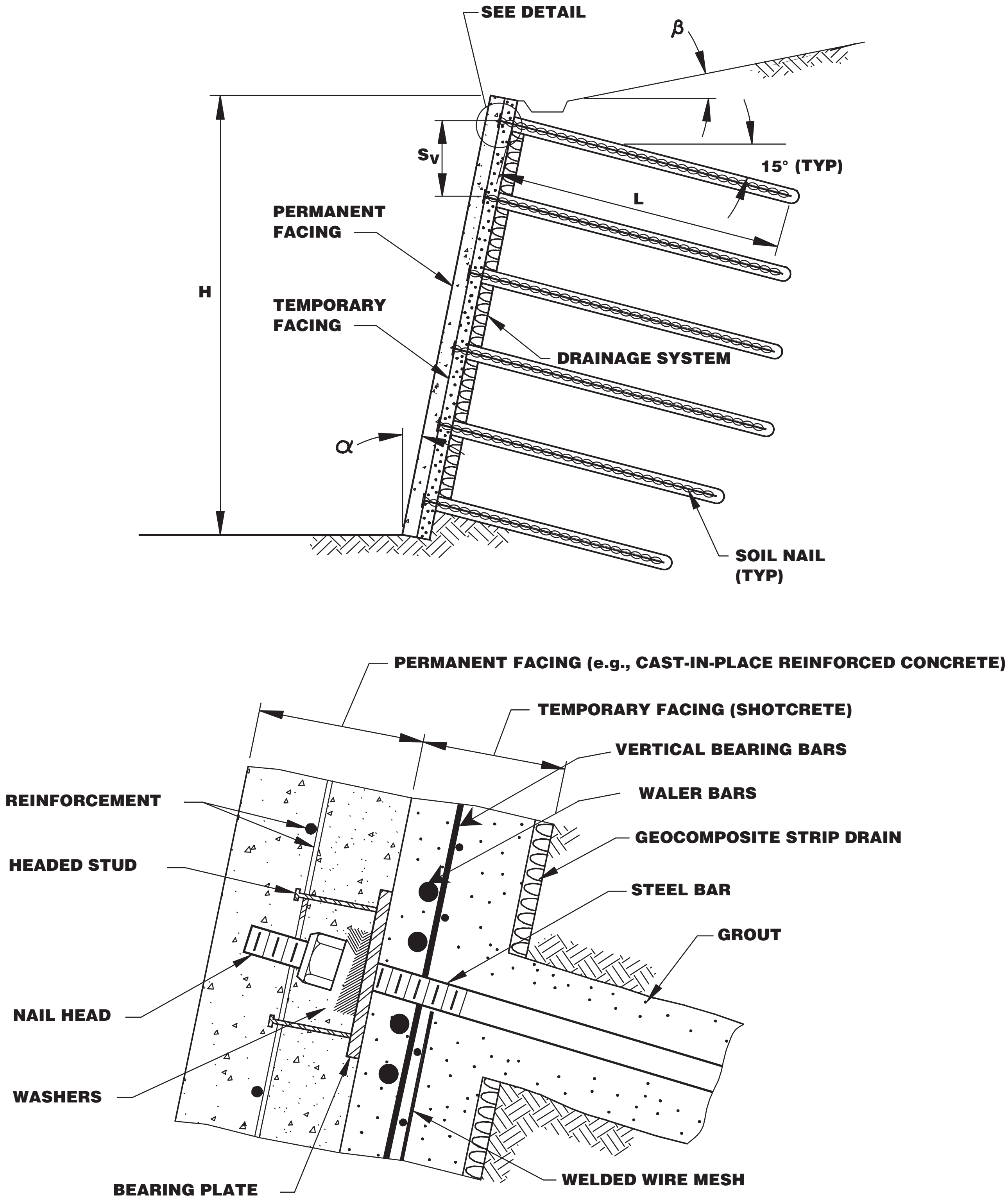


Figure 3-4. Typical cross section of a soil nail wall in common U.S. practice.

(see Appendix B). Drainage strips arrive at the site in rolls from the factory. Strips are unrolled vertically against the exposed face of each excavation lift; subsequently, shotcrete is applied over the drains and exposed soil. In the next excavation lift, more material is unrolled and is extended to the bottom of the excavation. Underdrains made of perforated plastic pipe may be also installed to collect and reroute groundwater accumulating at the SNW base water from the wall [see additional details in Appendix B and Lazarte et al. (2003)].

3.3.3.6 Corrosion Protection

Soil nails in permanent structures require chemical and/or physical protection (the latter referred to as encapsulation) from corrosion. The required level of corrosion protection increases as site conditions become more aggressive. In U.S. practice, the lowest level of corrosion protection is provided by the cement grout alone. If the grout mix is appropriately designed and suitable grouting techniques are applied, grout can provide adequate protection in noncorrosive to mildly corrosive environments. Higher levels of corrosion protection are required in permanent, more corrosive environments.

Higher levels of corrosion protection can be achieved by grouting the soil nail bars in a phased process that involves providing the bars with the first level of protection under controlled conditions. In this procedure, the bars are first inserted in a protective sheath consisting of corrugated high-density polyethylene (HDPE) or polyvinyl chloride (PVC) pipe. Then, the annulus between the sheath and bar is filled with grout and cured under controlled conditions at the shop. After the grout is fully cured, the sheathed bar is shipped to the site and placed in the drill-hole. Additional grout is pumped into the annulus between the sheathing and the drill-hole. Due to the two layers of grout that are in place, this system is usually referred to as double-corrosion protection level.

Corrosion protection also can be increased by using fusion-bonded, epoxy-coated bars, instead of bare bars. The combined use of epoxy-coated bars, sheathing, and final grout provides the highest level of corrosion protection. Other aspects of corrosion measures are addressed in Appendix B. A more detailed description of corrosion protection used in SNW applications is provided in Lazarte et al. (2003).

3.3.3.7 Other Elements and Materials

Other elements and materials used in the construction of SNWs include protection film, additives for shotcrete and grout, and fittings. Additional information on these elements is provided in Appendix B and in Lazarte et al. (2003).

3.4 Limit States in Soil Nail Walls

3.4.1 Introduction

Various SNW components including nails, facing, and nail head connectors contribute to stability and structural performance. As a result, every potential limit state involving these elements should be considered according to the design philosophy of LRFD. Each of the limit states identified for SNW design is addressed in the following sections. The terminology used herein regarding overall stability and strength limit states of SNWs is selected to be consistent with the terminology used in Section 11 of the LRFD Specifications. This terminology differs slightly from that used in Byrne et al. (1998) and Lazarte et al. (2003); however, the principles behind these limit states are similar in all of these publications.

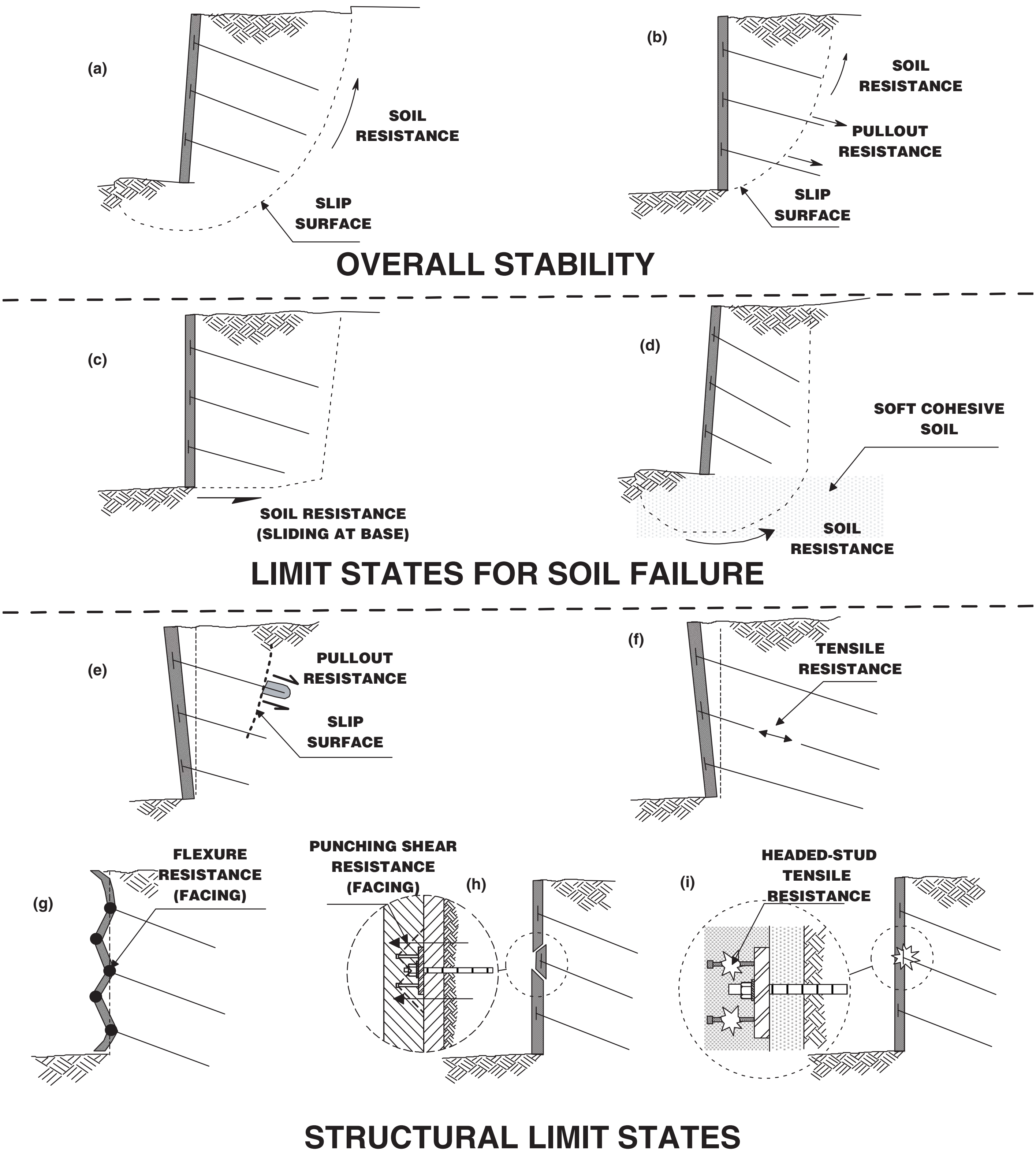
The following limit states are considered for SNW design:

- Service limit states:
  - Overall stability [Figures 3-5(a) and 3-5(b)];
  - Wall lateral displacement;
  - Wall settlement; and
  - Lateral squeeze.
- Strength limit states:
  - Safety against soil failure, including:
    - Sliding stability [Figure 3-5(c)] and
    - Basal heave [Figure 3-5(d)].
  - Structural limit states, including:
    - Nail pullout [Figure 3-5(e)];
    - Nail in tension [Figure 3-5(f)];
    - Facing structural limit states, including:
      - Flexure [Figure 3-5(g)];
      - Punching-shear [Figure 3-5(h)]; and
      - Headed-stud in tension [Figure 3-5(i)].

Extreme-event limit states for SNWs are commonly limited to those arising from seismic loads. Fatigue limit states, which are uncommon in the design of SNWs, are not addressed in this document.

For most practitioners, the consideration of overall stability as a service limit state may not be intuitive and may appear to be incorrect. However, this selection is necessary because load factors used in this state are equal to 1.0 in the current *LRFD Bridge Design Specifications* (AASHTO, 2007). This approach for overall stability may be modified in future editions of the AASHTO *LRFD Bridge Design Specifications;* therefore, appropriate changes should be also made for SNWs. In Section 3.5, more detailed discussions of overall stability in LRFD are presented.

Considering basal heave a service limit state is not intuitive either. However, because the load factors for basal heave are also 1.0, basal heave is considered a service limit state in this document, in order to be consistent with the current *LRFD Bridge Design Specifications* (AASHTO, 2007) approach.



Source: Modified after Lazarte et al. (2003)

Figure 3-5. Limit states in soil nail walls: overall stability: (a) slip surface not intersecting nails and (b) slip surface intersecting nails; soil failure: (c) sliding at base and (d) basal heave; and structural: (e) pullout, (f) nail in tension, (g) flexure of facing, (h) punching-shear in facing, and (i) headed-stud in tension.

However, sliding stability is considered a strength limit state as it is a "safety against soil failure" case, per Section 10, Foundations, of AASHTO (2007).

The limit states listed previously are discussed in the following sections.

3.4.2 Service Limit States

3.4.2.1 Overview

Service limit states related to a stability condition (i.e., overall stability and basal heave) are described in this section. Service limit states related to deformations under regular service conditions are described subsequently in Section 3.4.6.

3.4.2.2 Overall Stability

Overall stability of SNWs [shown schematically in Figures 3-5(a) and (b)] must be considered when a potential slip surface extends through the soil under and behind the wall and through some or all nails. If the slip surface does not intersect the nails [Figure 3-5(a)], the soil shear resistance mobilized along slip surfaces is the only contribution to stability. Soil resistance can be frictional, cohesive, or both, depending on the soil type and/or loading conditions (e.g., drained or undrained loading). If the slip surface intersects some or all nails, the nail pullout resistance mobilized in the soil nails behind the slip surface also contributes to stability. The nail tensile resistance is treated separately as a structural strength limit state, as discussed in Section 3.4.4.3.

In the ASD method, the verification of overall stability safety includes the use of a factor of safety, which is derived as a ratio between resisting and destabilizing forces or moments. In the LRFD framework, the safety for overall stability must be verified by demonstrating that the factored nominal resistances are greater than or equal to the overall effect of the factored loads. If the loads have a destabilizing effect, as most external loads do, load factors applied to these loads are greater than 1.0. If the acting loads have a stabilizing effect (e.g., passive earth pressures provided by berm at the wall toe resisting the outward SNW movement), the load factors applied to these loads are less than or equal to 1.0.

Overall stability of SNWs is commonly evaluated using procedures based on two-dimensional, limit-equilibrium methods used in traditional stability analyses. Similar to the stability analyses of slopes, in limit-equilibrium stability analyses of SNWs, several potential slip surfaces are considered and an *FS* is calculated for each case. The analysis is repeated until the surface with the lowest calculated *FS* is found. The lowest calculated *FS* must be equal to or greater than the minimum acceptable *FS* established for the structure and condition.

Various shapes of the slip surface have been considered in SNW design procedures, including (i) planar (Sheahan et al., 2003); (ii) bi-linear (Stocker et al., 1979; Caltrans, 1991); (iii) parabolic (Shen et al., 1981); (iv) log spiral (Juran et al., 1990); and (v) circular (Golder, 1993). A comparison of *FS* results obtained with different SNW design procedure and slip surfaces indicates the slip surface shape selection does not seem to affect significantly the calculated *FS* (Long et al., 1990).

Stability analyses for SNWs are commonly performed using computer programs specifically developed for the design of SNWs because these programs give design engineers greater ability to quickly analyze multiple design scenarios for these walls. The most commonly used programs are (i) SNAIL or SNAILZ—free, public-domain programs developed by the California Department of Transportation (Caltrans, 1991 and 2007, respectively)—and (ii) GOLDNAIL (Golder, 1993), a commercial program. Alternatively, simplified methods consisting of design charts (e.g., Byrne et al., 1998; Lazarte et al., 2003) can also be used in preliminary designs. General slope stability computer programs having the ability to model multilevel reinforcement can also be used to assess SNW stability.

Manual calculations of stability are rarely performed in real practice. However, the following paragraphs illustrate the manner in which forces participating in a typical SNW problem are considered in the assessment of overall stability using the LRFD methodology (Figure 3-6), where a hypothetical slip surface intersects all nails. Figure 3-6 shows a generic SNW of height *H* and face batter angle α from the vertical. The ground surface slopes at angle β behind the wall; nails are inclined at angle *i* from the horizontal. Loads consist of an external surcharge per unit width, *Q*, and the vertical earth load, *EV* [i.e., symbol used per Table 3.4.1-2 of AASHTO (2007)]. The slip surface selected in this simplified analysis is planar with an inclination, ψ, from the horizontal. This selection does not affect the validity of this procedure. R_S is the nominal soil resistance per unit width (or alternatively, per nail horizontal spacing, S_H) mobilized along the slip surface. *T* is the sum of the nominal pullout resistance of all soil developing behind the slip surface.

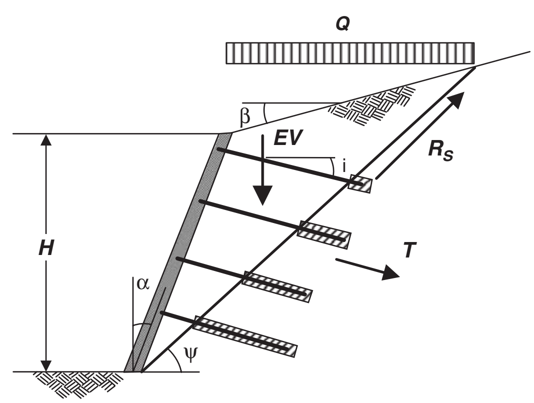


Figure 3-6. Main forces in overall stability.

Overall stability is achieved when the force components acting parallel to the failure plane meet the following requirement:

$$\sum \text{Factors} \times \text{Nominal Resistance} \geq \sum \text{Factors} \times \text{Destabilizing Forces} \qquad (3\text{-}28)$$

The factored nominal resistance is:

$$\sum \text{Factors} \times \text{Nominal Resistance} = \phi_s R_s + \phi_{PO}\, T \cos(\psi + i) \qquad (3\text{-}29)$$

where ϕ_s and ϕ_{PO} are resistance factors for soil shear resistance and nail pullout, respectively.

The assumption that T is a resultant force is valid provided that only force-equilibrium is considered. A more rigorous approach would require establishing moment and force-equilibrium conditions simultaneously while considering the distribution of soil nail forces over the wall height.

R_s is assumed to have both cohesive and frictional components and is expressed as:

$$R_s = c\, L_s + F_N \tan\varphi_f \qquad (3\text{-}30)$$

where

c = nominal soil cohesion,
L_S = length of the slip plane,
F_N = normal force per unit width acting on the slip surface, and
φ_f = soil effective friction angle.

The normal force, F_N, is calculated from force equilibrium as:

$$F_N = (EV + Q)\cos\psi + T\sin(\psi + i) \qquad (3\text{-}31)$$

The surcharge load may comprise permanent and transient loads originating from the superstructure. Assuming that only dead loads and live loads are present, the surcharge can be expressed as:

$$Q = Q_{DC} + Q_{LL} \qquad (3\text{-}32)$$

where Q_{DC} and Q_{LL} are the permanent/dead and live loads, respectively.

The factored destabilizing force along the slip plane is calculated as:

$$\sum \text{Factors} \times \text{Destabilizing Forces} = \sum \gamma_{Qi} Q_i = \left[(\gamma_p EV + \gamma_{DC} Q_{DC} + \gamma_{LL} Q_{LL})\sin\psi\right] \qquad (3\text{-}33)$$

where

γ_p = load factor for permanent, vertical earth loads;
γ_{DC} = load factor for dead load; and
γ_{LL} = load factor for live loads. As overall stability is treated as a service limit state (AASHTO, 2007), $\gamma_p = \gamma_{oc} = \gamma_{LL} = 1.0$.

With Equations 3-30 through 3-33, the force, T, that satisfies Equation 3-28 can be calculated to establish subsequently the required nail length. The nail tensile resistance is verified separately, after the maximum load, T_{max}, of all nails is obtained. The facing can be designed (or verified, if dimensions and reinforcement were estimated beforehand) for the maximum nail load.

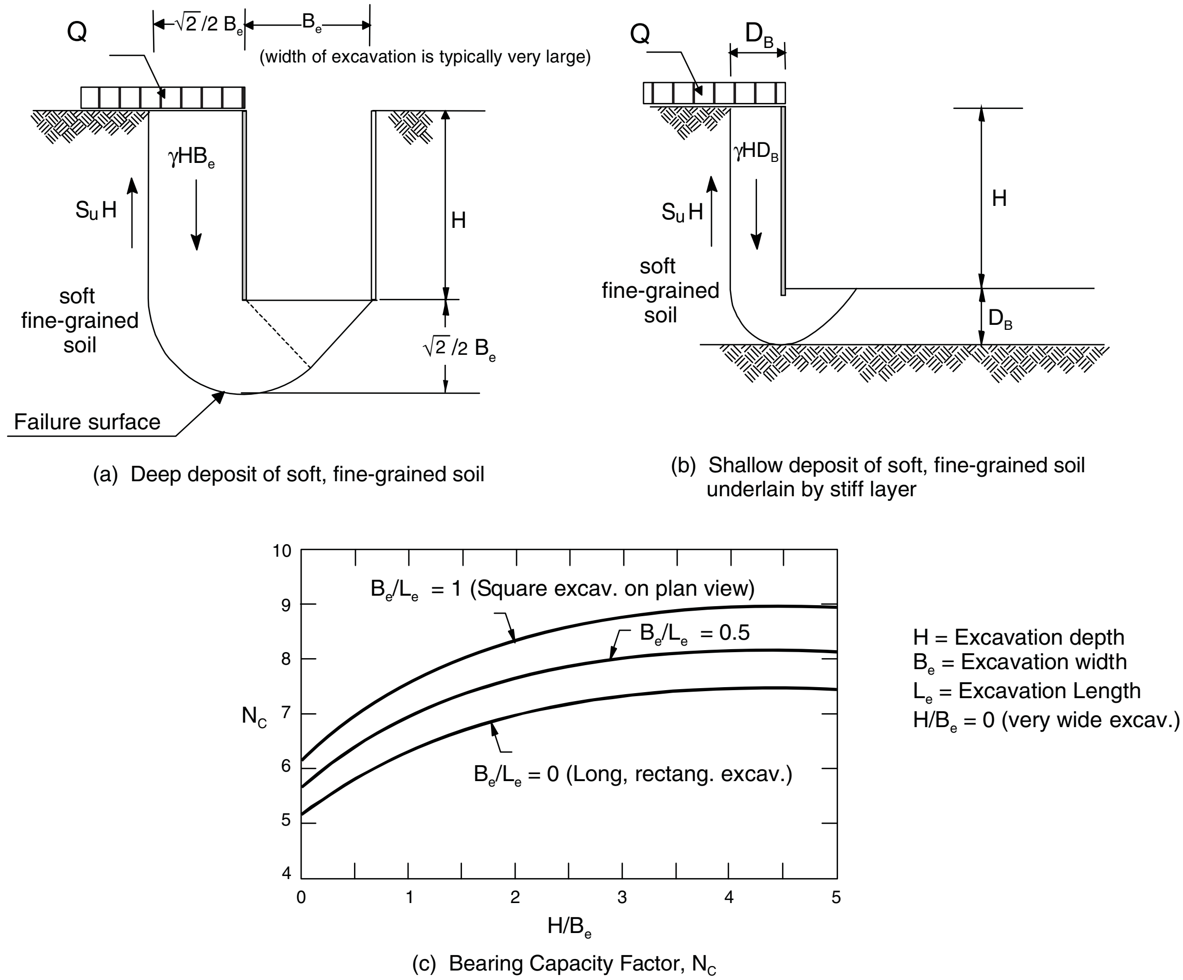
The equations presented above were developed for a single-wedge failure plane but can be extended for two- or three-wedge failure plane cases, which would result in more accurate but complex expressions (e.g., as used in the programs SNAIL and SNAILZ). The procedure above was presented to introduce some key aspects of overall stability analysis; however, as mentioned earlier, manual calculations are uncommon because versatile computer programs (or, alternatively, simplified design charts) are available to perform these calculations more efficiently.

3.4.2.3 Basal Heave

When soft, fine-grained soils exist behind and at the base of an SNW excavation [as illustrated on Figures 3-5(c) and 3-7], the potential for basal heave (i.e., mobilization of bearing resistance) should be evaluated. If the excavation depth is excessive for the existing soft soil conditions, unbalanced loads generated during excavation may cause the bottom of the excavation to heave and possibly cause a basal failure. SNWs may be more susceptible to basal heave than other retaining systems because the facing is usually not embedded. In contrast, soldier piles of anchored retaining walls are embedded a considerable depth and provide some resistance to basal heave. Note that basal heave is not common in SNWs as these structures are not routinely built in or over soft, fine-grained soils. This scenario is considered for completeness of feasible limit states for SNWs.

Basal heave is akin to a bearing resistance limit state and its evaluation should be similar to that of a bearing resistance limit state. One difference is that basal heave may arise over a short period of time and loads are more appropriately considered at the service level. Consequently, load factors are adopted equal to 1.0. In the current *LRFD Bridge Design Specifications* (AASHTO, 2007), basal heave is not specifically treated; however, some guidance is included to assess settlement occurring behind an anchored wall as a service limit state for movement (e.g., see Article 11.9.3, Movement and Stability at the Service Limit State). However, in that article, there are insufficient guidelines to establish whether an excavation in very soft soils is safe or not.

In this section, a methodology is proposed to evaluate cases where the potential instability of the base of the excavation is



Source: Modified after Sabatini et al. (1999)

Figure 3-7. Basal heave.

significant. In this procedure, this scenario is treated as a service limit state, and based on equilibrium. All load factors considered are then $\gamma = 1.0$.

In this limit state, the following requirement must be satisfied:

$$\phi_{BH} R_s \geq \sum_{1}^{N} Q_i \tag{3-34}$$

where

ϕ_{BH} = resistance factor for basal heave (AASHTO, 2007);
R_s = nominal soil shear resistance for basal heave per unit width [acting along the composite slip surface shown on Figure 3-7(a)]; and
Q_i = loads acting at the base of the soil block that may be displaced.

If all of the excavation is in cohesive soils, R_s is calculated as:

$$R_s = S_{u1} H + S_{u2} N_c B_e \frac{\sqrt{2}}{2} \tag{3-35}$$

where

S_{u1} = undrained shear resistance of the fine-grained soil behind the SNW;
S_{u2} = undrained shear resistance of the fine-grained soil below the SNW;
H = height of the wall;
N_c = cohesion bearing resistance factor (e.g., Terzaghi et al., 1996); and
B_e = excavation width.

The volume of soil that may be displaced and cause heave at the bottom of the excavation is controlled by the excavation width, as shown in Figure 3-7(a). In the simplified model of Figure 3-7(a), the width of the soil block that may be displaced is $\left(\sqrt{2}/2\right. B_e = 0.71\, B_e$. For wide excavations, the width of the soil block usually extends behind all nails.

When a deposit of soft, fine-grained or weak soil exists under the excavation with a maximum thickness D_B [Figure 3-7(b)] and a deposit of stiff material underlies the excavation within a depth $D_B \leq 0.71\, B_e$, the width of the heave area at the bottom of the excavation is limited to D_B. Therefore, in Equation 3-35, $\sqrt{2}/2\, B_e$ is replaced by D_B.

N_c depends on the excavation depth, width, and length (L_e) and is a function of the ratios H/B_e and B_e/L_e, as shown in Figure 3-7(c) (Terzaghi et al., 1996). Excavations for SNWs are typically very wide and rectangular (i.e., $L_e >> B_e$ and $B_e >> H$); therefore, it can be conservatively assumed that $H/B_e = B_e/L_e = 0$, which results in $N_c = 5.14$.

If the contribution of the soil resistance along the vertical surface behind the wall is disregarded (a very conservative assumption for most SNWs), the total nominal resistance reduces to:

$$R_s = S_{u2} N_c B_e \frac{\sqrt{2}}{2} \tag{3-36}$$

The sum of all loads at the base of the soil block is:

$$\sum_1^2 Q_i = \frac{\sqrt{2}}{2} H\, B_e \gamma_s + Q_{DC} \tag{3-37}$$

where γ_s is the unit weight of the soil behind the wall and Q_{DC} is the dead load.

The limit state for basal heave at the bottom of the soil block can be also expressed as:

$$\phi_{BH}\, 5.14\, S_{u2} \geq H\, \gamma_s + q \tag{3-38}$$

where $q = Q_{DC} / \left(\sqrt{2}/2\, B_e\right)$.

This expression is similar to one included in Article 11.9.3 of AASHTO (2007). Clear guidelines about a maximum resistance factor (or equivalent minimum "safety factor" in the ASD) for basal heave are not included in AASHTO (2007). In this document, a value of $\phi_{BH} = 0.70$ is proposed.

Neglecting the soil resistance behind the wall and assuming that $Q_{DC} = 0$, the following simplified expression can be used to estimate the minimum required undrained shear resistance of the soil at the base of the excavation to provide sufficient stability:

$$S_{u2} \geq \frac{1}{\phi_{BH}} \frac{H\, \gamma_s}{5.14} \tag{3-39}$$

The above equation can be used as a tool to conservatively estimate excavation depths that would result in safe construction. Therefore, for soft soils [i.e., those commonly classified with an undrained shear strength between 12.5 and 25 kPa (250 and 500 psf)] and assuming $\gamma_s = 17.3$ kN/m^3 (110 pcf), excavation depths of less than approximately 8 ft (for $S_{u2} = 250$ psf) and 16 ft (for $S_{u2} = 500$ psf) would result in safe construction.

3.4.3 Soil Failure Limit States

3.4.3.1 Overview

Strength limit states involving soil failure are generally achieved when the soil nominal resistance is mobilized along a slip surface, including sliding at the base [Figure 3-5(c)]. No other scenario of soil failure is considered for SNWs because overall stability and basal heave (both involving a slip surface) are considered service limit states. The limit state for sliding stability is described in the following paragraphs.

3.4.3.2 Sliding Stability

Sliding is an uncommon limit state for most SNWs and is considered here for completeness. Conceptually, this limit state can be considered a particular case of overall stability. The sliding limit state may arise when the block of reinforced soil is underlain by a weak soil layer (Figure 3-8) that determines the location of a critical slip surface. The procedure presented below can be applied for weak layers that are horizontal to sub-horizontal. For non-horizontal slip planes, alternative procedures (including general slope stability analysis) must be used. Software available in the United States has the capability to simulate lock-type slip surfaces and can thereby be used to evaluate sliding stability where a horizontal weak layer is present. However, the computer programs SNAIL (or SNAILZ) and GOLDNAIL have limited to no capabilities, respectively, to evaluate sliding stability.

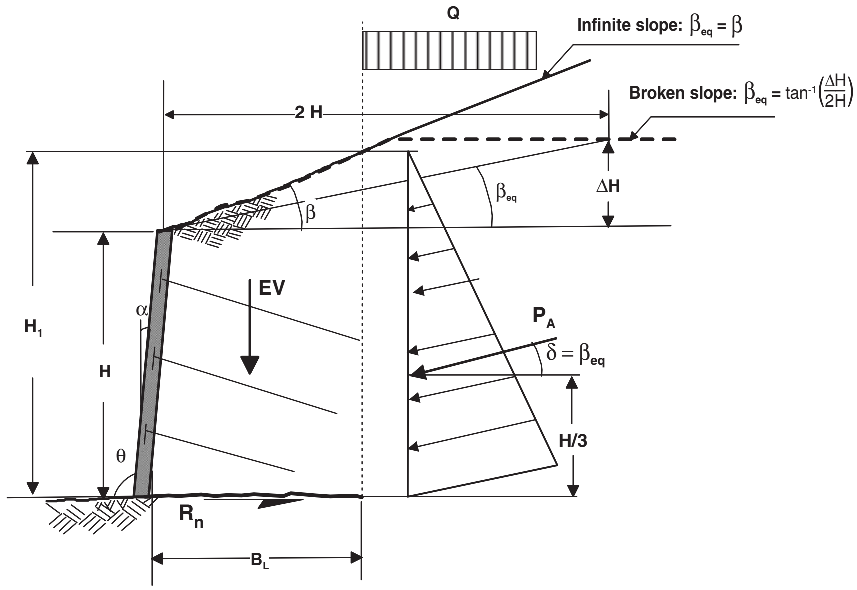
In the procedure presented below, loads caused by lateral earth pressures acting behind the soil block are explicitly considered. Unlike with overall stability scenarios for SNWs, loads in this limit state are assigned load factors ≥ 1.0 because destabilizing effects are clearly separated from stabilizing effects. Lateral earth loads can be evaluated using Rankine or Coulomb theories and by approximating the back surface to a vertical slip surface behind the soil block. The reader is referred to Article 3.11.5.1, Lateral Earth Pressure, of the *LRFD Bridge Design Specifications* (AASHTO, 2007) for additional information.

Sliding is verified using the following expression:

$$\phi_\tau\, R_s \geq \gamma_{EH}\, P_A \cos\delta \tag{3-40}$$

where

ϕ_τ = resistance factor for sliding (AASHTO, 2007);
R_s = nominal soil sliding resistance per unit width acting at the base of the soil block;
γ_{EH} = load factor for horizontal earth loads;
P_A = resultant of the lateral active earth load per unit width [i.e., designated as *EH* in Table 3.4.1-2 of AASHTO (2007)]; and



Source: Lazarte et al. (2003)

Figure 3-8. Sliding stability of a soil nail wall.

δ = inclination of the lateral earth load (typically assumed to be equal to the backslope angle, β_{eq}).

Based on recommendations presented in Section 11 of AASHTO (2007) for the verification of sliding limit states, any external load, Q, acting behind the retaining structure must be considered to extend outside the block of soil, i.e., up to the vertical dashed line shown in Figure 3-8. The nominal soil sliding resistance can be calculated as:

$$R_s = c\,B_L + \left(EV + P_A \sin\beta_{eq}\right)\tan\varphi_{fb} \qquad (3\text{-}41)$$

where

- c = the cohesive resistance of the soil at the base of the block of soil,
- B_L = the base length (considered herein a horizontal slip surface),
- EV = the weight of the soil block,
- P_A = the resultant of the lateral active earth load per unit width,
- β_{eq} = the equivalent angle of the backslope, and
- φ_{fb} = the effective friction angle at the base of the soil block.

External loads, Q, occurring behind the soil block must be taken into account as added lateral loads. Additional details to calculate the effect of these loads can be found in Article 3.11.5.1 of AASHTO (2007).

If the slope has no breaks within a horizontal distance $2H$ from the wall (e.g., ground surface shown as a solid line on Figure 3-8), the slope is considered "infinite" and $\beta_{eq} = \beta$. If the slope exhibits a slope break within a distance $2H$ from the wall (e.g., ground surface shown as a dashed line on Figure 3-8), the slope is assigned an equivalent inclination angle $\beta_{eq} = \tan^{-1}(\Delta H/2H)$, where ΔH is the slope rise over a distance $2H$ (Figure 3-8).

The design engineer must select c and φ_{fb} depending on soil drainage conditions (i.e., "free-draining" or "undrained" conditions) and possibly other conditions (e.g., cemented or uncemented soil). Depending on the nature of the soil under the wall, residual values for φ_{fb} may be used. The passive resistance generated in front of an SNW is disregarded because, in common practice, either SNW facings are not embedded or the embedment depth of an SNW is small. In principle, the resistance factor for sliding, ϕ_τ, could be selected differently whether drained or undrained conditions are prevalent because strength parameters for drained/undrained conditions are based on tests and models commonly producing different errors and uncertainties (e.g., Baecher and Christian, 2003). However, this practice is not yet included in AASHTO (2007).

The active force per unit width can be estimated as:

$$P_A = \frac{\gamma_s H_1^2}{2} K_A \qquad (3\text{-}42)$$

where

- γ_s = the unit weight of the soil behind the wall,
- H_1 = the effective height over which the earth pressure acts, and
- K_A = the active earth pressure coefficient for the soil behind the wall (can be estimated using the Coulomb or Rankine formulations).

H_1 is calculated as:

$$H_1 = H + (B_L - H\tan\alpha)\tan\beta_{eq} \tag{3-43}$$

where α is the wall face batter angle.

3.4.4 Structural Limit States

3.4.4.1 Introduction

Structural limit states (occasionally also referred to as internal limit states) arise when the nominal resistance is reached in structural elements of an SNW (i.e., bars, shotcrete, reinforcement, and other elements in the facing system). The five structural limit states considered for SNWs [shown schematically on Figure 3-5(e) through (i)] include:

- Nail pullout,
- Nail in tension, and
- Facing limit states (three different limit states).

In general, the tensile force of a nail varies along its length. Figure 3-9 shows a schematic distribution of tensile force along the nail. The magnitude of this force at a distance, *x*, from the bar end is represented by *T(x)*. *T(x)* increases from 0 at $x = 0$, to a maximum value, T_{max}, somewhere in the middle section of the nail, and then decreases to a value T_o at the facing. The maximum value, T_{max}, is used in evaluations of the pullout and tension limit states. In contrast, the nail load at the wall facing, T_o, is used to evaluate the facing limit states. Nominal pullout, tension, and facing resistances (i.e., herein identified as R_{PO}, R_T, and R_F) must be greater than T_{max} or T_o.

Byrne et al. (1998) proposed a model that illustrated the contribution of each resistance into the resistance of the nail. In this model (Figure 3-9), the pullout resistance increases from the distal end of the nail up to the location of the slip surface. The tension and facing resistances are also illustrated on Figure 3-9.

The value T_{max} is generally obtained from the output of overall stability analysis using SNAIL, SNAILZ, or GOLDNAIL or can be estimated using simplified methods (Byrne et al., 1998; Lazarte et al., 2003). Note that T_{max} values are a function of the load factor used in the analysis. However, T_{max} does not represent service conditions. For most cases of wall geometry and external load conditions, $T_{max\text{-}s}$ (service conditions) can be estimated from data presented by Byrne et al. (1998), as:

$$T_{max\text{-}s} = 0.70 \text{ to } 0.80\, K_A \gamma_s H\, S_H\, S_V \tag{3-44}$$

where S_V and S_H are the vertical and horizontal nail spacing, and K_A, γ_s, and H are as defined previously. Equation 3-44 is based on the analysis of monitoring results of SNWs under normal, working conditions (Byrne et al., 1998).

The force $T_{o\text{-}s}$ (service conditions) is estimated from $T_{max\text{-}s}$ with (Clouterre, 1991 and 2002):

$$T_{o-s} = T_{max-s}\left[0.6 + 0.05\left(S_{max}[\text{in feet}] - 3\right)\right] \le T_{max-s} \tag{3-45a}$$

$$T_{o-s} = T_{max-s}\left[0.6 + 0.2\left(S_{max}[\text{in m}] - 1\right)\right] \le T_{max-s} \tag{3-45b}$$

where S_{max} is the greater of S_V and S_H. In addition, based on the instrumentation of soil nails in various in-service SNWs, the following range for $T_{o\text{-}s}$ can be used (Byrne at al., 1998):

$$T_{o\text{-}s} \ge 0.60 \text{ to } 0.70\, K_A \gamma_s H\, S_H\, S_V \tag{3-46}$$

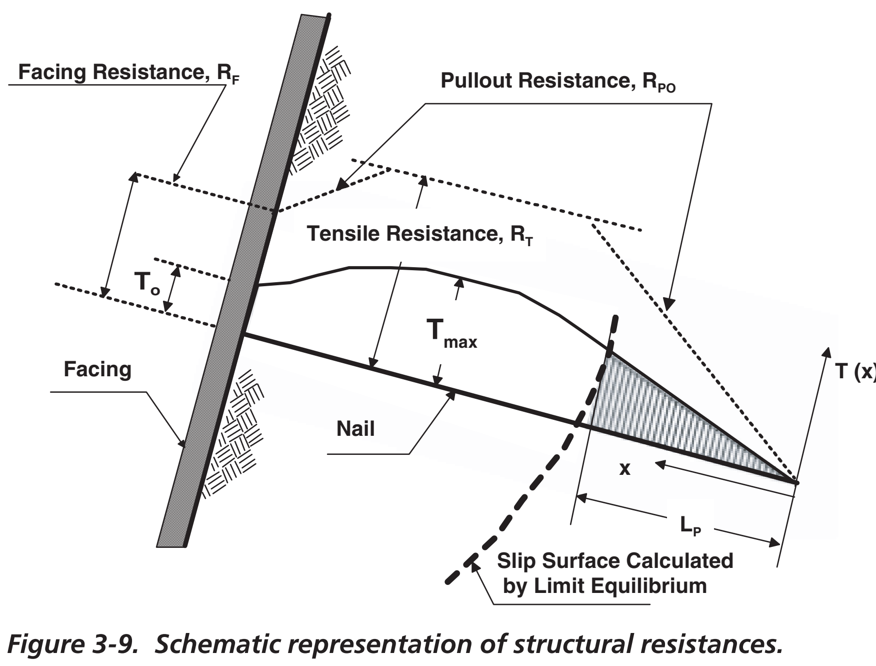


Figure 3-9. Schematic representation of structural resistances.

3.4.4.2 Pullout Resistance

An adequate level of pullout resistance [Figure 3-5(e)] developing along the soil-grout interface is necessary for overall stability. The pullout resistance along a length L_p (shaded area in Figure 3-9) contributes to stability and is mobilized behind the slip surface, as calculated in a limit-equilibrium stability analysis. The nominal unit pullout resistance, r_{PO}, (also referred to as load transfer rate) has units of force per unit length and is expressed as:

$$r_{PO} = \pi q_U D_{DH} \tag{3-47}$$

where

q_U = the nominal bond resistance of the nail/soil interface (with units of force per unit area) and

D_{DH} = the diameter of the drill-hole.

Actual distributions of bond stresses along the grout-soil interface can be complex and may exhibit significant variations along the nail. However, to simplify calculations, the distribution is commonly assumed to be constant along the pullout length; therefore, the nominal bond resistance q_U is considered an apparent, average value. For a given pullout length, L_P, occurring behind the slip surface, the resulting nominal pullout resistance, R_{PO}, is:

$$R_{PO} = r_{PO} L_P \tag{3-48}$$

Adequate nail pullout resistance is provided when:

$$\phi_{PO} R_{PO} \geq T_{max} \tag{3-49}$$

where ϕ_{PO} is the resistance factor for pullout resistance and T_{max} is the maximum tensile force on the bar, as calculated in stability, limit-equilibrium analyses. Note that this force is not a service load. Therefore, the required nail length behind the slip surface must be:

$$L_P \geq \frac{T_{max}}{\phi_{PO}\, \pi q_U D_{DH}} \tag{3-50}$$

Additional information regarding the bond resistance of soil nails is presented subsequently.

3.4.4.3 Tensile Resistance of Nails

An adequate nominal tensile resistance of a nail bar [see Figure 3-5(f)] must be established by verifying that:

$$\phi_T R_T \geq T_{max} \tag{3-51}$$

where

ϕ_T = the resistance factor for nail tension;

R_T = the nominal tensile resistance of the nail bar; and

T_{max} = the maximum tensile force on the bar, as calculated in limit-equilibrium analyses. As mentioned earlier, this force is not a service load.

The nominal tensile resistance of a nail bar is:

$$R_T = A_t f_y \tag{3-52}$$

where

A_t = the nail bar cross-sectional area, and

f_y = the bar nominal yield resistance (i.e., with units of force per square area).

The tensile resistance provided by the grout is disregarded.

3.4.4.4 Facing Strength Limit States

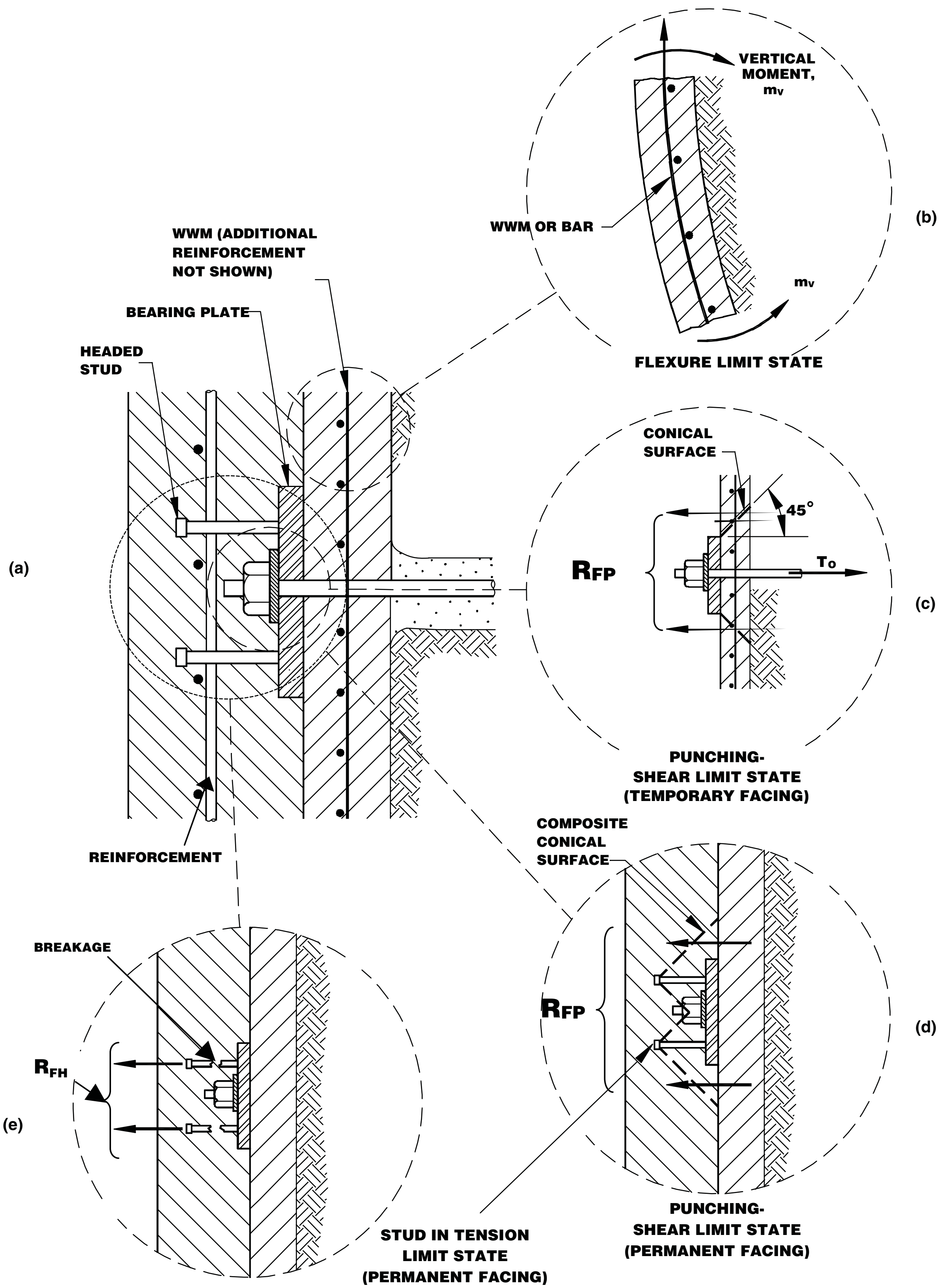
Facing strength limit states [shown schematically on Figure 3-5(g), (h), and (i)] are those affecting the shotcrete, shotcrete reinforcement (bars or WWM), bearing plate, and connectors at the nail head (Figure 3-10). The most common facing strength limit states include:

- Flexure (or bending),
- Punching-shear, and
- Headed-stud in tension.

These limit states are described in the following subsections.

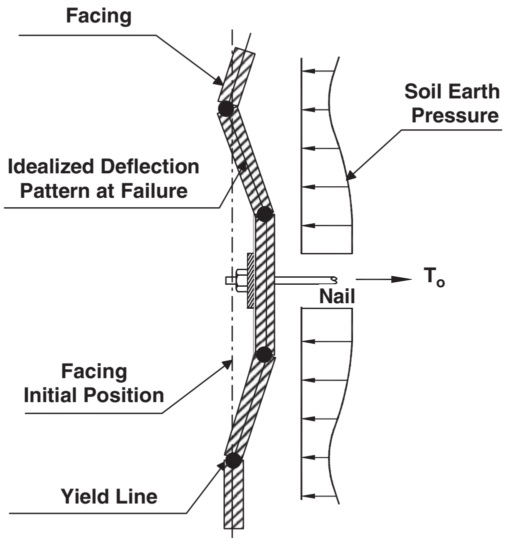
Flexure in Facings. Lateral earth pressures acting against the facing cause flexural or bending moments in the facing. For the purposes of this limit state, the facing can be considered to be a continuous two-way slab and the nails can be considered to be the supports of the slab. A flexural/bending limit state may be reached when the lateral loads increase, progressively deform the facing, form cracks, and ultimately produce a collapse mechanism (Figure 3-11). Moments on the facing produce tension on the outside of the facing between nails (i.e., conventionally, these are positive sign moments) or can generate tension on the inside of the facing around the nails (i.e., negative moments). Moments occur around a horizontal axis [i.e., vertical moments, m_V, as shown on Figure 3-10(b)] and a vertical axis (i.e., horizontal moments, m_H). Therefore, separate flexural resistances develop at two locations: the mid-span section between nails and the section around nails, with each section considered both along the horizontal and vertical directions. Therefore, four conditions must be evaluated. The locations where the reinforcement is computed are presented in Figure 3-12.

In SNWs, flexural resistance depends on several factors, including horizontal and vertical nail spacing; bearing plate size; facing thickness, *h;* reinforcement layout and type; and concrete resistance (Seible, 1996). The nominal flexural resistance (defined as the maximum resisting moment per unit width) of the facing can be estimated using conventional formulas for reinforced concrete design. When the flexural resistance is reached in the equivalent two-way slab, the "reaction" forces in the nails are considered the nominal resistance force, R_{FF}, for flexure to be used in LRFD equations.



Source: Modified after Lazarte et al. (2003)

Figure 3-10. Limit states in soil nail wall facings.



Source: Modified after Lazarte et al. (2003)

Figure 3-11. Schematic relation between flexure mechanism and nail forces in SNW facings.

The force that mobilizes in the nail as a reaction to the soil pressures could be also evaluated by multiplying the soil pressure by the contributing area around the nail, or $S_H \times S_V$. The calculation of the resistance R_{FF} is presented in Equation 3-54.

For the flexural limit state, it must be verified that:

$$\phi_{FF}\ R_{FF} \geq T_o \tag{3-53}$$

where

ϕ_{FF} = the resistance factor for flexure in the facing;
R_{FF} = the nominal resistance for facing flexure (considered a force herein); and
T_o = the nail maximum tensile force at the facing.

This limit state must be considered separately for both temporary and permanent facings; therefore, separate values of R_{FF} must be obtained for the temporary and permanent facings.

R_{FF} is estimated using the following expression:

$$R_{FF}\,[\text{kip}] = 3.8 \times C_F \times f_y\,[\text{ksi}] \times \text{lesser of} \begin{cases} (a_{vn} + a_{vm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_H\ h[\text{ft}]}{S_V}\right) \\ (a_{hn} + a_{hm})[\text{in}^2/\text{ft}] \times \left(\dfrac{S_v\ h[\text{ft}]}{S_H}\right) \end{cases} \tag{3-54}$$

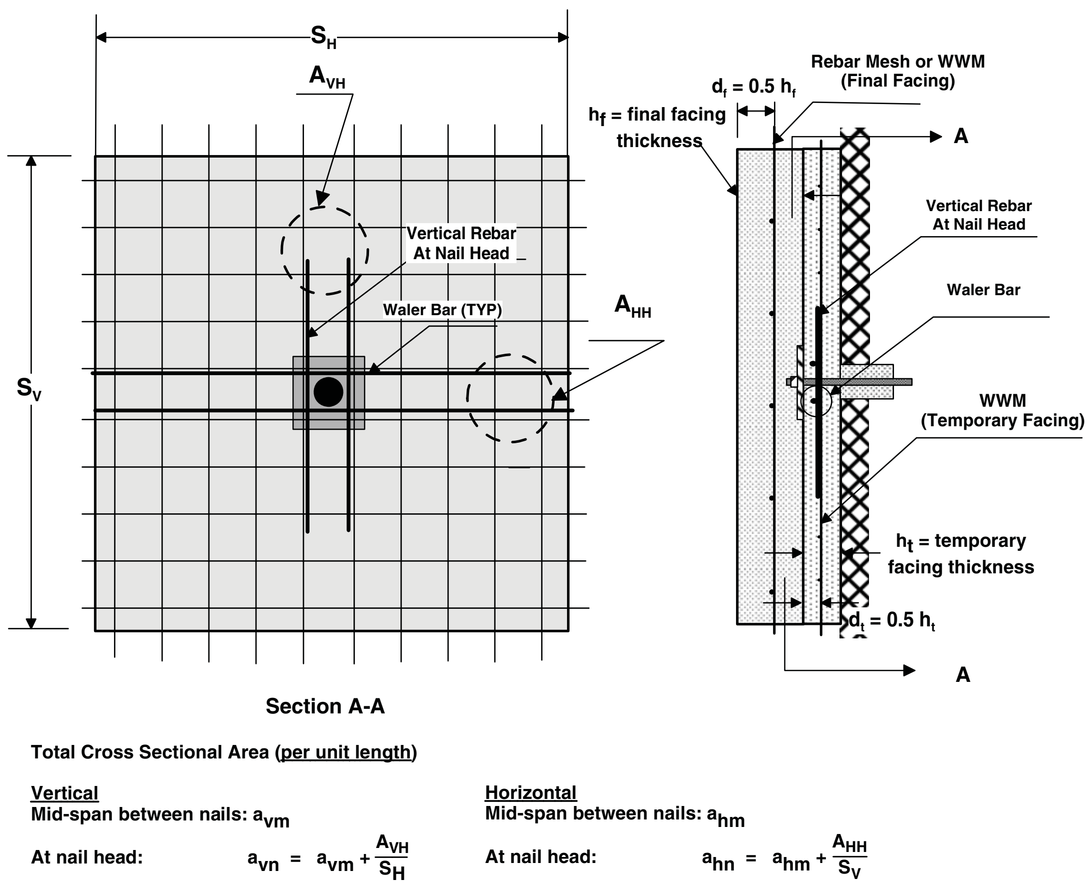


Figure 3-12. Resistance and reinforcement nomenclature for flexure limit state.

Table 3-1. Factor C_F.

Type of Facing	Facing Thickness, h_t or h_f (in.)	Factor C_F
Temporary	4	2.0
	6	1.5
	8	1.0
Permanent	All	1.0

where

C_F = a factor to be obtained from Table 3-1, which is based on Byrne et al. (1998), to consider the non-uniform distribution of soil pressures behind the facing;
f_y = the bar nominal yield resistance;
S_H = the horizontal nail spacing;
S_V = the vertical nail spacing;
h = the facing thickness (h_t for temporary facings and h_f for permanent facings);
a_{vn} = the cross-sectional area of the WWM (per unit length) in the vertical direction over the nail head;
a_{vm} = the cross-sectional area of the WWM (per unit length) in the vertical direction in the mid-span between nails;
a_{hn} = the cross-sectional area of the WWM (per unit length) in the horizontal direction over the nail head; and
a_{hm} = the cross-sectional area of the WWM (per unit length) in the horizontal direction in the mid-span between nails.

The directions and locations that these quantities refer to are shown on Figure 3-12.

Figure 3-11 shows a schematic diagram of a non-uniform distribution of soil pressure behind the facing. This distribution is affected by the wall displacement magnitude, soil conditions, facing thickness, and facing stiffness. The diagram of Figure 3-12 shows that the earth pressure is relatively low between nails, where relatively larger outward displacement tends to produce a stress relief. Earth pressures near the nail heads are larger than those occurring in mid-span because the soil confinement at the nail head is significantly larger. To account for these effects, the factor C_F is used to consider pressure distributions that are not uniform. Table 3-1 contains values of C_F for typical facing thickness. For permanent facings and for relatively thick (i.e., $h_f = 8$ in. or more) temporary facings, $C_F = 1$ (i.e., the soil pressure distribution is assumed to be uniform).

The cross-sectional areas of reinforcement per unit width in the vertical or horizontal direction and around and between nails are shown schematically in Figure 3-12. The nomenclature for the reinforcement areas per unit width is presented in Table 3-2.

In Equation 3-54, the reinforcement (wire mesh and bars) is assumed to be in the middle of the section, at a distance, d, of half the total thickness, $h/2$, from the facing surface (Figure 3-12). The total thickness can take the values h_t for temporary facings or h_f for permanent facings; correspondingly, d can take the values d_t for temporary facings or d_f for permanent facings (see Figure 3-12). Recommendations on the minimum and maximum reinforcement ratios in the facing and other considerations can be found in Lazarte et al. (2003) and in the design specifications contained in Appendix A.

Examples of the use of the formulation presented herein can be found in Lazarte et al. (2003).

Punching-Shear in Facings. Connectors installed at the nail head may be subjected to a punching-shear limit state, which may occur if the nominal shear resistance of the reinforced shotcrete section around the nails is exceeded. The nominal punching-shear resistance must be evaluated for both temporary and permanent facings (Figure 3-13) for the following situations:

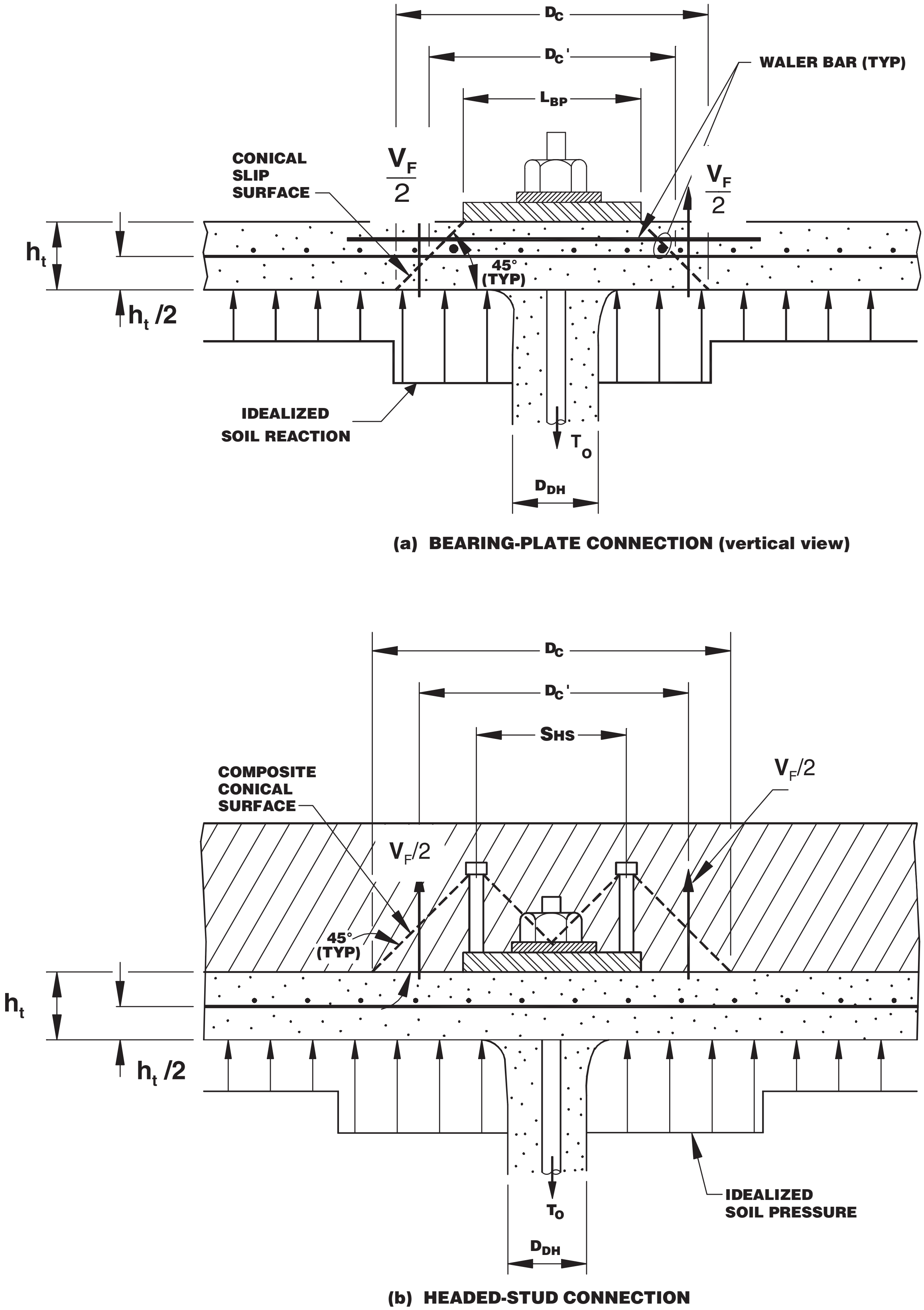
- Bearing-plate connection in temporary facings and
- Headed-stud connection in permanent facings.

Table 3-2. Nomenclature for facing reinforcement area per unit width.

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
vertical	nail head [(1)]	$a_{vn} = a_{vm} + \frac{A_{VH}}{S_H}$
	mid-span	a_{vm}
horizontal	nail head [(2)]	$a_{hn} = a_{hm} + \frac{A_{HH}}{S_V}$
	mid-span	a_{hm}

Notes: (1) At the nail head, the total cross-sectional area (per unit length) of reinforcement is the sum of the WWM area (a_{vm}) and the area of vertical waler bars (A_{VH}) divided by the horizontal spacing (S_H).

(2) At the nail head, the total area is the sum of the area of the WWM (a_{hm}) and the area of the horizontal bar (A_{HH}) divided by S_V.



Source: Modified after Byrne et al. (1998)

Figure 3-13. Limit states for punching-shear in facing—horizontal cross sections.

At the limit state, conical slip surfaces can form in the facing section around the nail head. The size of the conical slip surface is affected by the facing thickness and the dimension of the nail head components (i.e., bearing-plate or headed-studs) that are present.

For both situations, the nominal facing punching-shear resistance, R_{FP}, must meet the following condition:

$$\phi_{FP}\ R_{FP} \geq T_o \quad (3\text{-}55)$$

where ϕ_{FP} is the resistance factor for punching-shear in the facing. R_{FP} can be estimated as:

$$R_{FP} = C_P V_F \quad (3\text{-}56)$$

where

C_P = a dimensionless factor that accounts for the contribution to shear resistance of the soil support under the nail head area, and

V_F = the nominal punching-shear force acting through the facing section.

When the soil reaction is considered, C_P can be as high as 1.15. For design purposes, it is conservatively assumed that the soil support behind the wall is negligible, and C_P = 1.0. The punching-shear force can be calculated as:

$$V_F\,[\text{kip}] = 0.58\sqrt{f_c'[\text{psi}]}\ \pi\ D_c'[\text{ft}]\,h_c\,[\text{ft}] \quad (3\text{-}57)$$

where f_c' is the concrete nominal compressive resistance (in psi); D_c' is the effective equivalent diameter of the conical slip surface (in ft); and h_c is the effective depth of the conical surface (in ft). D_c' and h_c must be selected separately for the temporary and permanent facing, as follows.

The effective equivalent diameter of the conical slip surface can be calculated as:

Temporary facing [Figure 3-13(a)]

$$D_c' = L_{BP} + h_t \quad (3\text{-}58)$$

$$h_c = h_t \quad (3\text{-}59)$$

where L_{BP} is the bearing plate size, and h_t is the temporary facing thickness.

Permanent facing [Figure 3-13(b)]

$$D_c' = \text{minimum of} \begin{cases} S_{HS} + h_c \\ 2h_c \end{cases} \quad (3\text{-}60)$$

where $\quad h_c = L_S - t_H + t_P \quad (3\text{-}61)$

where

S_{HS} = the headed-stud spacing (Figure 3-13);

L_S = the headed-stud length (Figure 3-14);

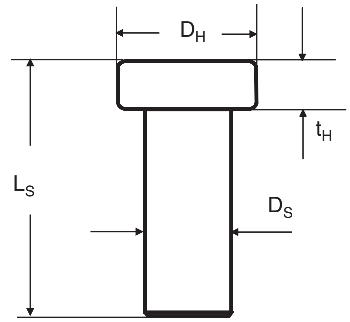


Figure 3-14. Geometry of headed-stud.

t_H = the thickness of the stud head (Figure 3-14); and

t_P = the bearing plate thickness.

Available sizes of headed-stud connectors can be found in Byrne et al. (1998), Lazarte et al. (2003), and in references provided by manufacturers.

Headed-Stud Tensile Resistance in Permanent Facings. The tensile resistance of headed-stud connectors in permanent facings, R_{FH}, must comply with:

$$\phi_{FH}\ R_{FH} \geq T_o \quad (3\text{-}62)$$

where ϕ_{FH} is the resistance factor for headed-stud tensile resistance.

R_{FH} is calculated as:

$$R_{FH} = N\ A_S\ f_y \quad (3\text{-}63)$$

where

N = the number of headed studs per nail head location (usually 4);

A_S = the cross-sectional area of the headed-stud shaft of diameter D_S (Figure 3-14); and

f_y = the tensile nominal yield resistance of the headed-stud.

Headed-studs are usually A307 steel or, less commonly, A325 steel (Byrne et al., 1998). To prevent the heads of the connectors from exerting an excessive amount of compressive stress on the concrete bearing surface, the following geometric constraints must be met (ACI, 1998):

$$A_H \geq 2.5\ A_S \quad (3\text{-}64)$$

$$t_H \geq 0.5(D_H - D_S) \quad (3\text{-}65)$$

where

A_H = the cross-sectional area of the connector head;

A_S = as defined earlier;

t_H = the connector head thickness;

D_H = the diameter of the connector head; and

D_S = the diameter of the connector shaft.

To provide an efficient anchorage of the connector in the facing, connector heads must extend beyond the plane containing the mesh, toward the exposed face, while a minimum shotcrete cover of 2 in. is maintained.

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

$$A_E = \frac{\pi}{4}\left[D_E - \left(\frac{0.9743}{n_t}\right)\right]^2 \tag{3-66}$$

where

D_E = the effective diameter of the bolt core; and

n_t = the number of threads per unit length.

3.4.5 Seismic Considerations in Extreme-Event Limit States of Soil Nail Walls

3.4.5.1 Introduction

Seismic forces must be considered in SNW design in areas with moderate to high seismic exposure and, according to the *LRFD Bridge Design Specifications,* seismic effects must be considered in the design of bridge substructures as an extreme-event limit state. In general, the response of SNWs to past strong ground motions has been very good to excellent. Observations made after earthquakes (i.e., 1989 Loma Prieta, California; 1995 Kobe, Japan; and 2001 Nisqually, Washington) indicate that SNWs did not show signs of significant distress or permanent deflection (Felio et al., 1990; Tatsuoka et al., 1997; Tufenkjian, 2002), although ground accelerations were as large as 0.7*g* near some of the surveyed walls. Vucetic et al. (1993) and Tufenkjian and Vucetic (2000) observed similar trends in centrifuge tests performed on reduced-scale models of SNWs. Observations suggest that SNWs have an intrinsic satisfactory seismic performance, which is attributed in part to the flexibility of SNWs. The seismic performance of SNWs appears to be comparable to that of MSE walls (i.e., another type of flexible retaining system).

The inertial forces that act on retaining earth systems (including SNWs) during a seismic event can be taken into account in stability evaluations using simplified procedures. In these procedures, seismic coefficients are used to calculate equivalent, pseudo-static forces that act at the centroid of the potentially unstable soil block being analyzed. The most commonly used pseudo-static procedure is the Mononobe-Okabe Method (MOM), which is an extension of the Coulomb theory (Mononobe, 1929; Okabe, 1926). The MOM, which is described by Seed and Whitman (1970) and Richards and Elms (1979), was originally developed for gravity walls and can also be used for SNWs (Lazarte et al., 2003). In this method, it is assumed that:

- The facing and the soil mass that is reinforced by nails act as a rigid block;
- Active earth pressure conditions develop behind the wall; and
- Lateral earth loads act behind the nails during a seismic event.

In the *LRFD Bridge Design Specifications* (AASHTO, 2007), earthquake loads are considered part of the load cases of the Extreme-Event I Limit State load combination. For this state, the resistance factor for soil is 1.0 and the load factor for the seismic force is $\gamma_{EQ} = 1.0$.

3.4.5.2 Seismic Coefficients

The main consideration in the seismic response of SNWs is the horizontal forces produced during a seismic event. Horizontal forces can be simplistically computed as the product of the seismic coefficient, k_h (if only horizontal forces are considered), and the mass of the potentially unstable soil block. The horizontal coefficient k_h is a fraction of the maximum acceleration coefficient, A_m. The coefficient A_m is the ratio of the acceleration occurring at the centroid of the soil block and the acceleration of gravity, g. A_m is a function of the peak ground acceleration coefficient, A:

$$A_m = (1.45 - A)A \tag{3-67}$$

A can be obtained from national seismic maps contained in AASHTO (2007), as described in Article 3.10.2 of *LRFD Bridge Design Specifications.*

Instead of considering k_h to be only a function of A, a more rational approach for flexible retaining earth systems, such as SNWs, is to use seismic horizontal coefficients that depend on the maximum seismically induced wall displacement (Richards and Elms, 1979; Kavazanjian et al., 1997; Elias et al., 2001; AASHTO, 2007). In this approach, k_h is expressed as:

$$k_h = 0.74 A_m \left(\frac{A_m}{d[\text{in.}]}\right)^{0.25} \tag{3-68}$$

where d is the maximum seismically induced wall displacement (expressed in inches) selected for the retaining structure.

Equation 3-68 should be used only for $1 \le d \le 8$ in., with typical values of d ranging between 2 and 4 in. A smaller value of d results in larger seismic coefficients and, therefore, longer nails. Equation 3-68 should not be used if:

- $A \geq 0.3$,
- The wall has a complex geometry (i.e., the distribution of mass and/or stiffness with height is abrupt), or
- The wall height is greater than approximately 45 ft.

These limitations are imposed because (i) ground response that typically occurs under large seismic events is non-linear (a condition not considered in the MOM) and (ii) higher modes of vibration of the wall may participate in the case of complex geometries and tall walls (a condition not considered in the MOM). If deep deposits of medium to soft fine-grained soils underlie the site, ground accelerations could be amplified significantly, inducing a non-linear site response. These conditions commonly require full dynamic site response analyses, which must thoroughly consider soil dynamic properties and representative ground acceleration time-histories.

The condition $A > 0.3$ arises for Seismic Zone 4, as defined in Table 3.10.4-1, Seismic Zones, of Section 3.10.4, Seismic Performance Zones, of AASHTO (2007). Various areas in the western United States are classified as Seismic Zone 4, including some of the most populated areas, such as most California coastal locations, and some areas in Idaho, Nevada, and Alaska.

3.4.6 Design for Service Limit States (Displacements)

3.4.6.1 Introduction

As part of the design of SNWs, the maximum lateral and vertical movements of the wall must be estimated and verified to be less than the tolerable deformation limits of the wall. These design consideration aspects are described in the following sections.

3.4.6.2 Soil Nail Wall Displacements

Because SNWs are passive reinforcement systems, some deformation of the wall should be expected during SNW construction and service life. Some small, tolerable deformation is a natural condition in SNWs as nails must deform to mobilize their tensile resistance. Most of the outward movement of SNWs tends to occur during or shortly after excavation and is commonly largest at the top of the wall. Post-construction deformation may increase due to added loads and soil creep. In general, lateral deflections increase with:

- Increases in:
  - Wall height,
  - Nail spacing,
  - Steepness of nail inclination, and
  - Surcharge magnitude; and
- Decreases in:
  - Wall batter,
  - Soil stiffness,
  - Nail length, and
  - Cross-sectional areas of bars.

Vertical displacements, which are also affected generally by the above factors, are largest near the facing and are commonly smaller than lateral deflections at the top of the wall.

Clouterre (1991) showed that the maximum long-term horizontal and vertical wall displacements at the top of the wall, δ_h and δ_v, can be estimated using Equation 3-69 if (i) the ratio of the nail length to the wall height is greater than 0.7; (ii) the surcharge is negligible; and (iii) $FS = 1.5$ is adopted for overall stability (e.g., in ASD calculation):

$$\delta_v \approx \delta_h = \left(\frac{\delta_h}{H}\right)_i \times H \qquad (3\text{-}69)$$

where $(\delta_h/H)_i$ is a factor that depends on soil conditions as indicated in Table 3-3.

Ground deformation can be significant up to a distance, D_{DEF}, behind the wall (Figure 3-15). This distance can be estimated as:

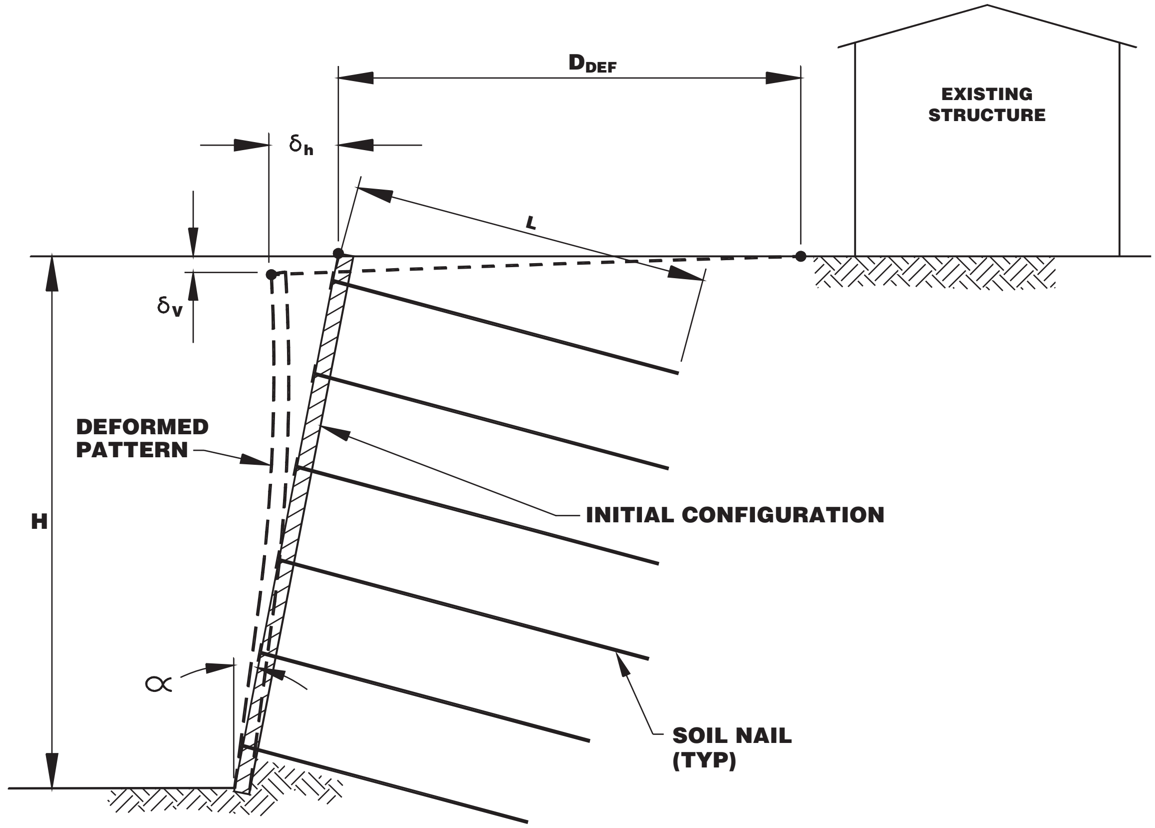
$$D_{DEF} = C(1-\tan\alpha)H \qquad (3\text{-}70)$$

where α is the wall batter angle, and C is a soil-dependent coefficient included in Table 3-3.

Typical movements of SNWs are usually small and comparable to those observed in braced systems and anchored walls. However, the criterion for tolerable deformation is project dependent. If important, sensitive structures occur near the SNW, an assessment of the potential impact of wall movement on these structures is warranted. When excessive deformations are presumed or observed, modifications must be

Table 3-3. Values of $(\delta_h/H)_i$ and C as functions of soil conditions.

Variable	Weathered Rock and Stiff Soil	Sandy Soil	Fine-Grained Soil
$(\delta_h/H)_i$	1/1,000	1/500	1/333
C	0.8	1.25	1.5



Source: Modified after Clouterre (1991) and Byrne et al. (1998)

Figure 3-15. Deformation of soil nail walls.

made to the wall geometry or soil nail layout (e.g., considering the factors listed above). See Lazarte et al. (2003) for additional recommendations.

3.4.6.3 Lateral Squeeze

If a SNW is part of a bridge abutment, it lies atop relatively soft soils, and it is subjected to unbalanced loads (e.g., embankment loads behind the wall abutment), a verification for lateral squeeze may be necessary to ensure that excessive lateral deflections do not occur at the toe of the wall. Guidance for evaluating lateral squeeze, as well as methods for stabilizing soils to prevent problems related to lateral squeeze, are presented in Hannigan et al. (2005).

3.5 Development of Resistance and Load Factors for Soil Nail Walls

3.5.1 Introduction

This section presents the basis for development of resistance and load factors for the limit states of SNWs identified in the previous section. Section 3.5.2 presents the load factors that are applicable in general to earth-retaining structures and presents a discussion on the load factors specifically for SNWs. Section 3.5.3 presents resistance factors for soil-related limit states in SNWs. Section 3.5.4 presents resistance factors for structural limit states in SNWs. Section 3.5.5 includes a preliminary range of resistance factors for pullout resistance prior to calibration. Finally, Section 3.5.6 presents a summary of resistance factors to be considered for the design of SNWs in the LRFD.

3.5.2 Common Load Factors in Earth-Retaining Structures

As mentioned previously, load factors are established for specific limit states and load types. In AASHTO (2007), the following 12 limit states and associated load combinations are included:

- Strength limit states (five load combinations, I through V);
- Extreme-event limit states (two load combinations, I and II);
- Service limit states (four load combinations, I through IV); and
- Fatigue limit states (one load combination).

Table 3-4, which is based on AASHTO (2007), presents a summary of the load combinations and load factors for each of the limit states listed above.

Table 3-4. Load factors and load combinations [Based on AASHTO (2007)].

Limit State and Load Combination	Permanent Loads (1)	Transient Loads (2)								Extreme-Event Loads (3)			
	DC, DD, DW, EH, EV, ES, EL	***LL, IM, CE, BR, PL, LS***	***WA***	***WS***	***WL***	***FR***	***TU, CR, SH***	***TG*** (5)	***SE*** (6)	***EQ***	***IC*** (7)	***CT*** (7)	***CV*** (7)
Strength I (unless noted)	γ_p (4)	1.75	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength II	γ_p (4)	1.35	1.00	–	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength III	γ_p (4)	–	1.00	1.40	–	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Strength IV (*EH, EV, ES, DW*)	γ_p (4)	–	1.00	–	–	1.00	0.50/1.20	–	–	–	–	–	–
(*DC* only)	1.5												
Strength V	γ_p (4)	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Extreme-Event I	γ_p (4)	γ_{EQ}	1.00	–	–	1.00	–	–	–	1.00	–	–	–
Extreme-Event II	γ_p (4)	0.50	1.00	–	–	1.00	–	–	–	–	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Service II	1.00	1.30	1.00	–	–	1.00	1.00/1.20	–	–	–	–	–	–
Service III	1.00	0.80	1.00	–	–	1.00	1.00/1.20	γ_{TG}	γ_{SE}	–	–	–	–
Fatigue *LL*, *IM* & *CE* only	–	0.75	–	–	–	–	–	–	–	–	–	–	–

Notes:

(1) Permanent Loads
- *DC* = dead load of structural components and non-structural attachments
- *DD* = downdrag
- *DW* = dead load of wearing surfaces and utilities
- *EH* = horizontal earth pressure load
- *EL* = locked-in effects from construction, including forces from post-tensioning
- *ES* = earth surcharge load
- *EV* = vertical pressure from dead load of earth fill

(2) Transient Loads
- *BR* = vehicular braking force
- *CE* = vehicular centrifugal force
- *CR* = creep
- *FR* = friction
- *IM* = vehicular dynamic load allowance
- *LL* = vehicular live load
- *LS* = live load surcharge

(2) Transient Loads (continued)
- *PL* = pedestrian live load
- *SE* = settlement
- *SH* = shrinkage
- *TG* = temperature gradient
- *TU* = uniform temperature
- *WA* = water load and stream pressure
- *WL* = wind pressure on vehicles
- *WS* = wind pressure on structures

(3) Extreme-Event Loads
- *CT* = vehicular collision force
- *CV* = vessel collision force
- *EQ* = earthquake
- *IC* = ice load

(4) Load factors for permanent loads vary with load type. See Table 3-5.
(5) Load factors for temperature gradient can be found in Article 3.4.1 of AASHTO (2007)
(6) Load factors for settlement can be found in Article 3.4.1 of AASHTO (2007)
(7) Use one of these loads at a time

For earth-retaining structures, the most critical loads are permanent loads associated with horizontal and vertical earth pressures (*EH, EV*), dead loads (*DC* and *DW*), and surcharge loads. Details on earth surcharges are described in Articles 3.11.6.1 and 3.11.6.2 of AASHTO (2007) and on live loads in Article 3.11.6.4. If the substructure is part of a bridge abutment, live loads (*LL*) and other transient loads transferred from the bridge superstructure must also be considered in the analysis.

Load factors for permanent loads in strength and extreme-event limit states must be selected based on (i) the type of permanent load being considered and (ii) whether the permanent load has unfavorable (i.e., destabilizing) or favorable effects, as described previously. Load factors for permanent loads are presented in Table 3-5. For all limit states, permanent load factors are assigned maximum or minimum values as presented in Table 3-5 to consider destabilizing or stabilizing effects. Maximum and minimum values will change based on the influence of permanent loads for each limit state being examined (e.g., bearing, eccentricity, global stability, etc.). As seen in Table 3-5, load factors for permanent loads $\gamma_p \geq 1.0$ must be selected if the load is destabilizing. For example, soil horizontal lateral pressures, *EH,* acting behind earth-retaining structures are destabilizing and γ_p should be selected to vary between 1.0 and 1.5, depending on the lateral earth pressure condition. Conversely, load factors $\gamma_p \leq 1.0$ must be selected if the permanent load is stabilizing. For example, for the weight of soil load, *EV,* acting behind a gravity wall, γ_p should be selected to vary between 0.9 and 1.0. Load factors for permanent loads $\gamma_p = 1.0$ must be selected for service limit states.

Based on the provisions for earth-retaining structures included in Article 11.5, Load Combinations and Load Factors of AASHTO (2007), the most common limit states for SNWs can be:

- Service limit states (e.g., Service I Limit State, which involves overall stability);
- Strength limit states (e.g., Strength I or IV Limit States that involve soil failure); and
- Extreme-event limit states (e.g., Extreme-Event I Limit State, which involves earthquake loads).

Some of these loads may be present where the SNW is used in a road-widening project under a bridge. Service II through IV Limit States should not be considered for overall stability, as these limit states are reserved to assess the condition of steel structures (Service II Limit State) and pre-stressed concrete superstructures (Service III and IV Limit States), per Section 3.4 of AASHTO (2007). Fatigue limit states are not typically considered for substructures; hence, they are not considered further in this document.

For consistency with the current AASHTO (2007) practice, overall stability will be considered in this document to be a service limit state. For compatibility with AASHTO (2007), load factors for earth loads in SNW design are temporarily adopted for $\gamma = 1.0$. However, the calibration of resistance factors will be made for a range of load factors varying from 1.0 to 1.75.

As shown in Table 3-4, the load factors associated with earth loads that participate in earth-retaining structures (i.e., *EH* and *EV*) are $\gamma = 1.0$ for the case of overall stability (i.e., Service I Limit State). A similar condition applies to load factors for live loads, *LL,* and other surcharge loads in the service limit state.

Table 3-5. Load factors, γ_p, for permanent loads.

Type of Load	Load Factor γ_p	
	Maximum[(1)]	Minimum[(2)]
DC: Dead load of structural components	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Dead load of wearing surface and utilities	1.50	0.65
EH: Horizontal earth pressure • Active • At-Rest • Locked-in Erection Stresses	 1.50 1.35 1.00	 0.90 0.90 1.00
EV: Vertical earth pressure • Overall stability • Retaining walls and abutment • Rigid buried structure • Rigid frame • Flexible buried structure other than metal box culvert • Flexible metal box culvert	 1.00 1.35 1.30 1.35 1.95 1.50	 N/A 1.00 0.90 0.90 0.90 0.90
ES: Earth surcharge	1.50	0.75

Notes: (1) For unfavorable effects of permanent load.
(2) For favorable effects of permanent load.

Source: Modified after Table 3.4.1-2 (AASHTO, 2007)

The selection of $\gamma = 1.0$ establishes that all uncertainty in design concentrates on only the resistance factor.

In the limit-equilibrium methods that are commonly used in the design of SNWs, the mass of soil above a potential slip surface is separated into several "slices" for analysis purposes. Slices located near the lower end of the slip surface tend to be stabilizing. Conversely, slices located near the upper end of the slip surface tend to be destabilizing. The weight of each slice contributes to the soil frictional resistance along the slip surface; this effect is considered a stabilizing effect. Assigning a different load factor, γ_P, for each load component of every slice depending on whether the effect is stabilizing or destabilizing must be considered in the software being used for analysis. However, most available software lacks these capabilities. Care must be exercised to not violate force and moment equilibrium, conditions that must be satisfied necessarily with unfactored values of weight and resistances. A uniform value $\gamma_P = 1.0$ is used with all slices in part because not all software have these capabilities. It is acknowledged in AASHTO (2007) that this approach is an interim solution due to the current lack of a satisfactory methodology and calibration data for applying LRFD methods to stability analysis computations.

3.5.3 Resistance Factors for Sliding, Basal Heave, Overall Stability, and Seismic Limit States

3.5.3.1 Introduction

This section provides a discussion of the resistance factors used for sliding, basal heave, overall stability, and seismic limit states that are associated with SNWs. These factors are based on the information provided in Section 11 of AASHTO (2007) for other retaining structures.

3.5.3.2 Sliding

The resistance factor for sliding in SNWs in this document is consistent with the approach in AASHTO (2007) for other earth-retaining systems, including abutments and conventional retaining walls [Section 11.6 of AASHTO (2007)], mechanically stabilized earth walls [Section 11.10 of AASHTO (2007)], and prefabricated modular walls [Section 11.11 of AASHTO (2007)].

The resistance factor for potential sliding of the mass of reinforced soil (considered as a block) must be selected for the condition of soil sliding on soil at the base of the soil block, per Sections 11.6, 11.10, and 11.11 of AASHTO (2007), all of which refer to Table 10.5.5-1 of AASHTO (2007). For sliding under this scenario, the resistance factor is specified to be $\phi_\tau = 0.90$.

3.5.3.3 Basal Heave

If an SNW is constructed in or over soft, fine-grained soil, basal heave should be considered a potential limit state. The resistance factor, ϕ_b, applicable for this case coincides with that used for bearing resistance, for which $\phi_b = 0.70$.

3.5.3.4 Overall Stability

Per Article 11.6.3.4 of AASHTO (2007), resistance factors for soil failure in overall stability evaluations are selected to be (i) $\phi_s = 0.75$ when the analyzed slope does not support a structure and (ii) $\phi_s = 0.65$ when the slope supports a structural element.

The current version of AASHTO (2007) includes a statement that differentiates the above two values for ϕ_s depending on whether (i) geotechnical parameters are well defined, in which case $\phi_s = 0.75$, or (ii) geotechnical parameters are based on limited information, in which case $\phi_s = 0.65$, per Article 11.6.2.3, Overall Stability. However, this stipulation appears to contradict the requirements set forth in Section 10.4 of AASHTO (2007), where directions are provided to ensure an adequate geotechnical investigation.

The following general condition for overall stability analysis is considered:

$$\phi_s R_n = \gamma Q \tag{3-71}$$

where

ϕ_s = resistance factor for overall stability analysis;
R_n = general term representing the soil nominal resistance in overall stability analyses;
γ = load factor; and
Q = loads.

If $\gamma = 1.0$, resistance factors for overall stability can be related to equivalent global stability *FS*, as defined previously. With $FS = R_n/Q$ and $\gamma = 1.0$, the above equation becomes:

$$\phi_s = \frac{Q}{R_n} = \frac{1}{FS} \tag{3-72}$$

Using Equation 3-72, it is feasible to calibrate the resistance factor directly from *FS*. This calibration approach is calibration Method B presented in Section 3.2.6. Conversely, *FS* can be derived from the resistance factor. For example, for cases when the slope does not support a structure or geotechnical parameters are well defined, $FS = 1.0/0.75 = 1.33$. For cases when the slope supports a structure or geotechnical parameters are based on limited information, $FS = 1.0/0.65 = 1.53$. These *FS* are consistent with minimum values currently employed to design SNWs using the ASD method. For example, in the ASD method developed for SNWs (Lazarte et al., 2003), $FS = 1.5$ and $FS = 1.35$ for permanent and temporary SNWs, respectively.

Byrne et al. (1998) selected separate resistance factors for the cohesive and frictional components of the soil resistance in overall stability analysis. For non-critical, permanent structures, Byrne et al. (1998) selected resistance factors as $\phi_s = 0.90$ and 0.75 for cohesion and friction, respectively. In Byrne et al. (1998), resistance factors were applied to tan φ or c (where φ and c are the soil friction angle and cohesion, respectively) rather than to global, integrated resistances, as is done in the *LRFD Bridge Design Specifications.* While the concept of differentiating a resistance factor for cohesion and friction seems a rational approach, only one resistance factor is provided in this report for geotechnical resistance, consistent with the current AASHTO LRFD practice.

3.5.3.5 Extreme Events—Seismic

Provisions of Article 11.6.5 of AASHTO (2007) specify that, for overall stability under seismic loads (i.e., Extreme-Event I Limit State), resistance factors for soil must be equal to $\phi_s = 0.90$, as was selected for earth-retaining structures. In Article 11.6.5 of AASHTO (2007), the restriction of $\phi_s < 1.0$ for overall stability appears to contradict the tenet presented in the same article, where it is stated that, "The effect of earthquake loading on multi-span bridges shall be investigated using the extreme-event limit state of Table 3.4.1-1 with resistance factors $\phi_s = 1.0$." Considering that $\gamma = 1.0$ for seismic loads in overall stability at the service limit state, it results that $FS = 1/0.90 = 1.1$ in this limit state. This result is consistent with FS values recommended in an ASD framework for SNW design (Lazarte et al., 2003) for permanent or critical structures. However, it is inconsistent with the approach developed by Byrne et al. (1998) in which the resistance factor for stability in seismic analysis was equivalent to $\phi_s = 1.0$. In this document, consistency with AASHTO (2007) is maintained and the values of ϕ_s are selected to be consistent with those for permanent structures, and $\phi_s = 0.90$. A value $\phi_s = 1.00$ may be acceptable, as long as permanent deformations are calculated and deformations are found to be within tolerable ranges. Currently, no differentiation exists for temporary structures in AASHTO (2007). A value of $\phi_s = 1.0$ (which corresponds approximately to $FS = 1.0$) is recommended for temporary structures.

Major changes have been incorporated in the seismic section of the 2008 interim version of the LRFD AASHTO standard (Anderson et al., 2008) and, therefore, adjustments to seismic design of SNWs are expected once the interim provisions become permanent. In *NCHRP Report 611: Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments* (Anderson et al., 2008), several changes are proposed in the procedures used to analyze the seismic performance of several types of retaining structures, including soil nail walls. In *NCHRP Report 611,* it is proposed that the seismic response of SNWs should be evaluated using deformation-based procedures that account for the expected ground motion characteristics at a given site, site response, soil conditions, and wall height. Anderson et al. (2008) propose that a fraction of the peak ground acceleration should be reduced to account for the permanent wall displacements. Similar, albeit simpler, recommendations had been provided in Lazarte et al (2003) and are included in this document. The incorporation of the proposals contained in Anderson et al. (2008) was not part of the original plan of this report; however, those provisions may also be considered when these proposed design specifications for SNWs are reviewed by AASHTO.

3.5.4 Resistance Factors for Structural Limit States

3.5.4.1 Resistance Factors for Tension in Soil Nails

The tensile resistance factor to be used in SNWs selected in this document is consistent for the case of load factors in overall stability or $\gamma = 1.0$. To this end, the resistance factor is adopted as follows: for nail bars of mild steel (i.e., ASTM A 615), $\phi_T = 0.56$; for high-resistance soil nail bars (e.g., ASTM A 722), $\phi_T = 0.50$. The value for mild steel is consistent with the ASD safety level used in Lazarte et al. (2003). For mild steel bars, the resistance factor is applied to the yield resistance, f_Y; for soil nails of high-resistance bars, the resistance factor is applied to the guaranteed ultimate tensile strength (GUTS).

Note that for the tension limit state of ground anchors walls, Table 11.5.6-1 of Section 11.5, Limit States and Resistance Factors of AASHTO (2007), gives $\phi_T = 0.90$ for soil nail bars of mild steel and $\phi_T = 0.80$ for high-resistance soil nail bars. However, these values were developed for load factors higher than 1.0. For example, in Strength I Limit State, a load combination of permanent dead loads and transient or live loads, $\gamma_{DC} = 1.25$ (maximum per Table 3-5), and $\gamma_{LL} = 1.75$.

For seismic events and $\gamma = 1.0$, $\phi_T = 0.74$ and $\phi_T = 0.67$ can be selected for mild steel bars and high-resistance steel bars, respectively. For cases with load factors similar to those of Strength I Limit State, $\phi_T = 1.00$ can be selected for both cases.

3.5.4.2 Resistance Factors for Flexure in Facing

Facing failures and instrumentation of facings are practically non-existent. Therefore, due to the lack of available data, resistance factors cannot be calibrated. As a result, the resistance factor for flexure of SNW reinforced concrete/shotcrete facings is selected to be similar to that for flexure of reinforced concrete per AASHTO (2007). Adopting the same resistance factors for shotcrete and concrete is akin to assuming that the uncertainty related to the strength of these materials is comparable. The current practice of shotcrete use involves (i) mix design principles that are as sophisticated as those used with concrete; (ii) pre-project submissions on material properties as thorough as those used in concrete; (iii) high qualifications/experience requirements for shotcrete application personnel; and (iv) frequent shotcrete verification testing. Therefore, it is justifiable to presume that the material variability in these materials is comparable. Overall, the practice of shotcrete placement bears similarities with that of in-situ cast reinforced concrete. Therefore, these similarities in practice justify the determination that, as a first approximation, the uncertainty related to shotcrete and concrete resistances are comparable. One aspect that might be different between these two material technologies is that the efficiency of the design equations used for flexure of shotcrete facing, although already tested (see below), may not have been quantified as much as those for reinforced concrete.

The resistance factor for flexure of SNW reinforced concrete/shotcrete facings is selected for load factors for overall stability $\gamma = 1.0$. The resistance factor for flexure of a SNW shotcrete facing is selected to be $\phi_{FF} = 0.67$, a value that is consistent with values included in Lazarte et al. (2003) for permanent structures in an ASD format and is consistent with AASHTO (2007), after corrections are made for $\gamma = 1.0$. Note that for the flexure limit state, Article 5.5.4.2.1 of AASHTO (2007) provides a resistance factor for flexure of reinforced concrete equal to $\phi_{FF} = 0.90$, a value obtained for load factors much higher than 1.0.

3.5.4.3 Resistance Factors for Punching-Shear in Facing

Laboratory tests were conducted at the University of California, San Diego (Seible, 1996) to study the structural response of SNW facings. Results obtained in controlled tests were compared to values obtained with a formulation presented in Byrne at al. (1998) to estimate the punching-shear resistance, R_{FP}. This comparison served to evaluate the predictive capabilities of those formulas. Comparisons between test results and estimated resistances indicate that the bias (i.e., measured over predicted resistances) ranges from 1.07 to 1.23. The number of test results was too small to develop reliable statistics of the bias for punching-shear resistance. Therefore, for punching, a full calibration cannot be completed of the resistance factor with empirical results. Hence, the resistance factor is adopted as follows.

For the punching-shear resistance in an SNW facing (either reinforced shotcrete or concrete), a resistance factor of $\phi_{FP} = 0.67$ is used. This value is consistent with values included in Lazarte et al. (2003) for permanent structures in an ASD format and is consistent with AASHTO (2007), after corrections are made for $\gamma = 1.0$. For cases with load factors similar to those of Strength I Limit State, the equivalent resistance factor for punching-shear resistance in an SNW facing would result in $\phi_{FP} = 0.90$.

3.5.4.4 Resistance Factors for Facing Headed-Studs in Tension

The tensile resistance of headed-studs in SNW facings are selected as $\phi_{FH} = 0.50$ for ASTM A 307 steel and $\phi_{FH} = 0.59$ for ASTM A 325 steel, consistent with the approach of adopting load factors for overall stability $\gamma = 1.0$. Note that a resistance factor of $\phi_{FH} = 0.50$ for bolts in tension (both of steel grades ASTM A 307 and ASTM A 325) is included in Section 6.5.4.2 of AASHTO (2007). However, as with previous cases of resistance factors for structural limit states, AASHTO (2007) resistance factors were developed for much higher load factors. Also note that the value $\phi_{FH} = 0.80$ in AASHTO (2007) coincides with the value adopted by Byrne et al. (1998) for the tensile limit state of ASTM A 325 steel headed-studs. Byrne et al. (1998) presented a separate resistance factor for ASTM A 307 steel at $\phi_{FH} = 0.67$.

3.5.5 Preliminary Values of Resistance Factors for Nail Pullout

Of the various calibration schemes that can be used to establish a resistance factor, a preliminary calibration was performed based on factors of safety (i.e., Calibration Method B). This procedure was used to develop the resistance factors for the structural limit states presented in Section 3.5.4.

In the case of the pullout resistance of soil nails, this factor can be computed from the LRFD equation assuming that nail loads are directly affected by the load factors, or:

$$\phi_{PO} \geq \frac{\sum \gamma_i Q_i}{FS_{PO} \sum Q_i} \tag{3-73}$$

If loads are comprised of permanent dead (Q_{DC}) and live loads (Q_{LL}), equation 3-73 can be expressed as:

$$\phi_{PO} \geq \frac{(\gamma_{DC} Q_{DC} + \gamma_{LL} Q_{LL})}{FS_{PO}(Q_{DC} + Q_{LL})} \tag{3-74}$$

which can be simplified as:

$$\phi_{PO} \geq \frac{\left(\dfrac{\gamma_{DC} Q_{DC}}{Q_{LL}} + \gamma_{LL}\right)}{FS_{PO}\left(\dfrac{Q_{DC}}{Q_{LL}} + 1\right)} \tag{3-75}$$

Equation 3-75 is useful because the load ratio Q_{DC}/Q_{LL}, not the actual magnitude of loads, is needed to estimate the resistance factor. If live loads are absent:

$$\phi_{PO} = \frac{\gamma_{DC}}{FS} \tag{3-76}$$

Table 3-6. Summary of pullout resistance factors ϕ_{PO} based on factors of safety.

Q_{DC}/Q_{LL}	Factor of Safety, FS_{PO}				
	1.50	1.75	2.00	2.25	2.50
	Resistance Factor, ϕ_{PO}				
3	0.92	0.79	0.69	0.61	0.55
4	0.90	0.77	0.68	0.60	0.54
5	0.89	0.76	0.67	0.59	0.53
10	0.86	0.74	0.65	0.58	0.52
∞	0.83	0.71	0.63	0.56	0.50

Equations 3-74 and 3-75 can be employed to derive resistance factors for a load combination of permanent dead loads and live loads per AASHTO (2007) Strength I Limit State (from $\gamma_{DC} = 1.25$ and $\gamma_{LL} = 1.75$). These load factors were used because they may represent typical cases of loading for a bridge abutment. The selected load ratio and the safety factor for pullout vary within the range of safety factors typically used for retaining structures. A summary of results is presented in Table 3-6.

Results plotted on Figure 3-16 show that the resistance factor is relatively insensitive to the load ratio for $Q_{DC}/Q_{LL} \geq 2.5$. Withiam and Nowak (2004) reported similar trends. For typical values $FS_{PO} = 2.0$ and $Q_{DC}/Q_{LL} \geq 2.5$, the range of calculated ϕ_{PO} is 0.63 to 0.70, with an average of approximately 0.65. Note that, for the case of a service limit state [i.e., $\gamma_{DC} = \gamma_{LL} = 1.0$ for Service I Limit State, per AASHTO (2007)]:

$$\phi_{PO} = \frac{1}{FS_{PO}} \tag{3-77}$$

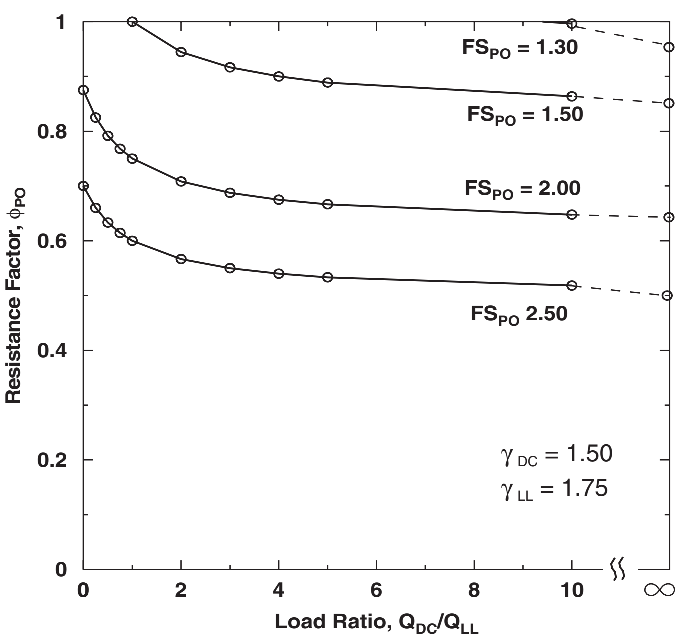


Figure 3-16. Pullout resistance factors as a function of load ratio and pullout safety factor.

For a typical $FS_{PO} = 2.0$, the pullout resistance factor is $\phi_{PO} = 0.5$.

Load ratios of 2.5 to 3.0 have been selected in the past for calibrating resistance factors of shallow foundations (e.g., Barker et al., 1991) and deep foundations (Paikowsky et al., 2004). SNWs used in highway applications (e.g., SNWs used as bridge abutments or retaining structures) have loads with relatively large load ratios; therefore, the above range is consistent with previous experience. Typical load ratios for SNWs that are part of a bridge abutment are significantly larger than $Q_{DC}/Q_{LL} = 2.5$, with the ratio tending to increase with the bridge length. For SNWs that are constructed along roadways and have very small or no traffic loads, the ratio Q_{DC}/Q_{LL} can be very large.

The range of resistance factors in Table 3-6 overlaps with the values of nominal pullout resistance of ground anchors to be used for presumptive nominal resistance values, which are included in Table 11.5.6-1 of AASHTO (2007) and presented below for various soil types:

- Cohesionless soils: $\phi_{PO} = 0.65$
- Cohesive soils: $\phi_{PO} = 0.70$
- Rock: $\phi_{PO} = 0.50$

A subsequent section presents the results of a full calibration of ϕ_{po} based on empirical data and reliability-based methods.

A summary of resistance factors for SNWs is included in Table 3-7.

3.6 Development of Soil Nail Test Pullout Resistance and Load Databases

3.6.1 Introduction

This section presents the basis for the development of databases of soil nail pullout resistances and loads. These databases were developed based on soil nail load-test results and case histories. The objective in compiling these databases was to develop a basis for preparation of probabilistic distributions and statistical parameters to be used in

Table 3-7. Summary of preliminary resistance factors for SNWs.

Limit State	Resistance	Condition		Resistance Factor	Value
Soil Failure	Sliding	All		ϕ_τ	0.90
	Basal Heave	All		ϕ_b	0.70
Overall Stability	NA	Slope does not support a structure		ϕ_s	0.75 (1)
		Slope supports a structure		ϕ_s	0.65 (2) (3)
		Seismic		ϕ_s	0.90 (4)
Structural	Nail in Tension	Static	Mild steel bars – Grades 60 and 75 (ASTM A 615)	ϕ_T	0.56 (5)
			High-resistance - Grade 150 (ASTM A 722)	ϕ_T	0.50 (5)
		Seismic	Mild steel bars – Grades 60 and 75 (ASTM A 615)	ϕ_T	0.74 (5)
			High-resistance - Grade 150 (ASTM A 722)	ϕ_T	0.67 (5)
	Facing Flexure	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FF}	0.67 (5)
	Facing Punching Shear	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FP}	0.67 (5)
	Facing Headed-Stud Tensile	A307 Steel Bolt (ASTM A 307)		ϕ_{FH}	0.50 (5)
		A325 Steel Bolt (ASTM A 325)		ϕ_{FH}	0.59 (5)
	Pullout	Presumptive nominal values		ϕ_{PO}	0.50–0.70 (5) (6)

Notes:
(1) AASHTO (2007) also considers this value when geotechnical parameters are well defined.
(2) AASHTO (2007) also considers this value when geotechnical parameters are based on limited information.
(3) For temporary SNWs, use $\phi_s = 0.75$.
(4) Per AASHTO (2007) but subject to modifications after new Standard is in place. A value $\phi_s = 1.00$ may be acceptable, as long as permanent deformations are calculated (see Anderson et al., 2008) and are found not to be excessive. For temporary structures under seismic loading, also use $\phi_s = 1.00$.
(5) Calibrated from safety factors.
(6) Preliminary values that will be updated with a reliability-based calibration.

the calibration of pullout resistance and load factors (specifically, the bias for these quantities). The soil nail pullout resistance database was developed by considering values of pullout resistance from several different sources, including (i) recommended ranges of values of pullout resistance for certain soil types commonly used in practice, as described subsequently; (ii) relationships between pullout resistance and field-measured soil parameters; and (iii) pullout resistance values obtained from verification and proof load tests. The soil nail load database was developed based on information obtained from several instrumented walls. The following subsections present a discussion on the main factors that influence pullout resistance and provide typical values of pullout resistances, as well as correlations between pullout resistance and several typical geotechnical engineering parameters. Additionally, a background of soil nail load testing is provided along with a description of the database of soil nail pullout resistance. The databases are included in Appendix C.

3.6.2 Soil Nail Bond Resistance: Influencing Factors and Typical Values

3.6.2.1 Influencing Factors

The nominal pullout capacity of a soil nail develops behind a slip surface and is a direct function of the bond resistance, q_u, which is the mobilized shear resistance along the interface between a grouted nail and the surrounding soil. Because the focus of this document is current U.S. practice, only drilled and gravity-grouted soil nails are considered. For these types of soil nails, the nominal bond resistance is affected by numerous factors, including:

- Conditions of the ground around soil nails, including:
  - Soil type;
  - Soil characteristics;
  - Magnitude of overburden; and
- Conditions at time of soil nail installation, including:
  - Drilling method (e.g., rotary drilled, driven casing, etc.);
  - Drill-hole cleaning procedure;

– Grout injection method (e.g., under gravity or with a nominal, low pressure);
– Grouting procedure (e.g., tremie method); and
– Grout characteristics (e.g., grout workability and compressive strength).

The soil type and conditions of the subsurface soils around the nails also affect the bond resistance. The magnitude of overburden has a larger effect on the nominal bond resistance of granular soils than on that of fine-grained soils. The nominal bond resistance of granular soils is largely influenced by the soil friction angle of the soil around the nail and the magnitude of overburden. While some publications (e.g., Clouterre, 2002) assign for design purposes a linear relationship between the nominal bond resistance of granular soils and its frictional component, the relationship is more complex than a liner relationship because other factors, including construction techniques and grout characteristics, also affect the nominal bond resistance in granular soils. The nominal bond resistance of nails installed and grouted in fine-grained soils is in general a fraction of the undrained shear strength of the soil, S_u. In relatively soft, fine-grained soils (i.e., cohesive), the ratio of bond resistance to soil undrained shear strength, q_u/S_u, is higher than in relatively stiff, fine-grained soils. The influence of construction techniques (i.e., drilling, installation, and grouting) on the bond resistance is more difficult to ascertain in these soils.

The nominal bond resistance of a soil nail can be estimated from the following sources:

- Typical values published in the literature,
- Relationships between q_u and parameters obtained from common field tests, and
- Soil nail load tests.

Besides these sources, some design engineers estimate the nominal bond resistance based on local experience, particularly in areas where some regional practice exists. In addition, the means and methods of an SNW contractor may affect the performance of the structure, including the nominal bond resistance. The nominal bond resistance is rarely measured in the laboratory because it is difficult to reproduce in the laboratory those key aspects that affect the nominal bond resistance, including field conditions, construction techniques, and grout placement procedures. Laboratory testing procedures to evaluate the nominal bond resistance of soil nails, if ever used, are not standardized.

Estimations of the nominal bond resistance of soil nails from various sources are discussed below.

3.6.2.2 Typical Values Published in Literature

Typical values of bond resistance have been presented in the literature for drilled and gravity-grouted soil nails installed in various types of soils/rocks and for different drilling methods. The most widely used source for typical bond resistance is Elias and Juran (1991), which presents values based on a substantial amount of project experience. Ranges of the nominal bond resistance for various ground conditions and construction techniques are included in Table 3-8 based on this source. The ranges in Table 3-8 are not presented as a function of measurable field parameters. Design engineers should select design values using judgment. In general, the values in Table 3-8 incorporate a certain degree of conservatism. Minimum and maximum values of the nominal bond resistance provided in this table correspond approximately to the least favorable and most favorable conditions in each case; the average of the range may be used as a preliminary value for design.

In addition, the Post-Tensioning Institute (PTI, 2005) presented presumptive values of the nominal bond strength of ground anchors that were grouted under gravity. These values can be also used as preliminary values for soil nails.

3.6.2.3 Correlations between Nominal Bond Resistance and Common Geotechnical Field Tests

Soil nail bond resistance, q_u, has been correlated to standard geotechnical field testing techniques, including the Pressuremeter Test (PMT) and the Standard Penetration Test (SPT). These correlations provide typical bond resistance of soil nails for a wide range of subsurface conditions, as described in the following subsections.

Correlation between q_u and Pressuremeter Test Results. A correlation between the PMT limit pressure, p_L, and q_u was developed for various soil types (Clouterre, 2002). The correlation has the following format:

$$q_u = a(p_L)^b \tag{3-78}$$

where a and b are parameters corresponding to various soil types, and p_L is the PMT limit-pressure (e.g., ASTM D 4719-87, "Standard Test Method for Pressuremeter Testing in Soils"; Briaud, 1989 and 1992). The limit-pressure is defined as the theoretical pressure at which the soil yields horizontally in the PMT. The correlation above was developed for sand, clay, gravel, and weathered rock, based on soil nail load and PMT tests that were conducted concurrently at the same site (Clouterre, 2002).

Equation 3-78 is unit dependent; therefore, when working with English units, p_L must be in tons per square foot (tsf) to obtain the nominal resistance, q_u, in pounds per square inch (psi). When working with SI units, p_L must be in megapascals (MPa) to obtain q_u in kilopascals (kPa). Table 3-9 presents the a and b parameters to be used with

Table 3-8. Estimated nominal bond resistance for soil nails in soil and rock.

Material	Construction Method	Soil/Rock Type	Nominal Bond Resistance, q_u (psi)
Rock	Rotary Drilled	Marl/limestone	45 – 58
		Phyllite	15 – 45
		Chalk	75 – 90
		Soft dolomite	60 – 90
		Fissured dolomite	90 – 145
		Weathered sandstone	30 – 45
		Weathered shale	15 – 22
		Weathered schist	15 – 25
		Basalt	75 – 90
		Slate/hard shale	45 – 60
Cohesionless Soils	Rotary Drilled	Sand/gravel	15 – 26
		Silty sand	15 – 22
		Silt	9 – 11
		Piedmont residual	6 – 17
		Fine colluvium	11 – 22
	Driven Casing	Sand/gravel	
		low overburden [(1)]	28 – 35
		high overburden [(1)]	40 – 62
		Dense Moraine	55 – 70
		Colluvium	15 – 26
	Augered	Silty sand fill	3 – 6
		Silty fine sand	8 – 13
		Silty clayey sand	9 – 20
Fine-Grained Soils	Rotary Drilled	Silty clay	5 – 7
	Driven Casing	Clayey silt	13 – 20
	Augered	Loess	4 – 11
		Soft clay	3 – 4
		Stiff clay	6 – 9
		Stiff clayey silt	6 – 15
		Calcareous sandy clay	13 – 20

Note: [(1)] Low and high overburden were not originally defined in Elias and Juran (1991).

English or SI units for ground conditions that include clay, gravel, and weathered rock.

Figures 3-17 through 3-20 show the relationship between q_u (in psi) and p_L (in tsf) for the mentioned soil types. The figures also show the data on which these correlations are based, as well as the 95% confidence intervals associated with each correlation. The correlations of q_u shown in these figures are non-linear functions of p_L (or $b \neq 1$).

Table 3-9. Parameters *a* and *b* for equation 3-78, correlation between q_U and p_L.

Material Type	*a*		*b*
	English Units [(1)]	SI Units [(2)]	
Sand	6.90	119	0.390
Gravel	5.87	122	0.469
Clays	5.89	120	0.461
Weathered Rock	6.33	177	0.595

Notes: (1) Enter p_L in tsf to obtain q_u in psi.
(2) Enter p_L in MPa to obtain q_u in kPa.

Because the PMT is not routinely used in geotechnical investigations for soil nail projects in the United States, the correlation with the PMT has not been widely used in this country.

Correlation between q_u and the Standard Penetration Test Results. Some correlations between the SPT (ASTM D 1586, "Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils") blow count (i.e., "*N*" value, expressed as number of blows per foot) and the nominal bond resistance have been developed. The SPT is the most commonly used field technique to assess subsurface conditions for soil nail projects in the United States. The SPT is routinely utilized in SNW projects for soil classification purposes and for soil sampling to estimate other engineering parameters. However, the estimation of the nominal bond resistance of soil nails using the SPT is uncommon.

Sabatini et al. (1999) presented presumptive, ultimate values of the load transfer rate (r_{PO}) of small-diameter, straight, gravity-grouted ground anchors installed in soils. The load transfer rate is equal to the nominal bond resistance, q_u, times the perimeter of the grouted nail ($2\pi D_{DH}$, where D_{DH} is the

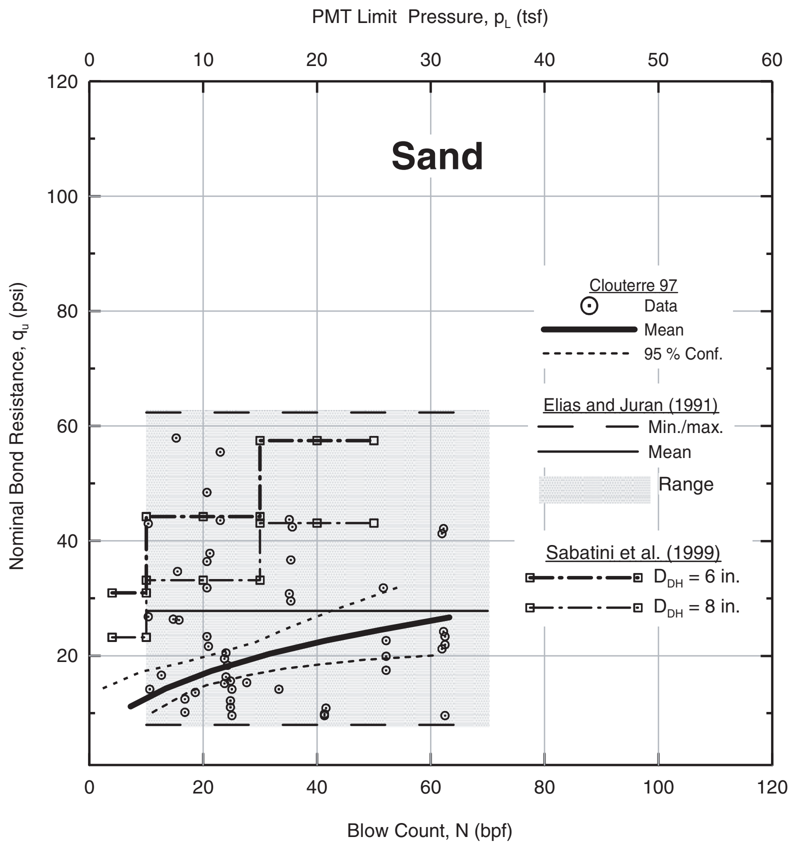


Figure 3-17. Relationship between q_u, p_L, and N for sand.

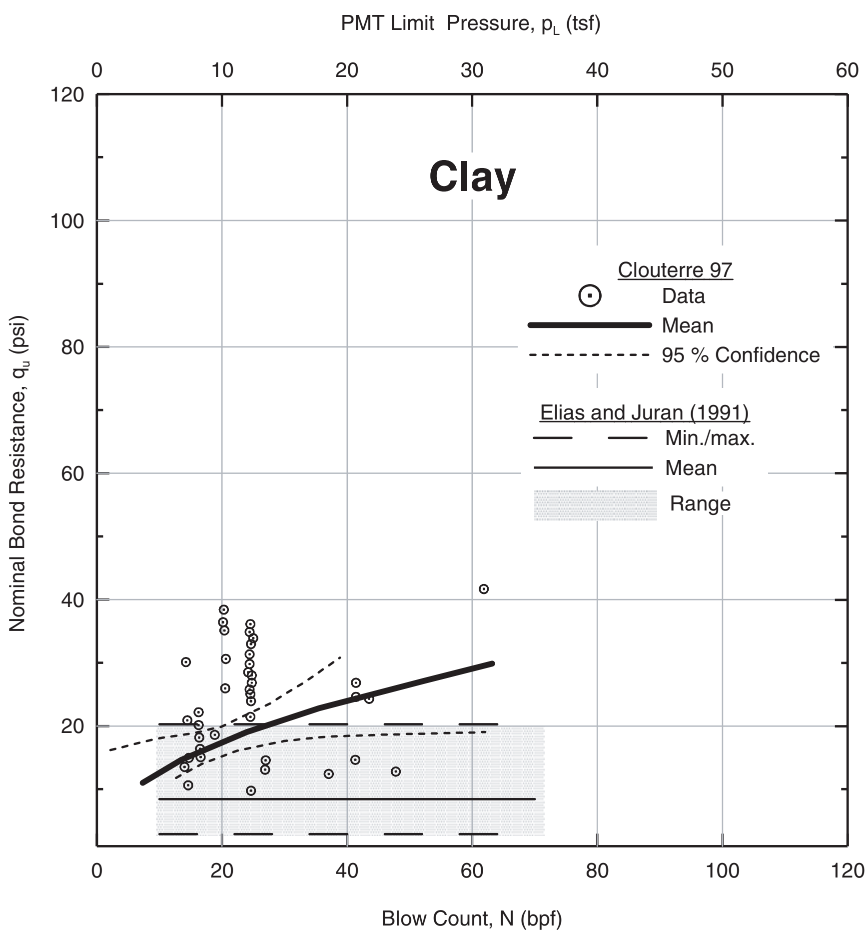


Figure 3-18. Relationship between q_u, p_L, and N for clay.

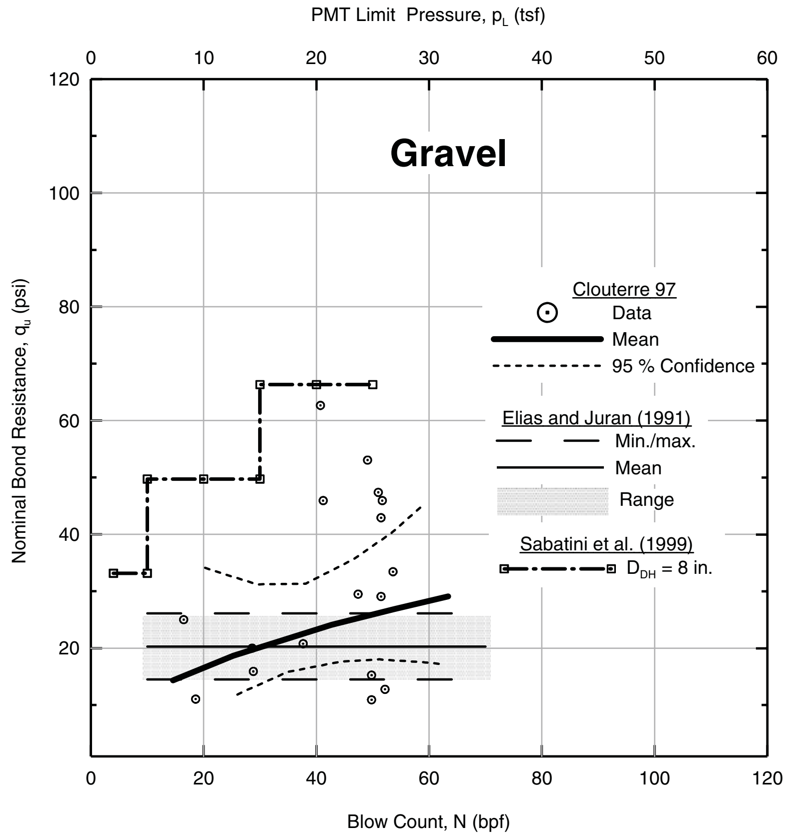


Figure 3-19. Relationship between q_u, p_L, and N for gravel.

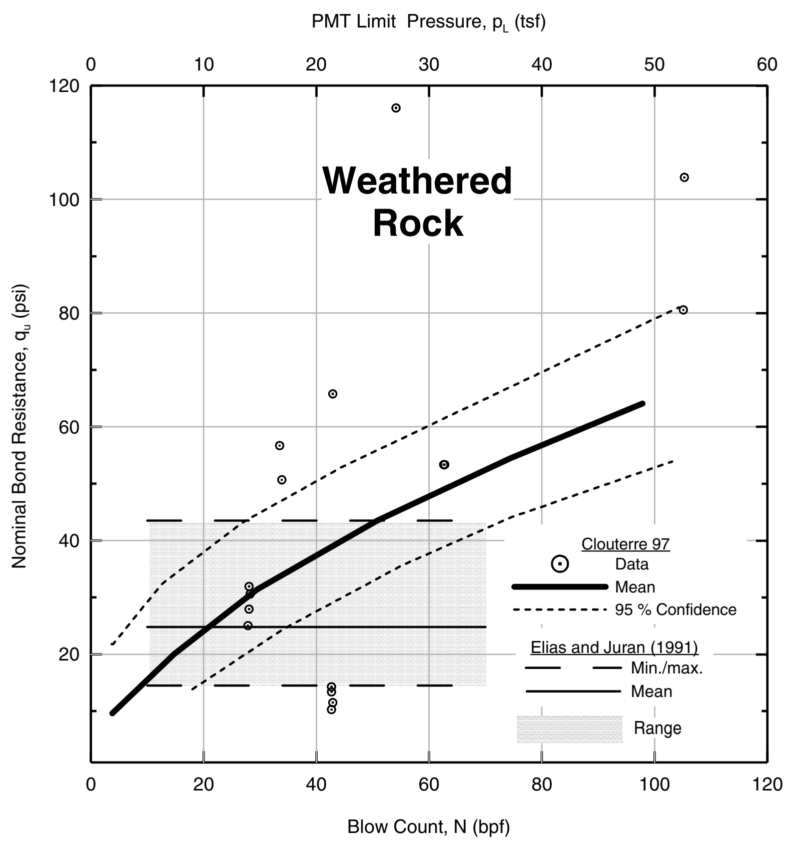


Figure 3-20. Relationship between q_u, p_L, and N for weathered rock.

Table 3-10. Presumptive values of soil nail load transfer rate in soils.(1)

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

Notes: (1) Modified after Sabatini et al. (1999). Values are for small-diameter, straight shaft, gravity-grouted ground anchors installed in soil.

(2) SPT values are corrected for overburden pressure.

diameter of the drill-hole). Table 3-10 presents presumptive values of r_{PO} (i) for four different soil types and (ii) as a function of soil density/consistency and N ranges. The soil types included in this table are sand/gravel, sand, sand and silt, and silt-clay mixtures of low plasticity/silt mixtures. Note that the ultimate load transfer rates in Table 3-10 are in units of force per unit length of bonded reinforcement.

Although the values contained in Table 3-10 were intended for the design of ground anchors, these presumptive values can also be used for the preliminary design of soil nails because the test ground anchors, on which the results are based, were grouted under gravity, which is the typical scenario for soil nails. However, designers must be cautious in using these values as some differences exist between the conditions for ground anchors and soil nails. The N-values included in Table 3-10 are related to relatively deep soils where the bonded length of a ground anchor would be installed, under relatively large in-situ soil overburden. However, soil nails are commonly shorter than ground anchors, tend to be grouted up the excavation face, and thereby their bonded lengths are in general under smaller soil overburden. Therefore, the values in Table 3-10 are probably somewhat unconservative for soil nails.

A correlation between SPT and q_u can be derived by applying relationships between the PMT p_L and SPT N-values. Briaud (1989) presented a correlation that related: p_L (tsf) = 0.5 N [or approximately p_L(MPa) ≈ 0.05 N]. By replacing this correlation in the PMT-based correlations with q_u, the nominal bond resistance of a soil nail can be estimated from N-values as:

$$q_u\,(\text{psi}) = a\left(\frac{N}{2}\right)^b \tag{3-79}$$

Figures 3-17 through 3-20 show a comparison of the original data obtained in 1995 (identified as Clouterre 97 in figures and published as Clouterre, 2002) and N vs. q_u correlations based on the Briaud (1989) N-p_L correlation. These figures also show the range, maximum, minimum, and average values of the q_u estimates provided in Table 3-8. The Elias and Juran (1991) values for sand appear to cover the range of all data points presented by Clouterre (2002) and to lie above the Clouterre (97) p_L vs. q_u curves for sand. For clays, the Elias and Juran (1991) values lie on the lower side of the Clouterre data points and correlation. Similar observations can be made for gravel and weathered rock. The ranges proposed by Sabatini et al. (1999) for two cases of drill-hole diameters, D_{DH}, are also presented in these figures.

3.6.3 Background of Soil Nail Load Testing

3.6.3.1 General

Load testing of soil nails consists of applying a tensile force to selected, individual bars in a controlled manner while measuring the developed forces and bar elongations with the purpose of verifying the pullout resistance along the bonded, grouted bar length. Note that soil nails are only partially grouted for testing purposes. The specific objectives of soil nail load testing are to:

(i) verify that the presumptive design load, *DL*, is achieved;
(ii) confirm that the *DL* is achievable for the installation means and materials specified in construction documents or proposed by the contractor;
(iii) investigate whether the soils subjected to testing loads experience excessive time-related deformation; and
(iv) verify that *DL*s are achieved if a different soil type is encountered or if construction procedures are modified.

In the definition above, the design load, *DL*, refers to the maximum tensile load that is expected to be achieved for service conditions (i.e., not ultimate conditions). *DL* develops along the bonded nail length, L_B, and is a fraction of the presumptive nominal bond resistance. In an ASD scenario, a reduced nominal bond resistance would correspond to the allowable bond strength.

The following types of load tests are performed on SNW projects: (i) verification load tests; (ii) proof load tests; and (iii) creep tests. Procedures for soil nail load testing are described in the suggested SNW construction specifications included in Appendix B. Detailed descriptions of soil nail testing are provided in Byrne et al. (1998) and Lazarte et al. (2003). Descriptions of the mechanisms participating in soil load tests are presented in the following section.

3.6.3.2 Mechanisms in Soil Nail Load Tests

A soil nail load test is illustrated in Figure 3-21. The drill-hole is assumed to have a uniform diameter, D_{DH} [Figure 3-21(a)]; a load, P, is applied and measured at the front end of the soil nail bar of length L_{tot}; the bar is partially bonded and unbonded in the respective lengths L_B and L_U. The bar elongation, Δ_{tot}, at the distal end of the bar is measured at the front end.

The bond shear stress $q(x)$ is a function of the coordinate x (measured from the back end of the bar) and is mobilized along the grout-soil interface of the bonded length, L_B [Figure 3-21(b)]. Actual distributions of the mobilized bond shear stress can be complex and depend on several factors, including bonded length, magnitude of the applied tensile force, grout characteristics, and soil conditions (e.g., Sabatini et al., 1999; Woods and Barkhordari, 1997). However, for design purposes, the mobilized stress is assumed to be constant along the bonded length [Figure 3-21(b)]. With this assumption, the nominal bond resistance, q_u, is the average of the mobilized stress distribution at the limit state.

The force per unit length (equivalent to the transfer load rate, r_{PO}, defined previously) is obtained by multiplying the stress $q(x)$ by the perimeter of the nail-soil interface, or:

$$r_{PO} = \pi\, q(x) D_{DH} \tag{3-80}$$

where all variables were defined previously. The increment of tensile force, dT, along a differential increment of length, dx, [Figure 3-21(a)] is:

$$dT = \pi\, D_{DH}\, q\, dx \tag{3-81}$$

The nail tensile force $T(x)$ at coordinate x can be obtained by integration. Assuming that $q(x)$ is uniform along the length of the drill-hole, $T(x)$ is:

$$T(x) = \int_0^x \pi\, D_{DH} q\, dx = \pi\, D_{DH} q\, x \tag{3-82}$$

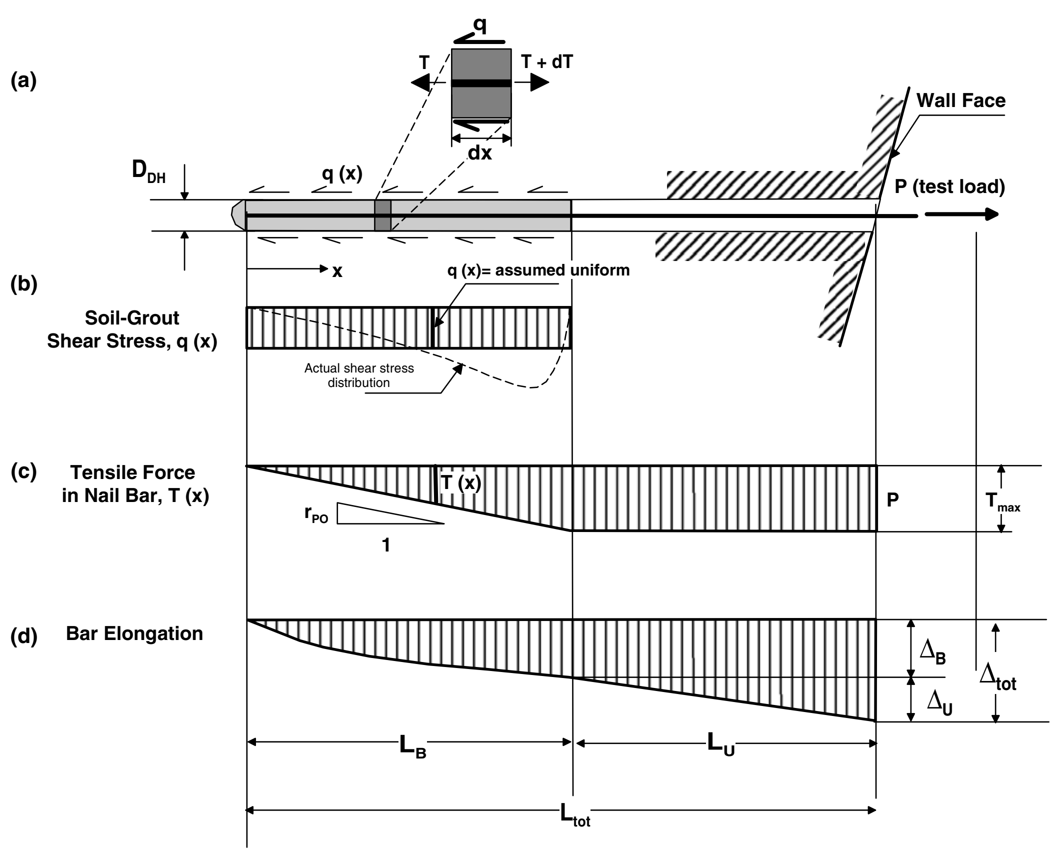


Figure 3-21. Loads and elongation in a soil nail load test.

The pullout capacity during a test, R_{PO}, results when the force $T(x)$ achieves a maximum value, or:

$$R_{PO} = T_{max} = \pi\, D_{DH}\, q_U\, L_B \tag{3-83}$$

As shown in Figure 3-21, T_{max} occurs at the end of the bonded length, remains approximately constant along the unbonded length, and is equal to the test load, P. The bond stress is then related to the test load as:

$$q = \frac{P}{\pi\, D_{DH}\, L_B} \tag{3-84}$$

The total elongation, Δ_{tot}, [Figure 3-21(d)] comprises the elongation Δ_U developing along the unbonded length L_U and the elongation Δ_B developing along the bonded length L_B.

Elongation Δ_U occurs as the steel bar deforms in tension. Δ_U remains within the elastic range as long as the nominal yield resistance of the bar is not exceeded. In general, test loads and the bonded length are designed to prevent the bar from exceeding its yield resistance during the test. This elongation is expressed as:

$$\Delta_U = \frac{P}{E\, A_t} \times L_U \tag{3-85}$$

where E is the elastic modulus of the nail bar, and A_t is the cross-sectional area of the nail bar.

Elongation Δ_B reflects the bar elongation in the bonded length, the grout deformation, the relative deformation or slippage between the grout and the soil, and the soil shear deformation around the nail. This elongation can be calculated as:

$$\Delta_B = \Delta_{tot} - \Delta_U \tag{3-86}$$

The relationship between the elongation Δ_B and applied loads is mostly linear when the applied loads are small; however, it tends to become non-linear for large loads because the typically non-linear response of the soil (at the soil-grout interface and around the soil nail) becomes more prominent. The relative movement between the nail bar and grout is negligible because of the high resistance to pullout of threaded bars embedded in grout.

Elongation Δ_B can be normalized as:

$$\varepsilon_B = \frac{\Delta_B}{L_B} \times 100 \tag{3-87}$$

The data obtained in a load test includes the applied load P and the total elongation (see example on Figure 3-22). The applied load P is increased in predetermined increments that are usually expressed as fractions or percentages of DL (see Appendix B for a typical schedule of test loads). Using the bar

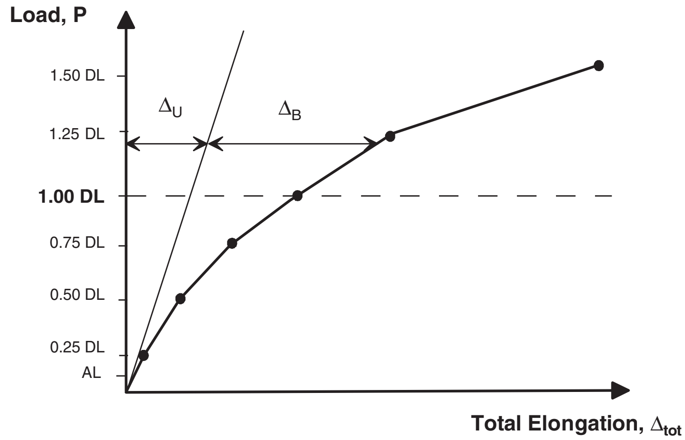


Figure 3-22. Reduction of soil nail load-test data.

geometric and material properties, it is possible to separate the total, measured elongation in the bonded and unbonded elongations, as shown on Figure 3-22.

3.6.3.3 Verification Tests

Verification tests are conducted to (i) confirm that the installation methods used by the SNW contractor are adequate for the project conditions; (ii) estimate or confirm the nominal pullout resistance used for design if verification tests are performed in the design phase; (iii) verify the presumptive values of pullout resistance used in design; and (iv) identify potential problems during soil nail installation.

The number of verification tests that are conducted in each project depends on several factors, including the project magnitude, variability of ground types at the site, presence of unusual ground conditions, and familiarity of the contracting agency with SNW technology. The common practice is to request that the contractor conduct a minimum of two verification tests in each major soil layer. Appendix B provides guidance on the minimum number of verification tests to perform.

In verification tests, the applied test load is increased typically up to 200 percent of DL. In verification tests where the applied loads do not result in pullout, the ratio of maximum load to DL is ≤ 2.0. Typically, true ultimate resistance conditions are not always achieved during verification tests. If the applied loads lead to a premature failure condition in the test, verification tests can, in principle, provide a direct measurement of the nominal bond resistance. Test nails used in verification tests do not become part of the permanent work but are "sacrificial" because a test load of 200% of DL is considered to be excessive for these nails to be used as part of the long-term system.

In some projects, the contractor may elect to apply test loads beyond 200% of DL, thus creating more opportunities to achieve the ultimate pullout strength. However, test loads higher than 200% of *DL* are rarely applied.

3.6.3.4 Proof Tests

During construction, proof tests are conducted on selected production nails, most commonly in every excavation lift. The maximum test load in proof tests is typically 150% of *DL*. Per specifications, proof tests are commonly conducted on a certain minimum percentage of permanent nails (typically 5%). Additional tests may be required when encountered ground conditions differ from those described in contract documents or when the nail installation procedures change, possibly due to the replacement of broken equipment or low productivity. If results of proof tests indicate that construction practices are inadequate or that the presumptive design pullout resistances are not achieved, the nail installation method or nail lengths/diameters are modified accordingly. Load failures during proof testing are rare.

Testing procedures and nail acceptance criteria of proof tests are usually included in specifications. Appendix B provides guidance on acceptance criteria of proof tests. After a proof test is completed, the unbonded length of the bar is grouted. Those test soil nails that are tested and approved are used as permanent nails in the SNW. In the event that a test soil nail is not approved, a new test soil nail must be installed and retested until approval requirements are met.

3.6.3.5 Creep Tests

Creep tests are conducted as part of verification or proof tests to assess the time-dependent elongation of the test nail under constant load. Creep tests are commonly performed to verify that design loads are resisted without excessive deformations occurring in the soils. In creep tests, the movement of the soil nail head is measured over a period of time of usually 10 to 60 minutes while the applied load is held constant. Creep tests can be performed at various levels of the test load; however, as a minimum, one creep test is performed for the maximum applied load test. Although creep tests may provide some indication that a "failure" condition is imminent when the measured nail head movement rates accelerate, this test does not allow for an easy interpretation that the maximum nominal bond resistance is achieved.

3.6.4 Database of Soil Nail Pullout Resistance

3.6.4.1 Introduction

To develop the database of soil nail pullout resistance, a very large volume of information and data was reviewed. This review revealed that the pullout resistance data exhibited scatter and variability when compared with typical conventional field test data. The review showed that it was not possible to derive complete or strong correlations between measured bond resistances and field test data because either the variability was excessive or the data was incomplete, unreliable, or inconsistent. Therefore, the database of soil nail pullout resistance was developed for various soil/rock types solely based on soil nail load-test results, which were obtained from a wide variety of sources. These sources are described in the following paragraphs.

The soil nail load-test results were carefully scrutinized and all germane information was reviewed. The reviewed information included the following:

- Soil nail test results:
  - Load applied to the soil nail, P;
  - Total measured elongation, Δ_{tot};
  - Observations made during tests (e.g., premature failure, proximity to failure); and
  - Design load, *DL*;
- Soil nail data:
  - Diameter of the drill-hole, D_{DH};
  - Nail total length and bonded length, L_{tot} and L_B; and
  - Nail bar diameter, D_B;
- Geotechnical data:
  - Site location;
  - Soil type description;
  - Data contained in geotechnical reports, including boring logs;
  - Blow count (N) and other field test results;
  - Groundwater table location;
  - Plans with SNW and boring locations;
  - Description of nail installation method; and
  - Drawings and specifications of soil nails.

3.6.4.2 Procedure

All data and related documents were checked for completeness and consistency. Data that showed inconsistencies, was incomplete, or was suspected to be inaccurate was disregarded for the database. Although several of the sources produced sufficient information for the objectives of deriving pullout resistance values, most of them lacked details regarding construction procedures and other information (e.g., drilling procedures, clean-up methods of the drill-hole, and information on grout mix and grouting procedures). In an attempt to minimize the effect caused by different construction practices, different levels of workmanship, and different drilling/installation equipment, preference was given to data derived from tests that were obtained in one site by the same contractor and using similar equipment. The data kept for the database was thereby internally consistent. As a result, the scatter in the database was smaller as the effect in the variability caused by construction aspects was reduced.

The data was classified by the predominant soil type in which the nails were installed. Four categories of soil type were considered: sandy soils, sandy/gravelly soils, clayey soils, and weathered rock. Some projects also provided soil nail load-test results for other soil conditions, including loess, cemented soils, and engineered fill. However, because the number of cases for these soil conditions was relatively small and insufficient to provide a trend, this data was not included in the database.

3.6.4.3 Results from Database

The measured and predicted results in the database are presented in Appendix C. Figures 3-23 through 3-25 present graphical representations of the measured and predicted data. The analysis of the measured and predicted pullout resistance allows an assessment of the bias in the resistance estimation. The bias of the pullout resistance data was calculated and plotted as a normal "variate" on the normal standard representation included on Figures 3-26 through 3-29. Log normal curves were plotted side by side next to the data points to verify whether this distribution was adequate. Note that normal distributions would be represented as straight lines on this type of graph. Based on these figures, it was concluded that the lognormal distribution was an acceptable choice to represent the pullout resistance.

The mean, standard deviation, and COV of the bias were obtained for the lognormal distribution for each of the soil types. In establishing these parameters, the lognormal distribution was adjusted to match the lognormal distribution with the lower tail of the resistance bias data points. The statistical parameters for these curves, which are summarized in Table 3-11, are used subsequently to perform the calibration of the pullout resistance factors.

3.6.5 Database of Soil Nail Loads

The statistics of the bias for loads to be used for the calibration of the pullout resistance factor were derived by examining 11 instrumented SNWs in the United States and abroad (Byrne et al., 1998; Oregon DOT, 1999) and by using simplified methods to estimate the maximum loads in the soil nails (Lazarte et al., 2003).

The maximum load in the nails was based on values presented in those reports. Byrne et al. (1998) provided a normalized distribution of measured soil nail loads, which is reproduced in Figure 3-30. The predicted nail load was obtained using simplified charts developed to estimate the maximum load occurring in soil nails (Lazarte et al., 2003) using the conditions that were present in the instrumented walls. Both measured and predicted maximum nail loads are shown in Figure 3-31. The cases are summarized in Table 3-12. The bias of these data was calculated and plotted as a normal

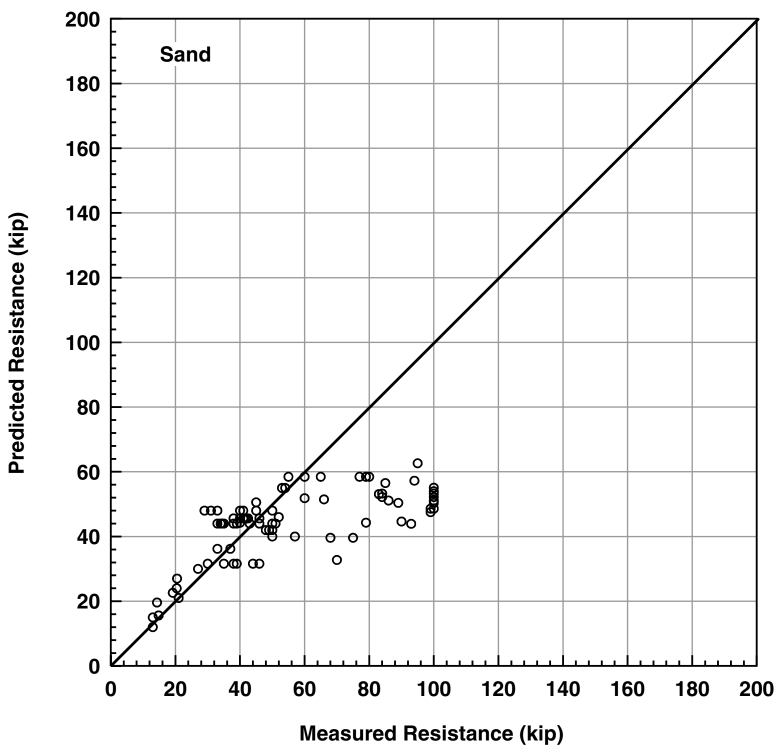


Figure 3-23. Measured and predicted pullout resistance—sand.

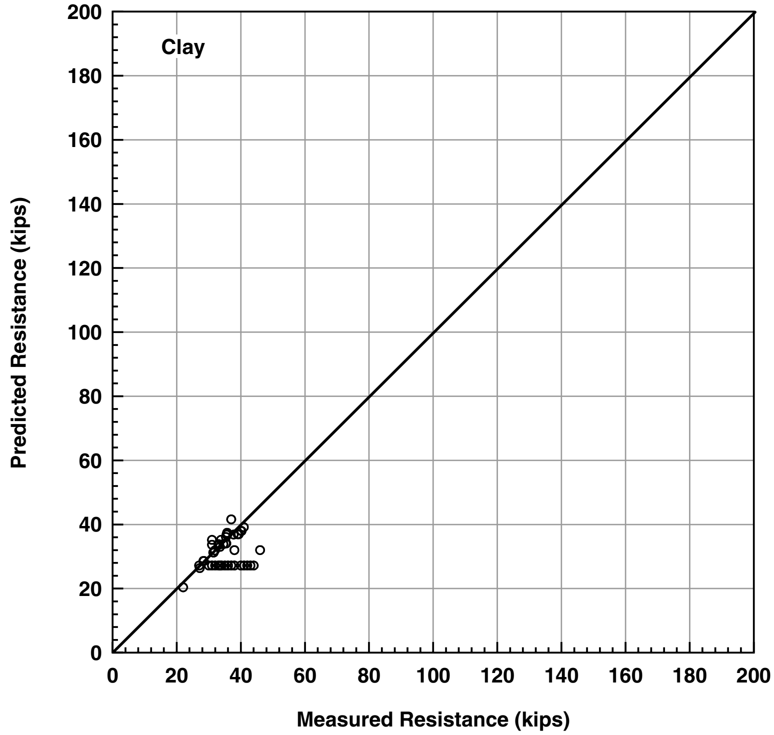


Figure 3-24. Measured and predicted pullout resistance—clay.

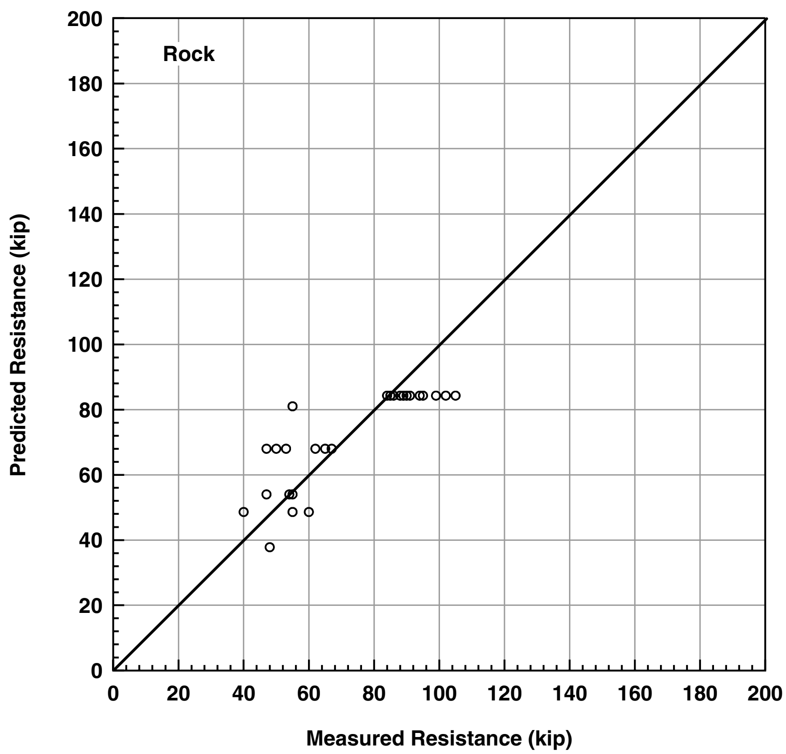


Figure 3-25. Measured and predicted pullout resistance—rock.

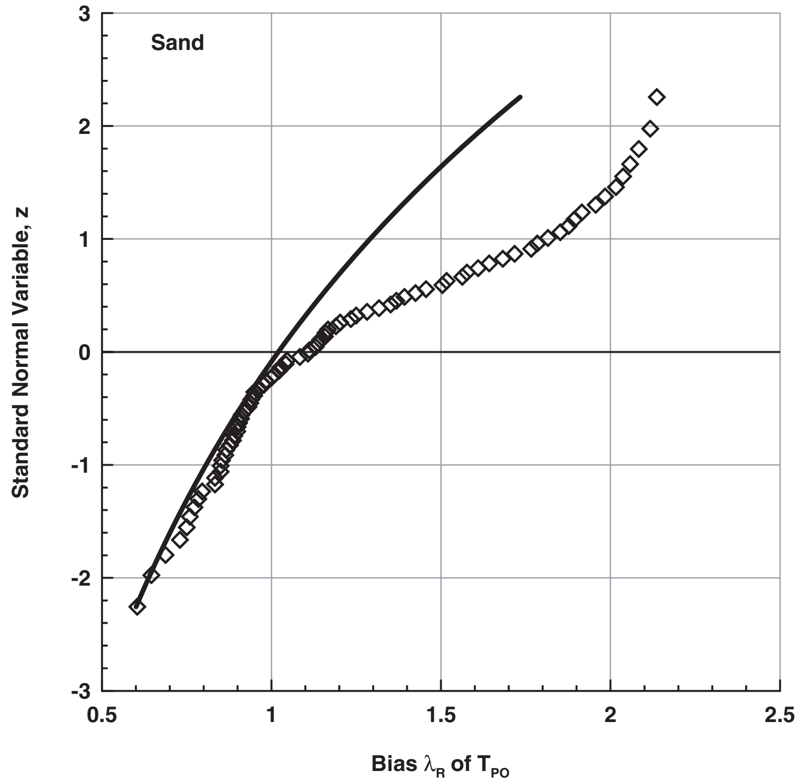


Figure 3-26. Bias λ_R of pullout resistance—sand.

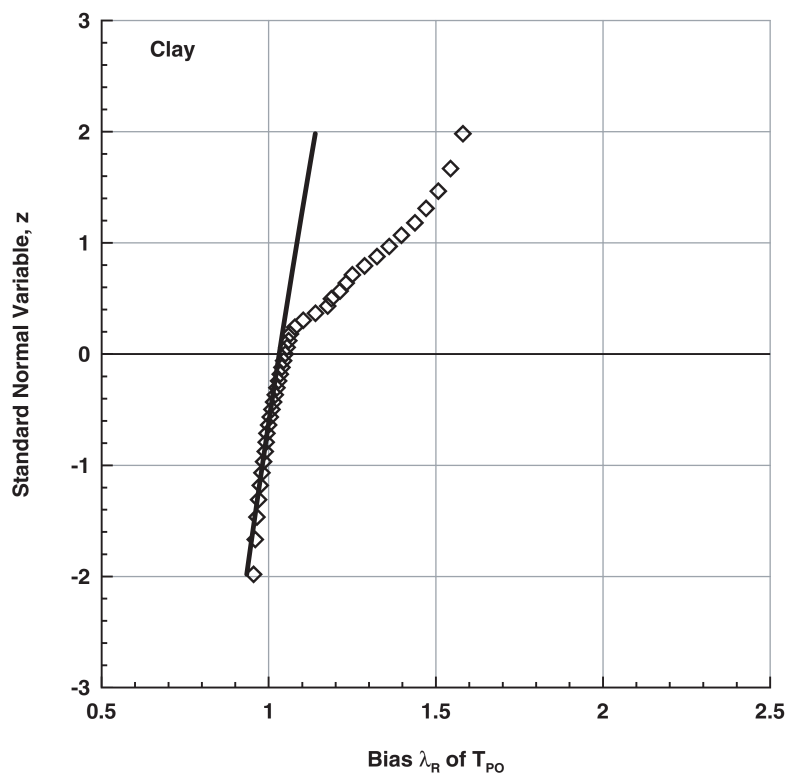


Figure 3-27. Bias λ_R of pullout resistance—clay.

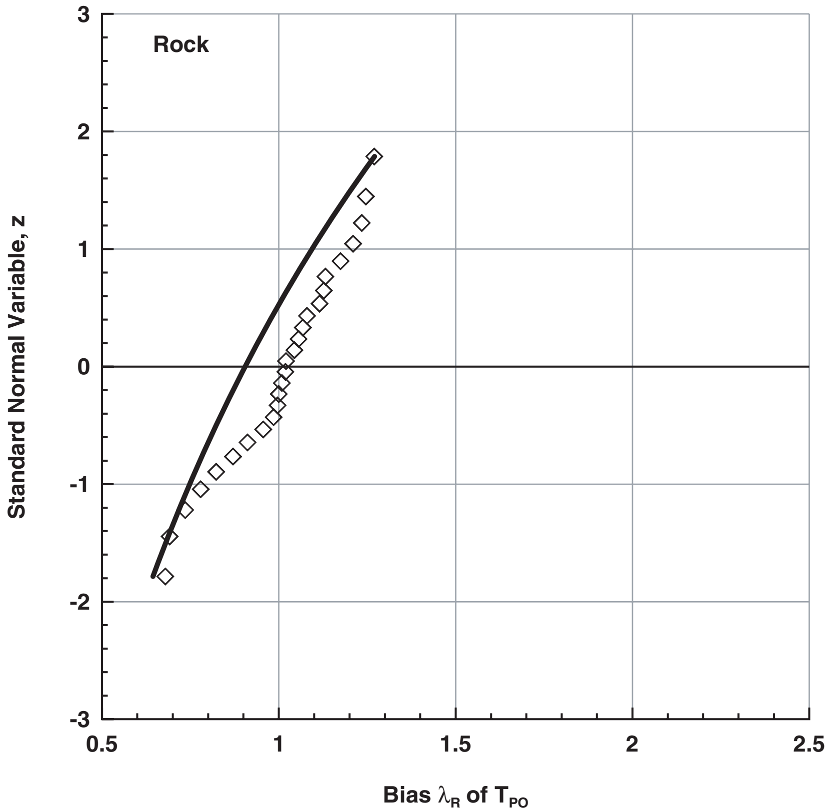


Figure 3-28. Bias λ_R of pullout resistance—rock.

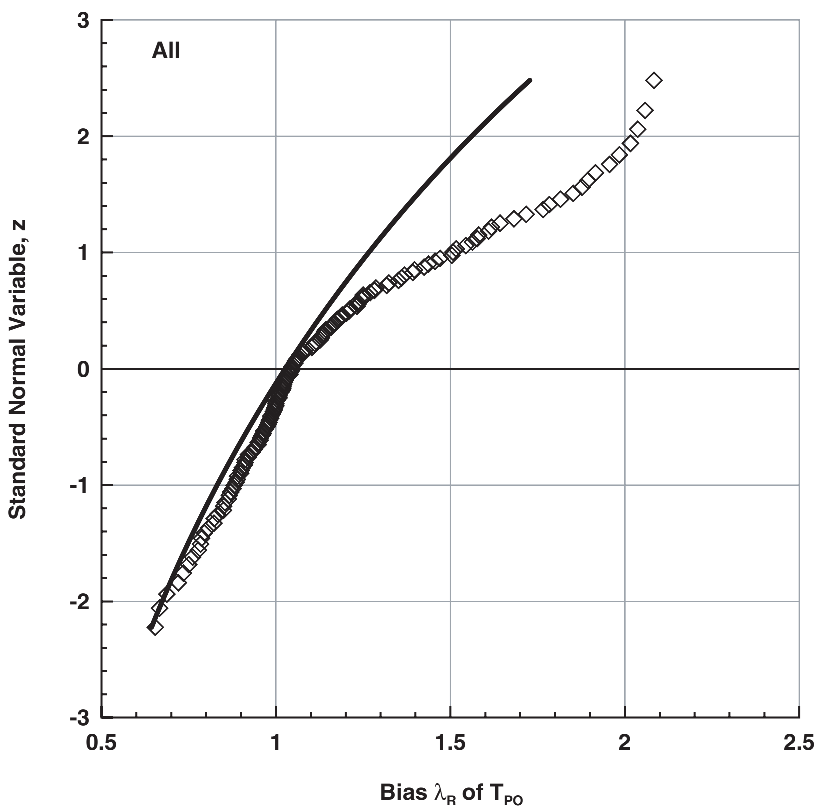


Figure 3-29. Bias λ_R of pullout resistance—all materials.

Table 3-11. Statistics of bias for nominal bond strength.

Material	Resistance Parameters						
	Number of Points in Database	Distribution Type	Mean of Bias	Standard Deviation	Coefficient of Variation	Log Mean of Bias	Log Standard Deviation
	N		λ_R	σ_R	COV_R	μ_{ln}	σ_{ln}
Sand and Sand/Gravel	82	Lognormal	1.050	0.25	0.24	0.02	0.24
Clay/ Fine-Grained	45	Lognormal	1.033	0.05	0.05	0.03	0.05
Rock	26	Lognormal	0.920	0.18	0.19	–0.10	0.19
All	153	Lognormal	1.050	0.22	0.21	0.03	0.21

variate on Figure 3-32. The distribution selected to fit the data was also a lognormal distribution that was adjusted to match the upper tail of the load bias distribution.

The bias calculated for each of these cases is presented in Table 3-13. Statistical parameters are summarized in Table 3-14. These parameters are also used in the calibration.

3.7 Calibration of Pullout Resistance Factors

3.7.1 Introduction

This section presents the results of the calibration of pullout resistance factors. The calibration was conducted applying the calibration framework developed by Allen et al. (2005), which was presented earlier in this chapter. Monte Carlo simulations were conducted to improve initial values presented previously in this chapter.

3.7.2 Description of Calibration Process

The calibration was performed using the following steps:

Step 1: Establish a limit state function;
Step 2: Develop PDFs and statistical parameters for loads and resistances;
Step 3: Select a target reliability index for SNW design;
Step 4: Establish load factors;
Step 5: Best-fit cumulative density functions to data points;
Step 6: Conduct Monte Carlo simulation;
Step 7: Compare computed and target reliability indices; and

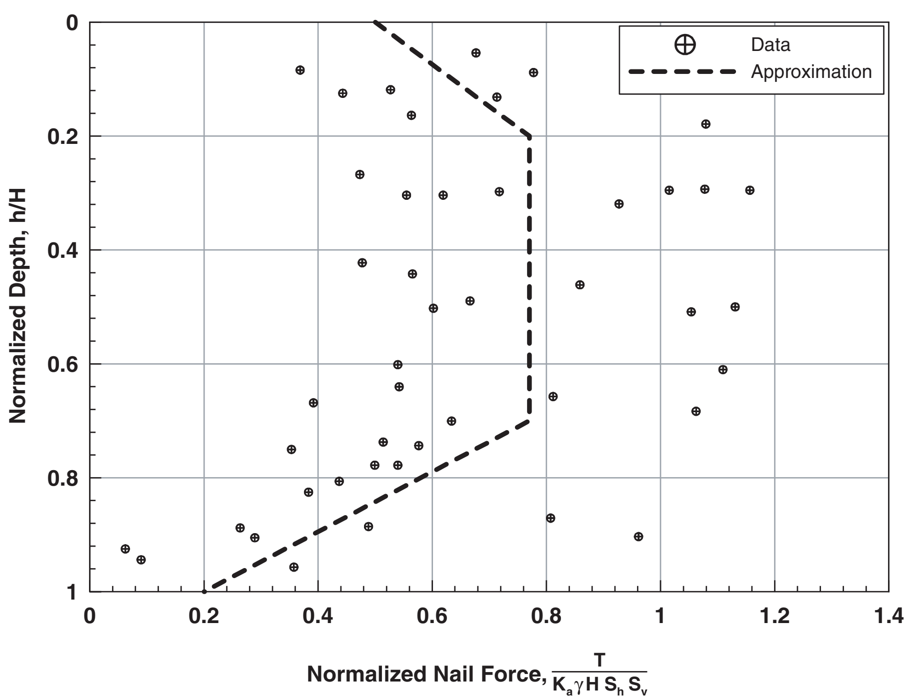


Figure 3-30. Summary of tensile forces measured in instrumented SNWs.

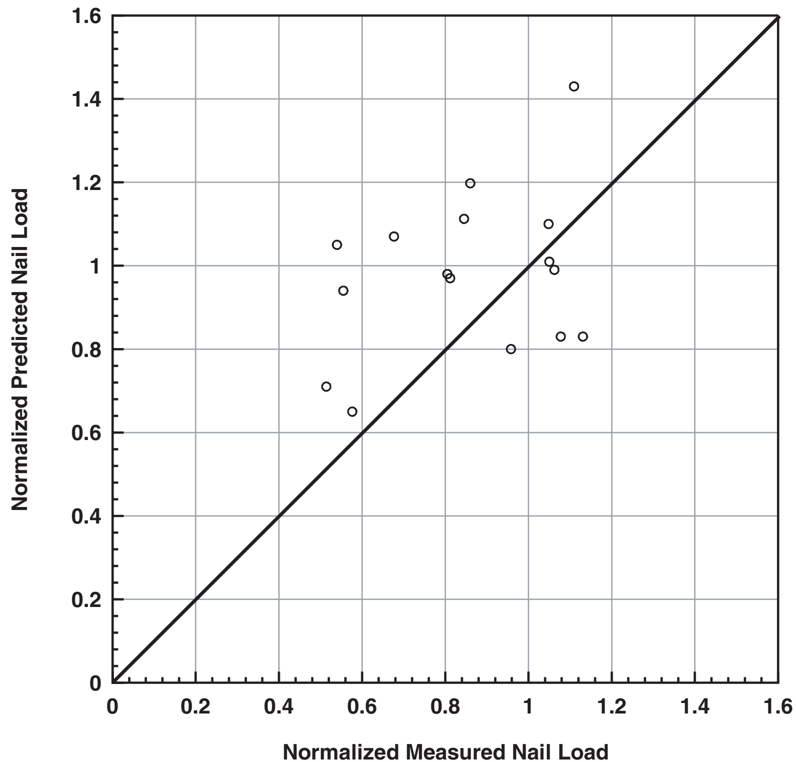


Figure 3-31. Measured and predicted maximum nail load.

Step 8: If computed and target β values differ, modify resistance factor and repeat until solution converges.

Each of these steps is described in the following sections.

Step 1: Establish a Limit State Function

The limit state function, M, for nail pullout is defined as:

$$M = \phi_{PO}\, R_{PO} - \gamma_Q\, T_{max} \tag{3-88}$$

where

ϕ_{PO} = the resistance factor for pullout,
R_{PO} = a random variable representing the nominal pullout resistance,
γ_Q = a load factor, and
T_{max} = a random variable representing the load in a nail.

At the limit state (i.e., $M = 0$), resistance can be expressed as:

$$R_{PO} = \frac{\gamma_Q}{\phi_{PO}} T_{max} \tag{3-89}$$

The limit state function can be rewritten as:

$$M = \frac{\gamma_Q}{\phi_{PO}} T_{max} - T_{max} \tag{3-90}$$

The two terms in Equation 3-90 that contain T_{max} must be interpreted as two independent random variables, each with different statistical parameters and each multiplied by the term T_{max}, which is not a random variable but a scaling factor. Both random variables are generated separately in the simulation.

As soil nail loads can be represented using a lognormal distribution, random values for the load in the nail is generated as:

$$T_{max\ i} = \exp\left(\mu_{ln} + \sigma_{ln} z_i\right) \tag{3-91}$$

where

$T_{max\ i}$ = a randomly generated value of the variable T_{max};
μ_{ln} = lognormal mean of the random variable that includes T_{max};
σ_{ln} = lognormal standard deviation of the random variable that includes T_{max};
z_i = inverse normal function, or $\Phi^{-1}(u_{ia})$; and
u_{ia} = a random number between 0 and 1 representing a probability of occurrence.

The lognormal mean and standard deviation of the random variable that includes T_{max} is obtained from normal parameters as:

$$\mu_{ln} = \ln\left(T_{max\ mean}\right) - \frac{\sigma_{ln}^2}{2} \tag{3-92}$$

Table 3-12. Characteristics of monitored soil nail walls.

Feature	Feature Case										
	Oregon	Swift-Delta Station 1	Swift-Delta Station 2	Polyclinic	Peasmarsh, U.K.	Guernsey, U.K.	IH-30, Rockwall, Section A	IH-30, Rockwall, Section B	San Bernardino	Cumberland Gap, 1988	I-78, Allentown
Height (m)	TBC	5.3	5.6	16.8	11	20	5.2	4.3	7.6	7.9	12.2
Face slope (deg)	TBC	0	0	0	20	30	0	0	6	0	3 m bench
Back slope (deg)	TBC	55 kN/m surcharge	27	0	0	0	0	75 kN/m surcharge	5	33	33
Type of facing	TBC	shotcrete	shotcrete	shotcrete	geogrid	geogrid	shotcrete	shotcrete	shotcrete	shotcrete	concrete panels
Nail length (m)	TBC	6.4	5.2	10.7	6–7	10	6.1	6.1	6.7	13.4	6.1–9.2
Nail inclination (deg)	TBC	15	15	15	20	20	5	5	12	15	10
Nail diameter (mm)	TBC	NA	NA	NA	NA	NA	152	152	203	114	89
Steel diameter (mm)	TBC	29	29	36	25	25	19	19	25	29	25–32
Spacing, H x V (m)	TBC	1.4 x 1	1.4 x 1	1.8 x 1.8	1.5 x 1.5	1.5 x 1.25	0.75 x 0.75	0.75 x .75	1.5 x 1.5	1.5 x 1.2	1.5 x 1.5

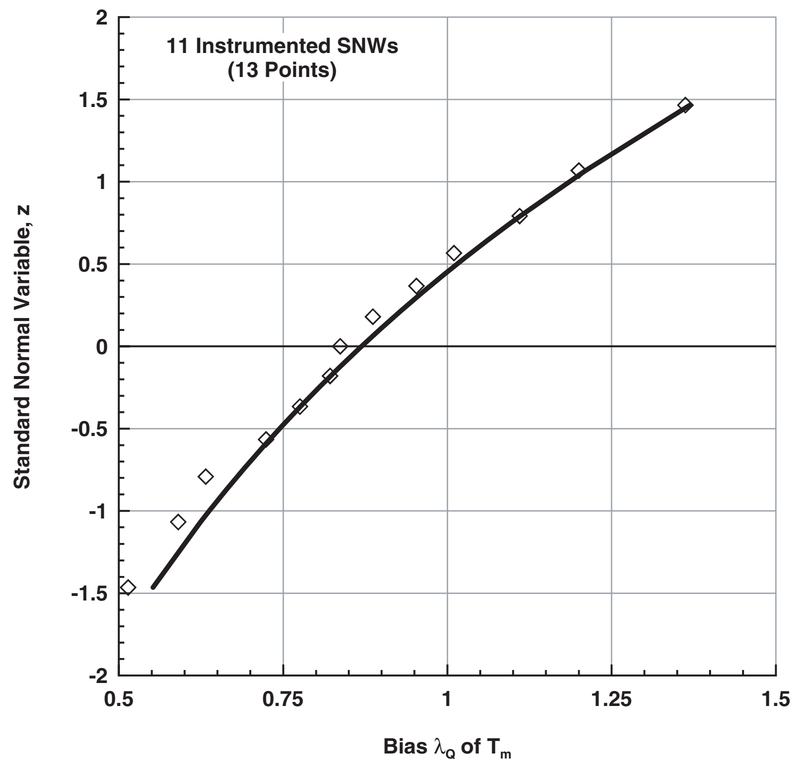


Figure 3-32. Bias λ_Q for maximum load in soil nails.

Table 3-13. Summary of normalized measured and predicted maximum nail load.

No.	Case	Normalized Measured Load, T_m	Normalized Predicted Load, T_p	Bias of Load
1	Cumberland Gap, 1988	0.54	1.05	0.51
2	Polyclinic	0.56	0.94	0.59
3	I-78, Allentown	0.68	1.07	0.63
4	Guernsey, U.K.	0.51	0.71	0.72
5	Swift-Delta Station 2	1.11	1.43	0.78
6	Oregon – 3-A	0.81	0.98	0.82
7	Swift-Delta Station 1	0.81	0.97	0.84
8	Peasmarsh, U.K.	0.58	0.65	0.89
9	Oregon – 2-B	1.05	1.10	0.95
10	IH-30, Rockwall, Section B	1.06	0.99	1.01
11	Oregon – 1-A	0.96	0.80	1.11
12	San Bernardino (R)	1.08	0.83	1.20
13	San Bernardino (L)	1.13	0.83	1.36

Table 3-14. Statistics of bias for maximum nail loads.

Load Parameters						
Number of Points in Database	Distribution Type	Mean of Bias	Standard Deviation	Coefficient of Variation	Log Mean of Bias	Log Standard Deviation
N		λ_Q	σ_Q	COV_Q	μ_{ln}	σ_{ln}
13	Lognormal	0.912	0.290	0.32	-0.140	0.31

and

$$\sigma_{ln} = \sqrt{\ln(COV_Q + 1)} \tag{3-93}$$

where

$T_{max\,mean}$ = mean of the random variable that includes T_{max}, and

COV_Q = coefficient of variation of the bias of the random variable that includes T_{max}.

If the pullout resistance is modeled as a lognormal variable, the right-hand side of Equation 3-89 is randomly generated as:

$$R_{PO\,i} = \frac{\gamma_Q}{\phi_{PO}} \exp(\mu_{ln\,R} + \sigma_{ln\,R} z_i) \tag{3-94}$$

where

$R_{PO\,i}$ = a randomly generated value of the variable R_{PO};

γ_Q = load factor;

ϕ_{PO} = resistance factor for pullout;

$\mu_{ln\,R}$ = lognormal mean of R_{PO};

$\sigma_{ln\,R}$ = lognormal standard deviation of R_{PO};

z_i = an inverse normal function, or $\Phi^{-1}(u_{ib})$; and

u_{ib} = a random number between 0 and 1 representing a probability of occurrence (this number is independent from the number u_{ia} defined previously).

The lognormal mean and standard deviation of R_{PO} is obtained from normal parameters for R_{PO} as:

$$\mu_{ln\,R} = \ln(R_{PO\,mean}) - \frac{\sigma^2_{ln\,R}}{2} \tag{3-95}$$

$$\sigma_{ln\,R} = \sqrt{\ln(COV_R + 1)} \tag{3-96}$$

where

$R_{PO\,mean}$ = mean of R_{PO}; and

COV_R = coefficient of variation of the bias of R_{PO}.

In addition,

$$R_{PO\,mean} = \lambda_R R_{max}$$

where

λ_R = the normal mean of the bias of R_{PO}, and

R_{max} = a non-random scaling factor, similar to the case of loads.

Step 2: Develop PDFs and Statistical Parameters for R and Q

Statistical parameters for soil nail pullout resistance were developed from the database presented in Appendix C. These values were summarized in Table 3-11 for various soil conditions. Statistical parameters for maximum loads on a soil nail were derived previously in this chapter based on the analyses of various instrumented walls. These values were summarized in Table 3-14.

Step 3: Select a Target Reliability Index for SNW Design

As discussed earlier in this chapter, the selection of the target reliability index, β_T, is a key factor in a reliability-based design. Because soil nails are installed relatively close to each other (i.e., vertical and horizontal spacing is typically 5 ft) and the resulting reinforcement density per unit area is relatively high, SNWs are considered structures with relatively high structural redundancy. To be consistent with the current practice of selection of a target reliability index for elements with high structural redundancy, $\beta_T = 2.33$ (and $P_f = 1\%$) was selected for this study.

Step 4: Establish Load Factors

The expression used to estimate the load factor is as follows:

$$\gamma_Q = \lambda_Q (1 + n_\sigma\, COV_Q) \tag{3-97}$$

where

γ_Q = load factor,

λ_Q = mean of the bias for the load,

COV_Q = coefficient of variation of the measured to predicted load ratio, and

n_σ = number of standard deviations from the mean.

Using the statistical parameters and $n_\sigma = 2$, the load factor can be estimated as:

$$\gamma_Q = 0.91(1 + 2 \times 0.32) = 1.49 \approx 1.5$$

The value $\gamma_Q = 1.5$ best represents the statistics used in AASHTO (2007). However, other load factors can be considered in the simulation and different resistance factors can be calculated. In this simulation (see Step 6), the following load factor values were considered to account for various loading scenarios of SNWs, $\gamma_Q = 1.0$, 1.35, 1.5, 1.6, and 1.75. Resistance factors for pullout were calculated for this series of load factors.

Step 5: Best-Fit Cumulative Density Functions to Data Points

CDFs for loads and resistances were generated via Monte Carlo simulations using the statistics for load and resistances. After the fitting curves were developed for each set of data points, they were plotted side by side, as shown in Figures 3-33 through 3-36. The abscissas on these figures are values of the random variables T_{max} and R_{PO}. The ordinates are values of

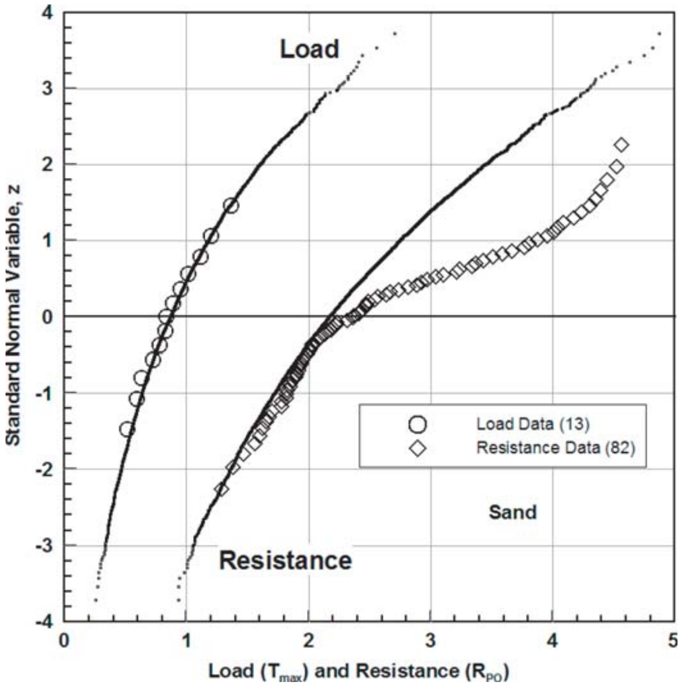


Figure 3-33. Monte Carlo curve fitting of load and resistance—sand.

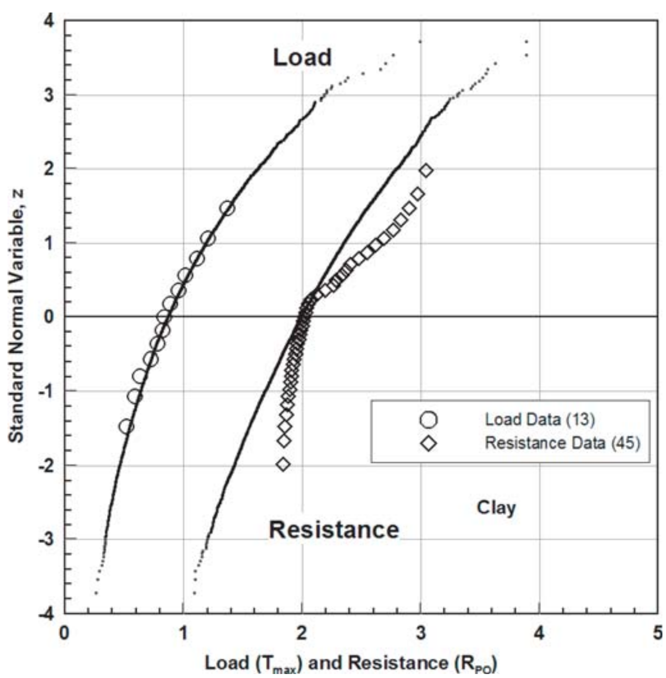


Figure 3-34. Monte Carlo curve fitting of load and resistance—clay.

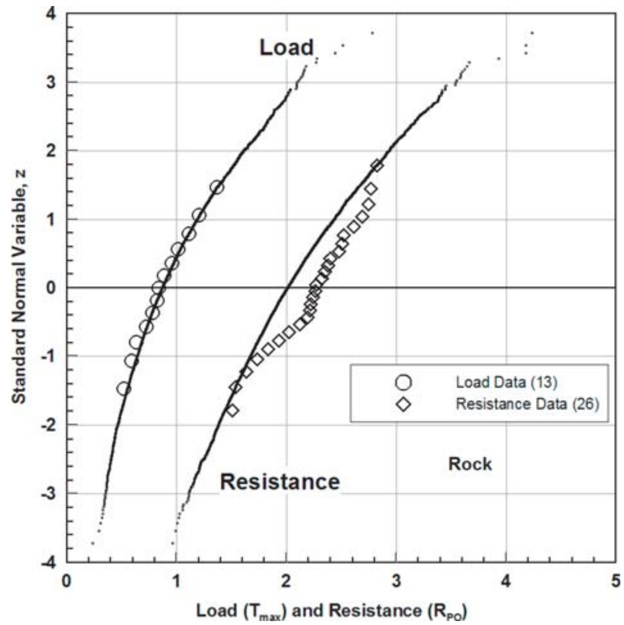


Figure 3-35. Monte Carlo curve fitting of load and resistance—rock.

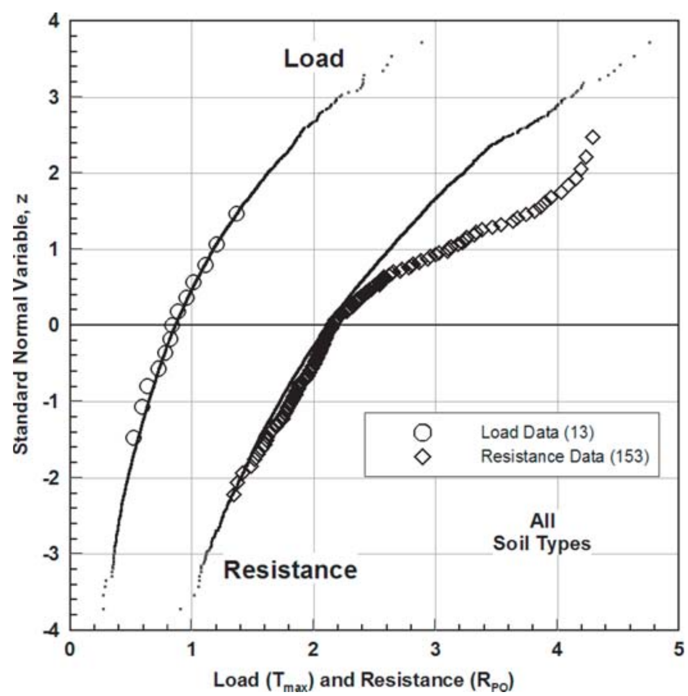


Figure 3-36. Monte Carlo curve fitting of load and resistance—all soil types.

the standard normal variable z. CDFs are shown as essentially continuous functions on Figures 3-33 through 3-36 (small markers can be observed at the tails of the CDFs). Data points for load (13 points) and resistance (varying number for each soil type) are plotted as circles and diamonds, respectively, in Figures 3-33 through 3-36. On the left of these figures, the generated CDF for loads was compared to the upper tail of the load data distribution and was verified to be equal or greater than all data points. Conversely, on the right of these figures, the generated CDF for pullout resistance was compared to the lower tail of the resistance data distribution.

The distribution for pullout resistance was best-fitted to match the lower tail of the resistance PDF. The curve-fitting accuracy is unaffected by the upper tail of the resistance CDF because it is the lower tail of the resistance distribution that controls the calculated reliability factor (Allen et al., 2005).

Step 6: Conduct Monte Carlo Simulation

The Monte Carlo simulation was conducted to artificially generate additional values of load and pullout resistance than the ones available from data points and to estimate the probability of failure accurately. For each soil type, random numbers were generated independently for the random variables containing T_{max} and R_{PO}. Independent values of the random numbers u_{ia} and u_{ib} were generated in 10,000 trials to calculate new values for $T_{max\,i}$ and $R_{PO\,i}$ and to develop complete distributions of these two random variables.

Pullout resistance factors were calculated for the range of γ_Q listed in Step 4. Figures 3-33 through 3-36 present the curve-fitting analysis using Monte Carlo for different soils and for $\gamma_Q = 1.75$. Figures 3-37 through 3-40 present results of the simulation of the limit function M for different materials and $\gamma_Q = 1.75$. In all cases, $\beta_T = 2.33$ and $P_f = 1\%$.

Steps 7 and 8: Compare Computed and Target Reliability Indices and Iterate, If Necessary

After a few iterations, results converged and the simulation was stopped when the difference between the computed and target reliability indices was smaller than 0.5%.

3.7.3 Results

The results of the calibration using Monte Carlo simulations are included in Table 3-15. Various pullout resistance factors were obtained for the various soil/rock types considered and for the range $\lambda_Q = 1.0, 1.35, 1.5, 1.6$, and 1.75 to show the dependency of these factors. This range represents values that can be commonly used for retaining structures that are part of bridge substructures. The case of $\gamma_Q = 1.0$, applicable for overall stability as a service limit state (per current AASHTO LRFD practice), is also included.

For the case of $\lambda_Q = 1.5$ (case based on load statistics), the range of ϕ_{PO} varies from 0.70 to 0.77. This range is comparable to the preliminary range varying from 0.63 to 0.70 obtained in Section 3.5.5 for $FS_{PO} = 2.0$ and $Q_{DC}/Q_{LL} \geq 2.5$.

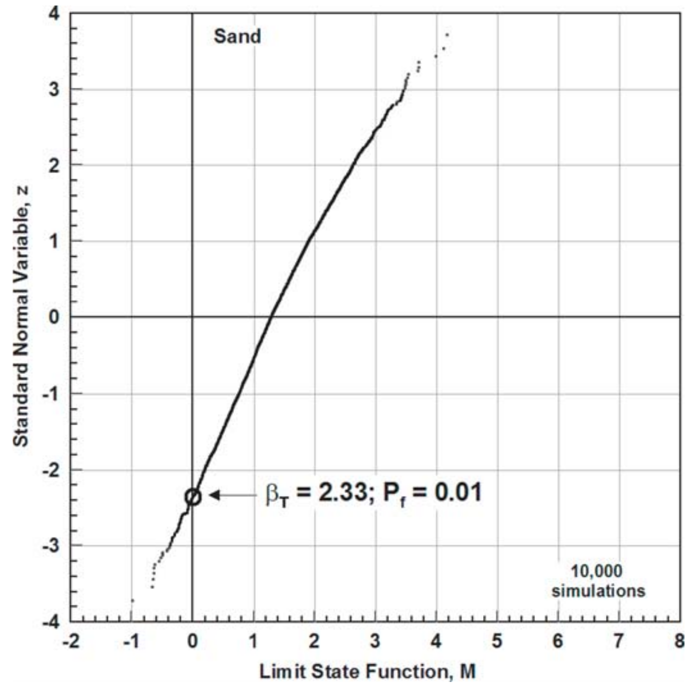


Figure 3-37. Monte Carlo simulation—sand.

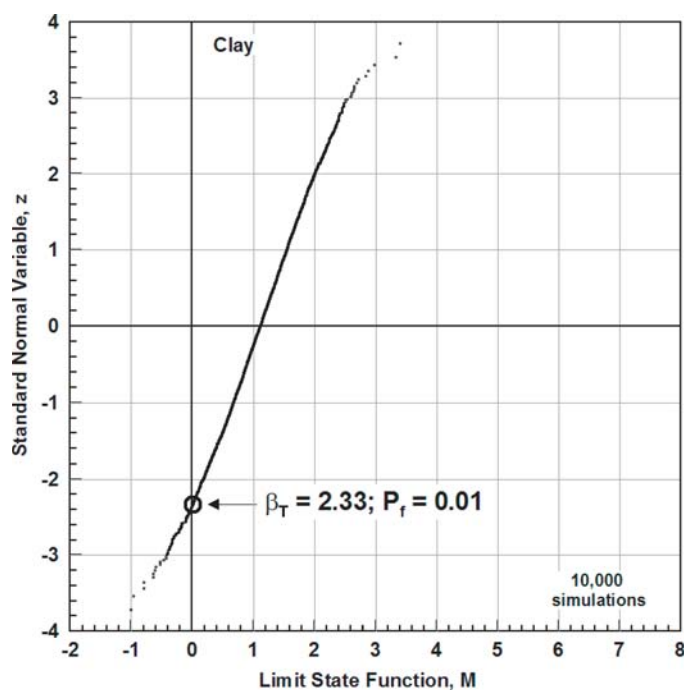


Figure 3-38. Monte Carlo simulation—clay.

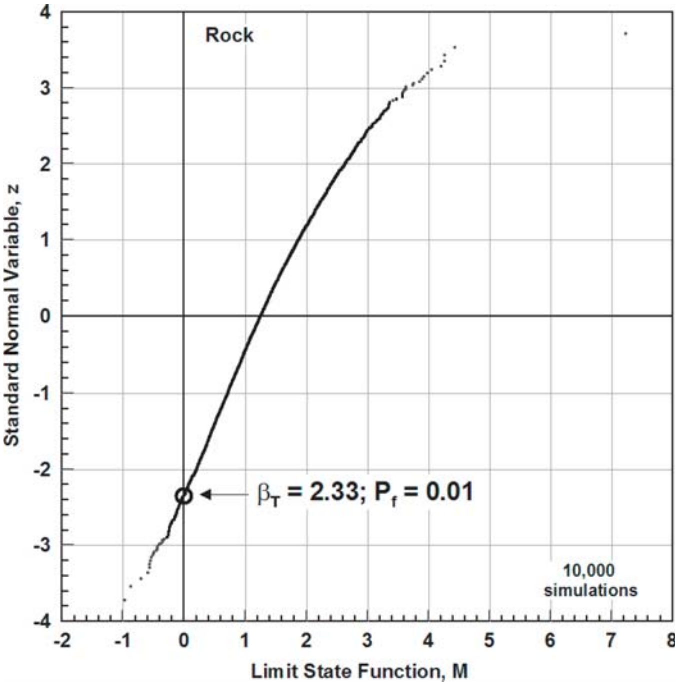


Figure 3-39. Monte Carlo simulation—rock.

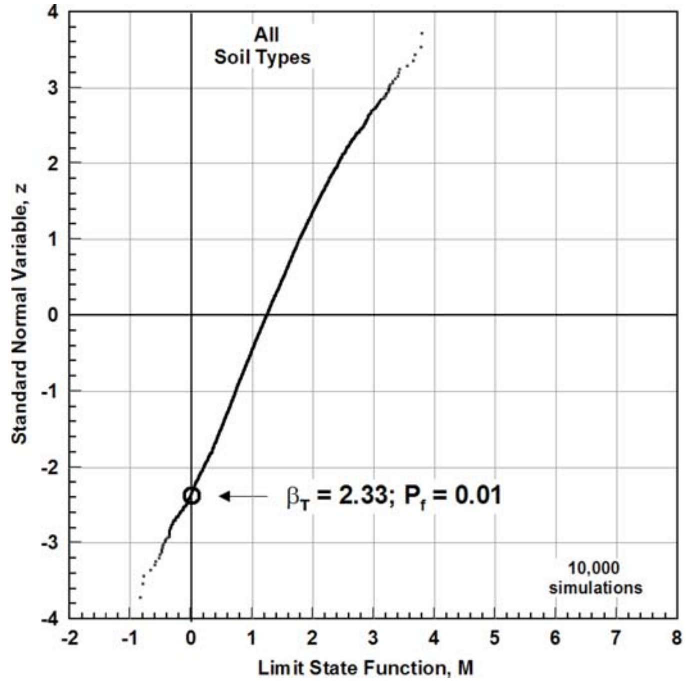


Figure 3-40. Monte Carlo simulation—all soil types.

For the case $\lambda_Q = 1.0$, the pullout resistance factors ϕ_{PO} for various soils vary between 0.47 and 0.51. This range encompasses the value $\phi_{PO} = 0.5$, which would be obtained based on the ASD-based method as the inverse of a global safety factor $FS_{PO} = 2$ (see Chapter 4 and Lazarte et al., 2003).

Because of the values of the calibrated resistance factors for pullout, it is expected that a LRFD-based SNW design that uses this range of resistance factors would not produce significant differences in results (i.e., in terms of soil nails, nail bar diameter, etc.) as compared to designs based on the ASD method when a safety factor $FS_{PO} = 2$ is used. Appendix D provides detailed comparative designs of SNWs under various conditions to quantify these differences. As will be seen, these differences are small.

The calibrated results also indicate that the reliability in design is approximately the same among all selected materials, with soil nails in weathered rock having a slightly lower resistance factor.

Overall, with reference to pullout resistances, the design of SNWs will not be affected significantly by use of the LRFD method in lieu of the ASD method. The same applies for other resistance modes including nail in tension, and facing resistances because the factors associated with these resistances were selected from the ASD practice.

Table 3-15. Summary of calibration of resistance factors for soil nail pullout for various load factors.

Material	Number of Points in Database	Distribution Type	Mean of Bias	Standard Deviation	Coefficient of Variation	Log Mean of Bias	Log Standard Deviation	λ_Q				
								1.75	1.60	1.50	1.35	1.00
	N		λ_R	σ_R	COV_R	μ_{ln}	σ_{ln}	$\phi_R = \phi_{PO}$				
Sand/Sandy Gravel	82	Lognormal	1.05	0.25	0.24	0.02	0.24	0.82	0.75	0.70	0.63	0.47
Clay/Fine-Grained	41	Lognormal	1.03	0.05	0.05	0.03	0.05	0.90	0.82	0.77	0.69	0.51
Rock	26	Lognormal	0.92	0.18	0.19	–0.10	0.19	0.79	0.72	0.68	0.61	0.45
All	149	Lognormal	1.05	0.22	0.21	0.03	0.21	0.85	0.78	0.73	0.66	0.49

CHAPTER 4

Conclusions and Suggested Research

4.1 Conclusions

This study was conducted in the following main steps: (i) review of guidance procedures and specifications for the design and construction of SNWs; (ii) compilation of soil nail load-testing data for developing pullout resistance information, and load data from instrumented walls for developing load statistics; (iii) development of databases for pullout resistance and loads in SNWs; (iv) development of resistance factors based on reliability methods and on the aforementioned databases; and (v) comparison of designs using the LRFD and ASD methods.

The review of existing procedures for the design and construction of SNWs was focused on U.S. practice, although the review also included international references. LRFD factors developed for comparable types of retaining structures were also reviewed, including interim editions and the latest edition of the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2007). Because the design of SNWs as conducted in the United States is based on limit-equilibrium methods (i.e., related to limit states of overall stability), the load combination selected to design SNWs was the service limit state, consistent with the approach currently adopted in AASHTO (2007) for the limit states of overall stability.

A significant amount of soil nail load-test data was collected from several sources. After several results were eliminated due to lack of information or inconsistencies, a database of nail pullout resistance was compiled to support the calibration of pullout resistance factors. The volume of pullout resistance data was sufficient to create data subsets for three subsurface conditions, namely predominantly sandy soils, clayey soils, and weathered rock. More data points were available from projects of SNWs constructed in sandy soils than in clayey soils and weathered rock. To reduce potential scatter in the database due to variable levels of workmanship and equipment among different contractors, data points were selected, as much as possible, from the same contractor using the same equipment at the same project. The information available that accompanied the soil nail load-test data was in general insufficient to study other aspects (e.g., construction methods) that may affect the variability of soil nail pullout resistance. In addition, a database of soil nail loads based on instrumented SNWs was created.

Resistance factors for elements that are common to other retaining systems (e.g., factor for the nominal tensile resistance of steel bars) were adopted from the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2007) for consistency. Current values were found to be acceptable for the design of SNWS. These resistance factors are presented in Table 4-1.

The calibration of the resistance factor for soil nail pullout was conducted using reliability methods and the resistance and load databases mentioned above. The calibration was conducted using the procedures suggested for developing load and resistance factors in general geotechnical and structural design (Allen et al., 2005). In this approach, several steps were followed, from selecting a target reliability index that is consistent with the level of structural redundancy of SNWs, to a Monte Carlo simulation to estimate pullout resistance factors.

For each soil/rock material considered in the pullout resistance database, statistical parameters were obtained for the bias of pullout resistance and loads in SNWs. In addition, the database of soil nail loads allowed an estimation of the statistical parameters for the bias of loads. Both load and resistance were considered to be random variables having lognormal distributions.

The target reliability index was selected based on a comparison of SNWs with other substructures that have a comparable level of structural redundancy and for which target reliability indices have been proposed. The reliability selected for SNWs was 2.33, which is consistent with the value used for the calibration of resistance parameters for pullout in MSE walls (Allen et al., 2005). SNWs and MSE walls have comparable reinforcement densities (i.e., number of reinforcement elements per unit of wall area), comparable reinforcement

Table 4-1. Summary of resistance factors for SNWs.

Limit State	Resistance	Condition		Resistance Factor	Value
Soil Failure	Sliding	All		ϕ_τ	0.90
	Basal Heave	All		ϕ_b	0.70
Overall Stability	NA	Slope does not support a structure		ϕ_s	0.75 (1)
		Slope supports a structure		ϕ_s	0.65 (2) (3)
		Seismic		ϕ_s	0.9 (4)
Structural	Nail in Tension	Static	Mild steel bars—Grades 60 and 75 (ASTM A 615)	ϕ_T	0.56 (5)
			High-resistance—Grade 150 (ASTM A 722)	ϕ_T	0.50 (5)
		Seismic	Mild steel bars—Grades 60 and 75 (ASTM A 615)	ϕ_T	0.74 (5)
			High-resistance—Grade 150 (ASTM A 722)	ϕ_T	0.67 (5)
	Facing Flexure	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FF}	0.67 (5)
	Facing Punching-Shear	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FP}	0.67 (5)
	Facing Headed-Stud Tensile	A307 Steel Bolt (ASTM A 307)		ϕ_{FH}	0.50 (5)
		A325 Steel Bolt (ASTM A 325)		ϕ_{FH}	0.59 (5)
	Pullout	Soil/Rock Type	Sand	ϕ_{PO}	0.47 (6)
			Clay	ϕ_{PO}	0.51 (6)
			Weathered Rock	ϕ_{PO}	0.45 (6)
			All	ϕ_{PO}	0.49 (6)

Notes: (1) AASHTO (2007) also considers this value when geotechnical parameters are well defined.
(2) AASHTO (2007) also considers this value when geotechnical parameters are based on limited information.
(3) For temporary SNWs, use $\phi_s = 0.75$.
(4) Per AASHTO (2007) but subject to modification after new Standard is in place. A value $\phi_s = 1.00$ may be acceptable, as long as permanent deformations are calculated (see Anderson et al., 2008) and are found not to be excessive. Currently, there is no differentiation for temporary or non-critical structures under seismic loading; therefore, use $\phi_s = 1.00$.
(5) Calibrated from safety factors.
(6) From reliability-based calibration. Values shown correspond to a load factor $\gamma = 1.00$.

length/wall height ratios, and thereby comparable and relatively high structural redundancies.

The calibration proceeded using an iterative scheme in a Monte Carlo simulation. Based on the statistical parameters for load and resistances selected earlier, up to 10,000 random simulations were conducted for each soil type in order to generate a complete distribution of load and resistance.

Although the load factor should be selected as 1.0 for service limit states (per current AASHTO LRFD practice, as mentioned previously), a series of pullout resistance factors was obtained for a range of load factors other than 1.0 to show the effect of load factors on the pullout resistance factor for each of the soil/rock types considered. The load factors selected were $\lambda_Q = 1.0, 1.35, 1.5, 1.6,$ and 1.75. This range represents the values that can be commonly used for retaining structures that are part of bridge substructures. The calibrated pullout resistance factors based on this range of load factors is presented in Table 4-2.

Calibration resistance factors were subsequently used to perform comparative designs for SNWs for a wide variety of conditions. The objective of the comparative designs was to evaluate differences of the required soil nail length, as obtained using computer programs with the ASD method or the LRFD method. Over 30 design cases were considered to assess the effect of several key factors in the design. These factors included wall height, soil friction angle, bond resistance, and surcharge loads. Results of the comparative designs indicate that the required soil nail length calculated using the LRFD method and the proposed resistance factors are comparable with those obtained with the ASD method. For all cases considered, the length difference is, on average, approximately 4% larger in the LRFD method. None of the factors appear to have a

Table 4-2. Summary of pullout resistance factors for various load factors.

Material	Load Factor λ_Q	Pullout Resistance Factor ϕ_{PO}
Sand	1.75	0.82
	1.6	0.75
	1.5	0.70
	1.35	0.63
	1.0	0.47
Clay	1.75	0.90
	1.6	0.82
	1.5	0.77
	1.35	0.69
	1.0	0.51
Weathered Rock	1.75	0.79
	1.6	0.72
	1.5	0.68
	1.35	0.61
	1.0	0.45
All	1.75	0.85
	1.6	0.78
	1.5	0.73
	1.35	0.66
	1.0	0.49

Note: Reliability Index: $\beta = 2.33$

greater influence than others, possibly with the exception of surcharge loads. The largest difference obtained in the comparative analysis was approximately 8%.

Discussions on the use of the computer programs GOLDNAIL and SNAILZ for LRFD-based design of SNWs are also provided in this document.

The comparative designs mentioned above have shown that the design of SNWs using the LRFD method would result in quantities comparable to, although slightly higher (i.e., approximately 4% increase of soil nail length on average) than, those obtained with the ASD method. Essentially there are no changes in the requirement of bar diameters, bar lengths, and facing dimensions and quantities. The use of the LRFD method allows for designing SNWs with a reliability level that is compatible with reliability levels of other elements of a bridge superstructure or other comparable retaining systems.

Proposed specifications for the design and construction of SNWs were also developed and are provided as appendices to this report. The proposed specifications follow the format of AASHTO (2007). The proposed design specifications include several sections:

- Sections 11.12.1 through 11.12.2 provide general descriptions, loading conditions, and controlling factors to be used in the design of SNWs.
- Section 11.12.3 provides guidance and commentary that aid in conducting evaluations of service limit states for both deformations and overall stability.
- Section 11.12.4 addresses safety against soil failure and provides guidance and commentary for conducting evaluations for the limit states of basal heave and sliding stability.
- Sections 11.12.5 and 11.12.6 provide guidance and commentary for structural limit states—including soil nail pullout and soil nail in tension—and all of the limit states for facings.
- Finally, Sections 11.12.7 through 11.12.8 provide guidance and commentary for conducting drainage evaluations and providing corrosion-protection for SNWs.

4.2 Suggested Research

The results of this research project have provided a basis for designing SNWs using the LRFD method for various soil conditions. However, some aspects related to SNW construction and design were not addressed in this project but can be expanded through additional research. Some of these aspects and areas of additional research are discussed below:

- Addressing limit-equilibrium problems as a service limit in current AASHTO LRFD practice is apparently an unresolved issue and will remain unresolved until additional information or studies are available. Although this topic is of general applicability for various bridge substructures, it will affect the design of SNWs if changes are made to the current practice.
- The current database of soil nail load tests can be expanded, relying on tests that exhibit clearly a limit state for pullout. This effort should help augment the current data sets not only for the three material types considered but also for other soil types and conditions (e.g., gravelly soils, residual soil, loess, and typical "regional" soils).
- The current database of pullout resistance based on soil nail load tests can be expanded and subdivided for certain construction procedures that directly affect pullout capacity, including drilling techniques, practice for cleaning the hole, grout characteristics, etc.
- The database for loads measured in SNWs can be expanded for other conditions, particularly for larger surcharge loads.
- Correlations between soil/rock properties, common field investigation techniques [i.e., SPT as mentioned in this report but also other popular field techniques including cone penetration testing (CPT)], and pullout resistance can be developed as additional predictive tools.
- The effect of the number and characteristics of soil nail load testing on the reliability of the design can be explored. It is reasonable to expect that conducting more verification tests, or increasing the test load in verification tests beyond

200% of the assumed design load, would help establish more precisely the ultimate resistance, would enhance the reliability of the pullout resistance, and possibly result in more economical designs. However, it is recognized that this approach may penalize competent contractors who have considerable experience and have the expertise to guarantee the specified bond strength with little testing.

- Effects of the spatial variability of subsurface conditions on pullout resistance, which are not commonly taken into account, can be explored in more detail when enough field exploration data is available (i.e., typically much more than what is conventionally produced). While this effect may not be significant for SNWs constructed over small areas, this effect may be significant in the use of SNWs along roadways or as part of the abutments for relatively long bridges. However, it is recognized that a reliable quantification of spatial variability can only be achieved if sufficient field exploration data is available. For most project conditions, it is unlikely that enough geotechnical data would be available to quantify spatial variability.
- New soil-nailing techniques and new soil nail materials can be considered for possible application for transportation projects. These innovations include self-boring nails, Glass-Fiber Reinforced Polymer (GFRP) bars, and different head nail connections.
- Aspects related to the seismic design of substructures that have been recently proposed in interim editions of the AASHTO *LRFD Bridge Design Specifications* may require evaluation in order to adapt those changes to the design of SNWs.
- The current criterion for estimating lateral deformation of SNWs is limited. The quantification of the effects of soil nail layout on the distribution and magnitude of deformations is also suggested as a follow-up research topic. To this end, numerical studies using the finite-element method or comparable techniques are suggested to obtain estimates of constructed and monitored walls. Comparisons of the numerically estimated and measured wall deformations will help calibrate the numerical methods, which can eventually be used to predict the deformation of future walls.

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Abbreviations

AASHTO	Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AFOSM	Advanced First-Order Second-Moment
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
API	American Petroleum Institute
ASD	Allowable Stress Design
CDF	Cumulative Density Function
CIP	Cast-in-Place
CIRIA	Construction Industry Research and Information Association
COV	Coefficients of Variation
CPT	Cone Penetration Testing
DL	Design Load
DOT	Department of Transportation
FHWA	Federal Highway Administration
FOSM	First-Order Second-Moment
FS	Factors of Safety
ft	feet, foot
GEC	Geotechnical Engineering Circular
GFRP	Glass-Fiber Reinforced Polymer
GUTS	Guaranteed Ultimate Tensile Strength
HDPE	High-Density Polyethylene
in.	inch(es)
kPa	kilopascal
LRFD	Load and Resistance Factor Design
MOM	Mononobe-Okabe Method
MPa	megapascal
MSE	Mechanically Stabilized Earth
NCHRP	National Cooperative Highway Research Program
PDF	Probability Density Function
PMT	Pressuremeter Test
psi	pounds per square inch
PTI	Post-Tensioning Institute
PVC	Polyvinyl Chloride
SI	International System
SNW	Soil Nail Wall

SPT	Standard Penetration Test
tsf	tons per square foot
U.S.	United States
WWM	Welded Wire Mesh

Symbols

A_E = Effective cross-sectional area of threaded anchors (or bolts)

A_H = Cross-sectional area of the connector head

a_{hm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at midspan between soil nails

a_{hn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at soil nail heads

A_{HH} = Total cross-sectional area of additional reinforcement (i.e., waler bars) in wall facing, in the horizontal direction and around soil nail heads

A_S = Cross-sectional area of headed-stud shaft

A_t = Nail bar cross-sectional area

A_{VH} = Total cross-sectional area of additional reinforcement (rebar) in wall facing, in the vertical direction and around soil nail heads

a_{vm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction in the mid-span between soil nail heads

a_{vn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction over the soil nail heads

C = Coefficient used for the estimation of the soil nail wall displacement

C_F = Factor that considers non-uniform soil pressures behind a soil nail wall facing and is used in the estimation of nominal resistances at the soil nail head

C_P = Factor that accounts for soil contribution to support and is used in the estimation of nominal resistances at the soil nail head

D'_c = Effective, equivalent diameter of the potential slip conical failure in the facing around soil nail heads

D_{DEF} = Horizontal distance behind soil nail wall where ground deformation can be significant

D_{DH} = Average diameter of soil nail drill-hole

D_E = Effective diameter of the core of a threaded anchor

D_H = Diameter of the head of a soil nail head connector (i.e., headed-stud)

D_S = Diameter of the shaft of a soil nail head connector (i.e., headed-stud)

f'_c = Concrete compressive nominal resistance

f_y = Yield tensile nominal resistance of soil nail bar

$f_{y\text{-}f}$ = Yield tensile nominal resistance of reinforcement in facing

$f_{y\text{-}hs}$ = Yield tensile nominal resistance of headed-stud in facing

h = Thickness of facing

H = Wall height

h_c = Effective depth of potential conical slip surface forming in facing around soil nail head
h_f = Thickness of permanent facing
h_t = Thickness of temporary facing
K_A = Active earth pressure coefficient of soils behind soil nail wall
L = Soil nail length
L_{BP} = Bearing plate side dimension
L_P = Pullout length extending behind slip surface
L_S = Length of headed-stud
m_{hm} = Horizontal flexural resistance (moment per unit length) mid-span between soil nails
m_{hn} = Horizontal flexural resistance (moment per unit length) at soil nail head
m_{vm} = Vertical flexural resistance (moment per unit length) mid-span between soil nails
m_{vn} = Vertical flexural resistance (moment per unit length) at soil nail head
N_H = Number of headed-studs in soil nail head connection
nt = Number of threads per unit length in threaded anchor (i.e., bolt)
q_U = Nominal bond resistance of soil nails
R_{FF} = Nominal resistance for flexure in facing
R_{FH} = Nominal resistance for tension of headed-studs located in facing
R_{FP} = Nominal resistance for punching-shear in facing
R_{PO} = Nominal pullout resistance of soil nails
r_{PO} = Nominal pullout resistance per unit length of soil nails
R_T = Nominal resistance of a soil nail bar in tension
S_H = Horizontal spacing of soil nails
S_{HS} = Spacing of headed-studs
S_V = Vertical spacing of soil nails
t_H = Head thickness of headed-studs
T_{max} = Maximum load in a soil nail
T_o = Maximum load in the head of a soil nail
t_P = Thickness of bearing plate
V_F = Punching-shear force acting through facing, around soil nail head
α = Angle of batter of soil nail wall
β = Backslope angle
δ_h = Horizontal displacement at the top of a soil nail wall
δ_v = Vertical displacement at the top of a soil nail wall
ϕ_{FF} = Resistance factor for flexure in facing
ϕ_{FH} = Resistance factor for facing headed-stud in tension
ϕ_{FP} = Resistance factor for punching-shear in facing
ϕ_{PO} = Resistance factor for nail pullout
ϕ_T = Resistance factor for nail bar in tension
γ_s = Unit weight of soil
ρ_{ij} = Reinforcement ratio in "i" direction (vertical or horizontal) and location "j" (at nail head "n," or midspan "m" between soil nails)
ρ_{max} = Maximum reinforcement ratio in facing
ρ_{min} = Minimum reinforcement ratio in facing

APPENDIX A

Proposed LRFD Design Specifications for Soil Nail Walls

Revisions to SECTION 11
AASHTO LRFD Bridge Design Specifications

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11.1 SCOPE

This section provides requirements for design of abutments and walls. Conventional retaining walls, non-gravity cantilevered walls, anchored walls, mechanically stabilized earth (MSE) walls, prefabricated modular walls, and soil nail walls are considered.

C11.1

11.2 DEFINITIONS

Soil Nail Walls – A soil-retaining system that derives lateral resistance from a regular pattern of soil nails. Soil nails are sub-horizontal closely spaced steel bars (spacing in each direction of approximately 5 FT or with a tributary area of generally no more than 36 sq FT), that are most commonly installed in a predrilled hole and subsequently encased in grout. Other installation methods, including self-drilling nails, exist. Soil nails are most commonly installed as passive elements whereby no post-tensioning is applied. Soil nails are connected with a facing, which is a structurally continuous reinforced shotcrete or concrete layer covering the soil nails.

11.3 NOTATION

11.3.1 General

A_E = Effective cross-sectional area of threaded anchors (or bolts) (C11.12.6)

A_H = Cross-sectional area of the connector head (IN^2) (11.12.6)

a_{hm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at midspan between soil nails (IN^2/FT) (11.12.6)

a_{hn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at soil nail heads (IN^2/FT) (11.12.6)

A_{HH} = Total cross-sectional area of additional reinforcement (i.e., waler bars) in wall facing, in the horizontal direction and around soil nail heads (IN^2) (C11.12.6)

A_S = Cross-sectional area of headed-stud shaft (IN2) (11.12.6)

A_t = Nail bar cross-sectional area (IN^2) (11.12.5)

A_{VH} = Total cross-sectional area of additional reinforcement (rebar) in wall facing, in the vertical direction and around soil nail heads (IN^2) (C11.12.6)

a_{vm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction, at soil nail heads (IN^2/FT) (11.12.6)

a_{vn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction, at soil nail heads (IN^2/FT) (11.12.6)

C = Coefficient used for the estimation of the soil nail wall displacement (FT) (11.12.3)

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

C_F = Factor that considers non-uniform soil pressures behind a soil nail wall facing and is used in the estimation of nominal resistances at the soil nail head (DIM) (11.12.6)

C_P = Factor that accounts for soil contribution to support and is used in the estimation of nominal resistances at the soil nail head (DIM) (11.12.6)

D'_C = Effective, equivalent diameter of the potential slip conical failure in the facing around soil nail heads (FT) (11.12.6)

D_{DEF} = Horizontal distance behind soil nail wall where ground deformation can be significant (FT) (11.12.3)

D_{DH} = Average diameter of soil nail drill-hole (IN) (11.12.5)

D_E = Effective diameter of the core of a threaded anchor (IN) (C11.12.6)

D_H = Diameter of the head of a soil nail head connector (i.e., headed-stud) (IN) (11.12.6)

D_S = Diameter of the shaft of a soil nail head connector (i.e., headed-stud) (IN) (11.12.6)

f_c' = Concrete compressive nominal resistance (PSI) (11.12.6)

f_y = Yield tensile nominal resistance of soil nail bar (KSI) (11.12.5)

f_{y-f} = Yield tensile nominal resistance of reinforcement in facing (KSI) (11.12.6)

f_{y-hs} = Yield tensile nominal resistance of headed-stud in facing (KSI) (11.12.5)

h = Thickness of facing (IN) (11.12.6)

H = Wall height (FT) (11.12.3)

h_C = Effective depth of potential conical slip surface forming in facing around soil nail head (FT) (11.12.6)

h_f = Thickness of permanent facing (IN) (11.12.6)

h_t = Thickness of temporary facing (IN) (11.12.6)

K_A = Active earth pressure coefficient of soils behind soil nail wall (DIM) (C11.12.6)

L = Soil nail length (FT) (11.12.6)

L_{BP} = Bearing plate side dimension (FT) (11.12.6)

L_P = Pullout length extending behind slip surface (FT) (11.12.5)

L_S = Length of headed-stud (FT) (11.12.6)

m_{hm} = Horizontal flexural resistance (moment per unit length) mid-span between soil nails (KIP-IN/FT) (11.12.6)

m_{hn} = Horizontal flexural resistance (moment per unit length) at soil nail head (KIP-IN/FT) (11.12.6)

m_{vm} = Vertical flexural resistance (moment per unit length) mid-span between soil nails (KIP-IN/FT) (11.12.6)

m_{vn} = Vertical flexural resistance (moment per unit length) at soil nail head (KIP-IN/FT) (11.12.6)

N_H = Number of headed-studs in soil nail head connection (DIM) (11.12.6)

nt = Number of threads per unit length in threaded anchor (i.e., bolt) (IN) (C11.12.6)

q_U = Nominal bond resistance of soil nails (KSI) (11.12.5)

R_{FF} = Nominal resistance for flexure in facing (KIP) (11.12.6)

R_{FH} = Nominal resistance for tension of headed-studs located in facing (KIP) (11.12.6)

R_{FP} = Nominal resistance for punching-shear in facing (KIP) (11.12.6)

R_{PO} = Nominal pullout resistance of soil nails (KIP) (11.12.5)

r_{PO} = Nominal pullout resistance per unit length of soil nails (KIP/FT) (11.12.5)

R_T = Nominal resistance of a soil nail bar in tension (KIP) (11.12.5)

S_H = Horizontal spacing of soil nails (FT) (C11.12.6; 11.12.6)

S_{HS} = Spacing of headed-studs (FT) (11.12.6)

S_{max} = Maximum spacing of soil nails (FT) (C11.12.6)

S_V = Vertical spacing of soil nails (FT) (C11.12.6; 11.12.6)

t_H = Head thickness of headed-studs (FT) (11.12.6)

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PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

T_{max} = Maximum load in a soil nail (KIP) (11.12.6, 11.12.6)

T_o = Maximum load in the head of a soil nail (KIP) (11.12.6)

t_P = Thickness of bearing plate (FT) (11.12.6)

V_F = Punching-shear force acting through facing, around soil nail head (KIP) (11.12.6)

α = Angle of batter of soil nail wall (DEG) (11.12.3)

β = Backslope angle (DEG) (11.12.1)

δ_h = Horizontal displacement at the top of a soil nail wall (FT) (11.12.3)

δ_v = Vertical displacement at the top of a soil nail wall (FT) (11.12.3)

ϕ_{FF} = Resistance factor for flexure in facing (DIM) (11.12.6)

ϕ_{FH} = Resistance factor for facing headed-stud in tension (DIM) (11.12.6)

ϕ_{FP} = Resistance factor for punching-shear in facing (DIM) (11.12.6)

ϕ_{PO} = Resistance factor for nail pullout (DIM) (11.12.5)

ϕ_T = Resistance factor for nail bar in tension (DIM) (11.12.5)

γ_s = Unit weight of soil (KCF) (C11.12.6)

ρ_{ij} = Reinforcement ratio in "i" direction (vertical or horizontal) and location "j" (at nail head "n," or midspan "m" in-between soil nails) (PERCENT) (C11.12.6)

ρ_{max} = Maximum reinforcement ratio in facing (PERCENT) (11.12.6)

ρ_{min} = Minimum reinforcement ratio in facing (PERCENT) (11.12.6)

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS

11.5 LIMIT AND RESISTANCE FACTORS

11.5.2 Service Limit States

Deflections of soil nail walls shall be limited to the ranges presented in Section 11.12.4.

11.5.4 Resistance Requirement

Abutments …… 11.10, 11.11, or 11.12

11.5.6 Resistance Factors

The limit states shall be as specified in Article 1.3.2. Wall-specific provisions are contained in this article.

Walls shall be proportioned so that the factored resistance is not less than the effects of the factored loads specified in Section 3.

PROPOSED COMMENTARY

C11.5.2

In general, soil nail walls with concrete/shotcrete facing or with precast panels are more rigid than MSE walls with welded wire or geosynthetic facing.

C11.5.4

11.10, 11.11, and 11.12….., and soil nail walls, respectively

C11.5.6

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

Table 11.5.6-1 Resistance Factors

Limit State	Resistance	Condition		Resistance Factor	Value
Soil Failure	Sliding	All		ϕ_τ	0.90
	Basal Heave	ALL		ϕ_b	0.70
Overall Stability	NA	Slope does not support a structure		ϕ_s	0.75 (1)
		Slope supports a structure		ϕ_s	0.65 (2) (3)
		Seismic		ϕ_s	0.90 (4)
Structural	Nail in Tension	Static	Mild steel bars – Grades 60 and 75 (ASTM A 615)	ϕ_T	0.56
			High-resistance – Grade 150 (ASTM A 722)	ϕ_T	0.50
		Seismic	Mild steel bars – Grades 60 and 75 (ASTM A 615)	ϕ_T	0.74
			High-resistance – Grade 150 (ASTM A 722)	ϕ_T	0.67
	Facing Flexure	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FF}	0.67
	Facing Punching-Shear	Temporary and final facing reinforced shotcrete or concrete		ϕ_{FP}	0.67
	Facing Headed-Stud Tensile	A307 Steel Bolt (ASTM A 307)		ϕ_{FH}	0.50
		A325 Steel Bolt (ASTM A 325)		ϕ_{FH}	0.59
	Pullout	Soil/Rock Type	Sand	ϕ_{PO}	0.47 (5)
			Clay	ϕ_{PO}	0.51 (5)
			Weathered Rock	ϕ_{PO}	0.45 (5)
			All	ϕ_{PO}	0.49 (5)

Notes:
(1) Also when geotechnical parameters are well-defined.
(2) Also when geotechnical parameters are based on limited information.
(3) For temporary SNWs, use $\phi_s = 0.75$.
(4) Per current practice but subject to modifications. A value ϕ_s = 1.00 may be acceptable, as long as permanent deformations are calculated (see Anderson et al., 2008) and are found not to be excessive. Currently, there is no differentiation for temporary structures under seismic loading; therefore, use ϕ_s = 1.00.
(5) From reliability-based calibration. Values shown correspond to a load factor $\gamma = 1.00$.

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS

11.12 SOIL NAIL WALLS

11.12.1 General Considerations

Soil nail walls most commonly consist of: (a) a soil nail (i.e., steel bar) that is placed in a pre-drilled hole, then grouted along its entire length in the hole; (b) connectors in the soil nail head; and (c) a structurally continuous reinforced concrete or shotcrete cover (facing) connecting all nail heads. Figure 11.2.1-1a shows a cross-section of a typical soil nail wall and main components.

Horizontal nail spacing, S_H, is typically the same as vertical nail spacing, S_V, and can be between 4 and 6.5 FT, and most commonly 5 FT. Soil nail spacing may be modified to accommodate the presence of existing underground structures or utilities behind the wall.

Soil nail spacing in horizontal and vertical direction must be such that each nail has an influence area $S_H \times S_V \leq 40$ FT^2.

PROPOSED COMMENTARY

C11.12.1

Soil nail walls are top-down construction structures that are particularly well suited for ground conditions that require vertical or near-vertical cuts. Favorable ground conditions make soil nailing technically feasible and cost effective, compared with other techniques, when:

- the soil in which the excavation is advanced is able to stand unsupported in vertical or nearly vertical, 3- to 6-FT high cuts for one to two days;
- all soil nails are above the groundwater table; and
- the long-term integrity of the soil nails can be maintained through corrosion protection.

Subsurface conditions that are generally well suited for soil nails applications include stiff to hard fine-grained soils, dense to very dense granular soils with some cohesion (apparent cohesion due to cementation), weathered rock without weakness planes, and other competent soils with a wide gradation (i.e., glacial tills).

Examples of unfavorable soil types and ground conditions include dry, loose, poorly graded cohesionless soil, soils with high groundwater, soils with cobbles and boulders, soft to very soft fine-grained soils, organic soil, highly corrosive soil (e.g., cinder, slag), weathered rock with weakness planes, karstic ground, loess, and soils that generally have a liquidity index ≥ 0.2.

Corrosion protection is provided by grouting, epoxy coating, galvanized coating, or encapsulation [not shown in Figure 11.12.1-1(a)]. See Section 11.12.8 "Corrosion Protection" for references to consider corrosion protection in the design.

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

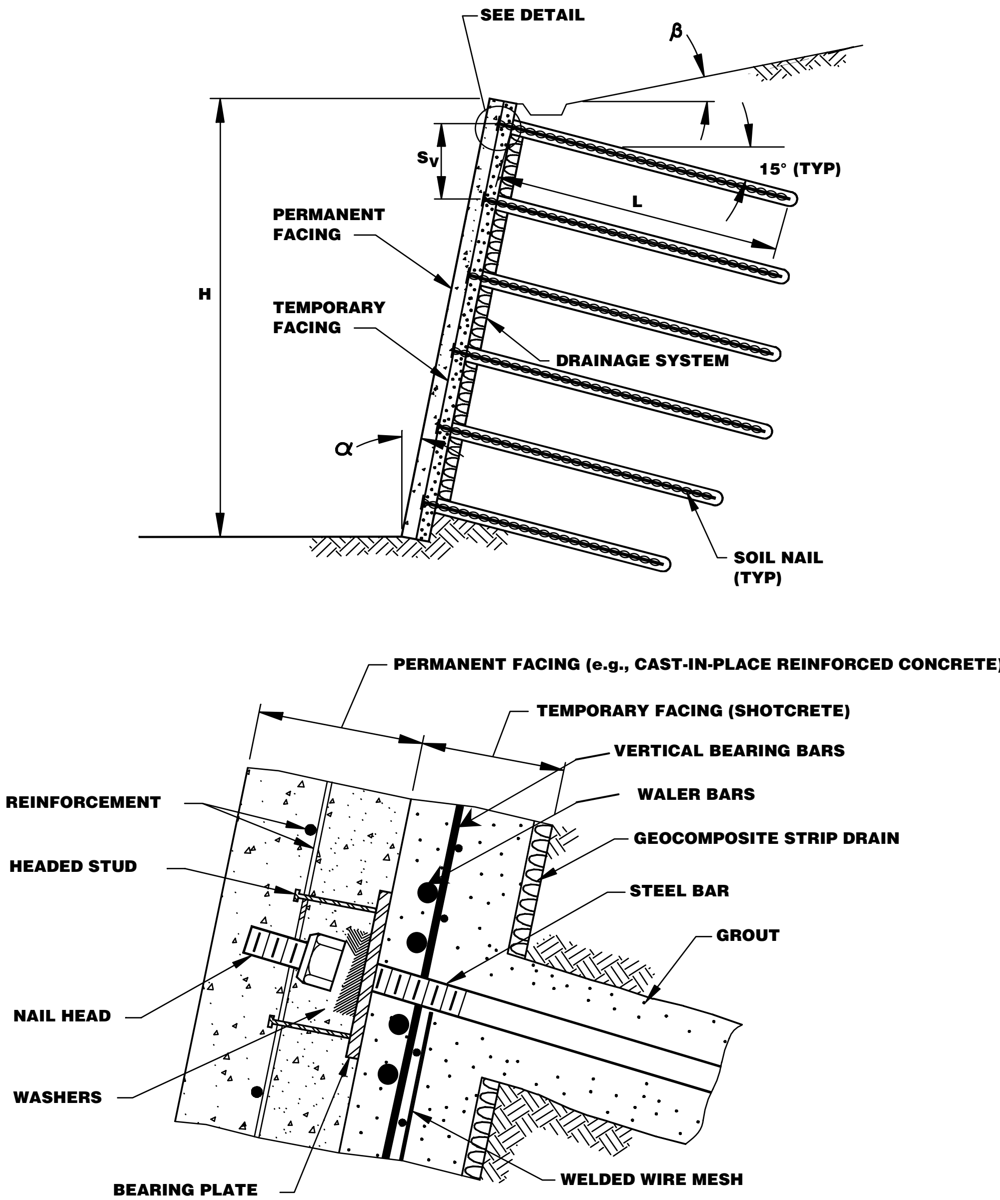

Figure 11.12.1-1 Soil Nail Wall : (a) Typical Section, (b) Nail Head and Facing Details

11.12.2 Loading

The provisions of Article 11.5 shall apply.

C11.12.2

When a soil nail wall is part of a bridge abutment, the effect on the soil nail wall due to shrinkage and temperature from the bridge deck shall be evaluated from structure analysis.

11.12.3 Movement and Stability at the Service Limit State

11.2.3.1 Abutments

The provisions of Articles 10.6.2.4, 10.6.2.5, 10.7.2.3 through 10.7.2.5, 10.8.2.2 through 10.8.2.4, and 11.5.2 shall apply as applicable.

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PROPOSED SPECIFICATIONS

11.12.3.2 Displacements

The considerations of Article 11.6.2.2 shall be considered.

A soil nail wall shall be designed so as the movements of the wall remain within tolerable ranges.

PROPOSED COMMENTARY

C11.12.3.2

In addition to the considerations of article 11.6.2.2, the effects of the movement of a soil nail wall on adjacent structures shall be considered in the design.

Empirical data indicate that for soil nail walls with: (a) nail-length ratios, L/H, between 0.7 and 1.0; (b) negligible surcharge loads; and (c) adequate safety margins achieved for overall stability, the maximum long-term horizontal and vertical displacements at the top of the wall, δ_h and δ_v, respectively, can be estimated as follows (Byrne et al., 1998):

$$\delta_h = \left(\frac{\delta_h}{H}\right) \times H \qquad \text{(C11.12.3-1)}$$

$$\delta_v \approx \delta_h \qquad \text{(C11.12.3-2)}$$

where:

(δ_h/H) = ratio presented in Table 11.12.3.2-1 (DIM)

H = wall height (FT)

Ground deformation considered to be of significance can occur within a horizontal distance, D_{DEF}, which can be estimated as follows:

$$\frac{D_{DEF}}{H} = C\,(1 - \tan\alpha) \qquad \text{(C11.12.3-3)}$$

where:

α = batter angle of wall (DEG)

C = coefficient presented in Table 11.12.3.2-1 (DIM)

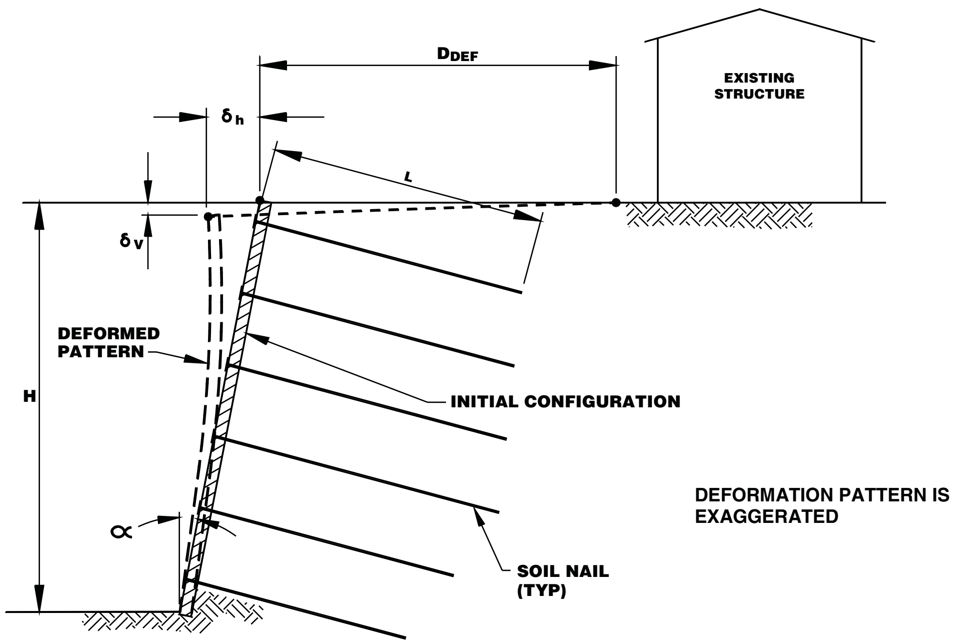
For soil nail walls resisting relatively large loads (e.g., walls being part of bridges abutments), more advanced methods (e.g., finite element method) may be required to produce a more precise estimation of the wall deformation.

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

Table 11.12.3.2-1 Values of (δ_h/H) and C as Functions of Soil Conditions

Variable	Weathered Rock and Stiff Soil	Sandy Soil	Fine-Grained Soil
(δ_h/H)	1/1,000	1/500	1/333
C	0.8	1.25	1.5



Modified after Byrne et al. (1998)

Figure 11.12.3.2-1 Deformation of Soil Nail Walls

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS

11.12.3.3 Overall Stability

The provisions of Article 11.6.2.3 shall apply.

The evaluation of overall stability of soil nail walls shall be performed using acceptable methods that consider all reinforcement elements of a soil nail and loads.

Global stability analyses may be necessary for intermediate excavation conditions.

The potential slip surfaces to be considered in overall stability may or may not intersect soil nails (Figure 11.12.3.3-1). For the case of slip surfaces intersecting soil nails, the nominal resistance of soil nails shall be adequately considered in analyses.

For soil nail walls with complex geometry (e.g., multiple-tiered walls) involving composed failure surfaces, the provisions of Article 11.10.4.3 shall apply.

PROPOSED COMMENTARY

C11.12.3.3

Overall stability of soil nail walls is commonly evaluated using two-dimensional limit-equilibrium-based methods, in which the contribution of nails is accounted for in equilibrium equations.

Stability analyses of soil nail walls are commonly performed using computer programs specifically developed for the design of soil nail walls. Other computer programs developed for general slope stability analysis can also be used, if various reinforcement bars developing pullout resistance can be considered by the software.

Figure 11.12.3.3-1 Limit States in Soil Nail Walls—Overall Stability: (a) Slip Surface not Intersecting Nails; (b) Slip Surface Intersecting Nails

11.12.3.4 Seismic Effects on Global Stability

The pseudo-static method shall be routinely used for the seismic stability analysis of soil nail walls. The provisions of Article 11.6.5 shall apply to consider the effect of seismic loads on the global stability of soil nail walls.

In general, the vertical seismic coefficient is disregarded in global stability analysis.

For flexible structures such as soil nail walls, it is reasonable to use horizontal seismic coefficients that are a function of the expected seismically induced wall displacement. The following expressions can be used to estimate the horizontal seismic coefficient as a function of the tolerable seismically induced wall lateral movement, d, in inches before any wall/sliding block takes place (Kavazanjian et al., 1997; Elias et al., 2001):

$$k_h = 0.74\,A_m \left(\frac{A_m}{d}\right)^{0.25} \qquad \text{(C11.12.3.4-1)}$$

where:

k_h = horizontal seismic coefficient (DIM)

PROPOSED SPECIFICATIONS

PROPOSED COMMENTARY

A_m = normalized horizontal acceleration (DIM)

d = seismically induced wall lateral movement (INCH)

The value of A_m is a function of the normalized peak ground acceleration coefficient, A, which is defined in Appendix 11A, Seismic Design of Abutments and Gravity Retaining Structures.

Equation C11.12.3.4-1 should be used only for $1 \leq d \leq 8$ IN, with more typical values of d between 2 and 4 IN The selection of smaller tolerable seismically induced deformation results in larger seismic coefficients, which results in larger nail lengths.

Elias et al. (2001) recommend that Equation C11.12.3.4-1 should not be used when:

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

If the seismically induced displacement is not available, it is acceptable, in general, to select a seismic coefficient for soil nail walls between:

$$k_h = 0.5\,A_m \quad \text{to} \quad 0.67\,A_m \qquad \text{(C11.12.3.4-2)}$$

11.12.4 Stability at Strength Limit States: Safety Against Soil Failure

Soil nail walls shall be proportioned to satisfy sliding and bearing criteria normally associated with gravity structures as shown in Figure 11.12.4-1.

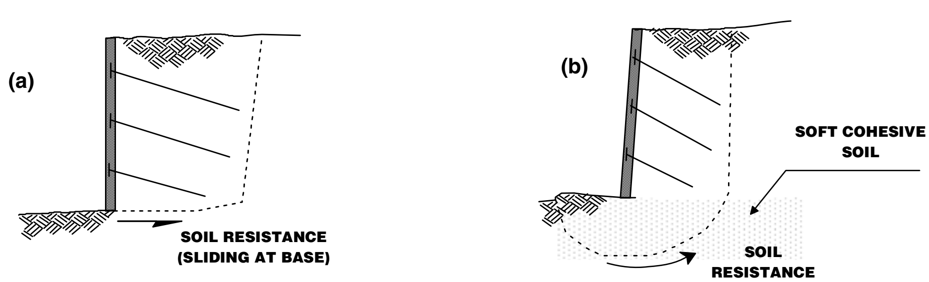


Figure 11.12.4-1 Soil Limit States: (a) Sliding Stability; (b) Basal Heave

11.12.4.1 Sliding

Soil nail walls shall resist sliding along the base of the retained system in response to lateral earth pressures behind the soil nails.

C11.12.4.1

Sliding is a feasible but uncommon limit state for soil nail walls and is considered here for consistency with other retaining systems. Sliding may become a

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS

The general principles referred to in Article 10.6.3.4 shall apply.

11.12.4.2 Basal Heave

The bearing resistance shall be evaluated if the soil nail wall is constructed in or over soft fine-grained soils.

The bearing resistance shall be evaluated as a service limit state, based on equilibrium, not deformations.

11.12.5 Stability at Strength Limit States: Safety Against Structural Failure

11.12.5.1 General

The structural limit states to consider for soil nail walls include soil nail pullout and soil nail in tension, as illustrated schematically in Figure 11.12.5.1-1.

Figure 11.12.5.1-1 Structural Limit States: (a) Pullout; (b) Nail in Tension

11.12.5.2 Nail Pullout Resistance

The nominal pullout resistance (per unit length) of soil nails, r_{PO}, can be expressed as:

$$r_{PO} = \pi\, q_U\, D_{DH} \times 12 \qquad (11.12.5.2\text{-}1)$$

where:

q_U = nominal bond resistance (KSI)

D_{DH} = average diameter of drill-hole (IN)

The pullout resistance, R_{PO} (KIP), is computed as:

$$R_{PO} = r_{PO}\, L_P \qquad (11.12.5.2\text{-}2)$$

It shall be verified that:

PROPOSED COMMENTARY

more realistic limit state when the block of soil resisted by a soil nail wall is underlain by a weak soil layer. In this case, the critical slip surface may be oriented along the weak soil layer.

C11.12.4.2

When soft cohesive soils exist at the base of a soil nail wall, the potential for basal heave at the base of the excavation should be evaluated. If the loads generated due to the excavation are excessive for the existing soft soil conditions, the bottom of the excavation may heave and possibly cause a basal heave failure. SNWs may be more susceptible to basal heave than other retaining systems (e.g., anchored walls) because the facing of soil nail walls is not or seldom embedded in the underlying soil.

C11.12.5.2

The nominal pullout resistance is also referred to as nominal load transfer rate, with units KIP/FT.

A uniform distribution of resistance along the pullout length behind the slip surface, L_P, is assumed.

PROPOSED SPECIFICATIONS

$\phi_{PO} R_{PO} \geq T_{max}$

where:

R_{PO} = nominal pullout resistance (KIP)

ϕ_{PO} = resistance factor for soil nail pullout (DIM)

T_{max} = maximum tensile load in a soil nail (KIP).

11.12.5.3 Nominal Bond Resistance

Table 11.12.5.3-1 provides presumptive values of the nominal bond resistance for soil nails installed in soil or rock.

The nominal bond resistance is in general a function of the soil/rock type, soil nail installation method, and soil/rock condition, as seen in Table 11.12.5.3-1.

Verification load tests (and possibly proof load tests) can provide information to assess nominal values of the soil nail bond resistance. See details on soil nail load testing in Appendix B, Proposed LRFD Construction Specifications for Soil Nail Walls; Byrne et al. (1998); and Lazarte et al. (2003).

Proof load tests shall be conducted on at least 5 percent of all production soil nails and up to a load of 150 percent of test design loads. Design loads are derived from presumptive nominal pullout resistances and test bonded lengths. Verification load tests should be conducted on a project-specific basis and up to a load of 200 percent of the test load.

Pullout nominal resistance values of soil nails can be estimated as the maximum load obtained from verification tests (i.e., 200 percent of the design load) times the pullout resistance contained in Table 11.5.6-1.

PROPOSED COMMENTARY

C11.12.5.3

The nominal bond resistance of drilled and grouted soil nails is affected by various factors, including:

- Conditions of the ground around soil nails, namely:
  - soil type;
  - soil characteristics;
  - magnitude of overburden.
- Conditions at time of soil nail installation, namely:
  - drilling method (e.g., rotary drilled, driven casing, etc.);
  - drill-hole cleaning procedure;
  - grout injection method (e.g., under gravity or with a nominal, low pressure);
  - grouting procedure (e.g., tremie method); and
  - grout characteristics (e.g., grout workability and compressive strength).

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

Table 11.12.5.3-1 Presumptive Nominal Bond Resistance for Soil Nails in Soil and Rock

Material	Soil Nail Installion Method	Soil/Rock Type	Nominal Bond Nominal Resistance, q_u (psi)
Rock	Rotary Drilling	Marl/limestone	45 - 58
		Phyllite	15 - 45
		Chalk	75 - 90
		Dolomite (soft)	60 - 90
		Dolomite (fissured)	90 - 145
		Sandstone (weathered)	30 - 45
		Shale (weathered)	15 - 22
		Schist (weathered)	15 - 25
		Basalt	75 - 90
		Slate/hard shale	45 - 60
Cohesionless Soils	Rotary Drilling	Sand/gravel	15 - 26
		Silty sand	15 - 22
		Silt	9 - 11
		Piedmont residual	6 - 17
		Fine Colluvium	11 - 22
	Driven Casing	Sand/gravel	
		low overburden	28 - 35
		high overburden	40 - 62
		Dense Moraine	55 - 70
		Colluvium	15 - 26
	Auger	Silty sand fill	3 - 6
		Silty fine sand	8 - 13
		Silty clayey sand	9 - 20
Fine-Grained Soils	Rotary Drilling	Silty clay	5 - 7
	Driven Casing	Clayey silt	13 - 20
	Auger	Loess	4 - 11
		Soft clay	3 - 4
		Stiff clay	6 - 9
		Stiff clayey silt	6 - 15
		Calcareous sandy clay	13 - 20

Modified after Elias and Juran (1991).

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11.12.5.4 Limit State for Soil Nail in Tension

The limit state for a soil nail in tension shall be verified as follows:

$$\phi_T \, R_T \geq T_{max} \quad (11.12.5.4\text{-}1)$$

where:

ϕ_T = resistance factor for soil nail in tension (DIM)

R_T = nominal tensile resistance of a soil nail (KIP)

T_{max} = maximum load in soil nail (KIP)

The nominal tensile resistance of a soil nail shall be computed as:

$$R_T = A_t \, f_y \quad (11.12.5.4\text{-}2)$$

where:

A_t = cross-sectional area of a soil nail bar (IN^2)

f_y = nominal yield resistance of soil nail bar (KSI)

11.12.6 Strength Limit States: Limit States for the Facing of Soil Nail Walls

11.12.6.1 General

The limit states of the facing of a soil nail wall that shall be considered include: (a) flexure; (b) punching-shear; and (c) headed-stud in tension. These limit states are shown schematically in Figure 11.12.6.1-1(a) and (c).

PROPOSED COMMENTARY

C11.12.5.4

The contribution of the grout to the nominal resistance in tension shall be disregarded.

T_{max} is estimated from global stability analyses performed with computer programs.

C11.12.6.1

The limit states for flexure and punching-shear in the facing shall be considered separately for the temporary and the permanent facing. The limit state for tension in the headed-stud shall be considered only in permanent facings.

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PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

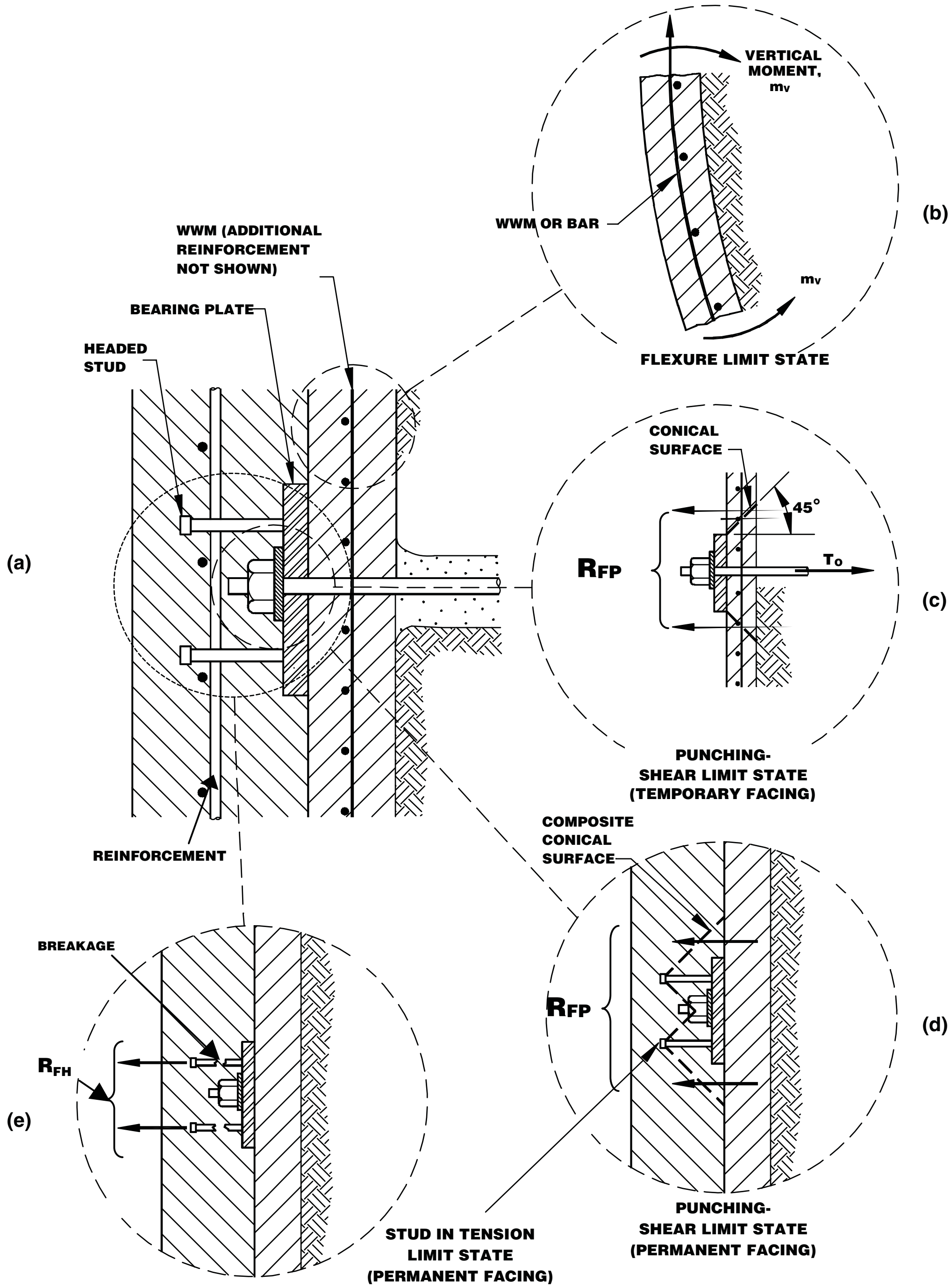

Figure 11.12.6.1-1 Limit States in Soil Nail Wall Facings: (a) Typical Section; (b) Flexure; (c) Punching-Shear in Temporary Facing; (d) Punching-Shear in Permanent Facing; and (e) Headed-Stud in Tension

11.12.6.2 Flexural Limit State

For the limit state of flexure in the facing, it shall be verified that:

$$\phi_{FF}\ R_{FF} \geq T_o \qquad (11.12.6.2\text{-}1)$$

C11.12.6.2

Section 11 - Abutments, Piers and Walls

PROPOSED SPECIFICATIONS

where:

ϕ_{FF} = resistance factor for flexure in the facing (DIM)

R_{FF} = nominal tensile resistance for flexure in the facing (KIP)

T_o = maximum tensile load at soil nail head (at facing) (KIP)

R_{FF} can be estimated using the following expression:

$$R_{FF}\,[\text{kip}] = 3.8 \times C_F \times f_y\,[\text{ksi}] \times \text{greater of} \begin{cases} \left(a_{vn} + a_{vm}\right)\left[\text{in}^2/\text{ft}\right] \times \left(\dfrac{S_H}{S_V} h[\text{ft}]\right) \\ \left(a_{hn} + a_{hn}\right)\left[\text{in}^2/\text{ft}\right] \times \left(\dfrac{S_V}{S_H} h[\text{ft}]\right) \end{cases} \quad (11.12.6.2\text{-}2)$$

where:

C_F = f actor that considers non-uniform soil pressures behind a soil nail wall facing and is used in the estimation of nominal resistances at the soil nail head (DIM)

h = thickness of facing (IN) that can take the values h_t or h_f.

a_{vn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction, at soil nail heads (IN^2/FT)

a_{vm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the vertical direction, at soil nail heads (IN^2/FT)

a_{hn} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at soil nail heads (IN^2/FT)

a_{hm} = Cross-sectional area (per unit width) of mesh reinforcement in the wall facing, in the horizontal direction, at midspan between soil nails (IN^2/FT)

$f_{y\text{-}f}$ = Yield tensile nominal resistance of reinforcement in facing (KSI)

The cross-sectional areas of reinforcement per unit width in the vertical or horizontal direction and around and in-between nails are shown schematically in Figure

PROPOSED COMMENTARY

The nail head tensile force may be estimated based on the equations below (Clouterre, 1991) that were developed for working conditions:

$$T_o = T_{max}\left[0.6 + 0.057\,(S_{max} - 3)\right] \le T_{max} \quad (C11.12.6.2\text{-}1)$$

where:

T_{max} = maximum nail load (KIP)

S_{max} = maximum soil nail spacing (i.e., greater of S_V and S_H) (FT)

The maximum nail load under working conditions typically varies from $T_o = 0.60\,K_A\,\gamma H\,S_V\,S_H$ to $0.70\,K_A\,\gamma_s\,H\,S_V\,S_H$ (Byrne et al., 1998), where K_A is the active earth pressure coefficient, γ_s is the unit weight of the soil behind the wall, H is the wall height, and S_V and S_H are the nail vertical and horizontal spacing, respectively.

The nominal resistance for flexure in the facing depends on the soil pressures mobilized behind the facing, horizontal and vertical soil nail spacing, soil conditions, and facing stiffness. To account for non-uniform soil pressure distributions and other conditions, C_F is used (Byrne et al., 1998).

Table C11.12.6.2-1 presents values of C_F for typical facing thickness. For all permanent facings and "thick" (i.e., $h_t \ge 8$ IN) temporary facings, the soil pressure is assumed to be relatively uniform.

Reinforcement can be welded wire mesh (WWM) or concrete reinforcement bars.

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

$$a_{vn} = a_{vm} + \frac{A_{VH}}{S_H} \quad (C11.12.6.2\text{-}1)$$

where:

A_{VH} = Total cross-sectional area of additional reinforcement (rebar) in wall facing, in the

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11.12.7.2-2.

The nomenclature for the reinforcement areas per unit width is presented in Table 11.12.3.2-2:

PROPOSED COMMENTARY

vertical direction and around soil nail heads (IN^2)

Similar concepts can be applied if additional horizontal rebar (i.e., waler bars) is used in this direction. The total reinforcement area per unit length in the horizontal direction can then be calculated as:

$$a_{hn} = a_{hm} + \frac{A_{HH}}{S_V} \tag{C11.12.6.2-2}$$

A_{HH} = Total cross-sectional area of additional reinforcement (i.e., waler bars) in wall facing, in the horizontal direction and around soil nail heads (IN^2)

Table 11.12.6.2-1 Factor C_F

Type of Wall	Nominal Facing Thickness, h_t or h_f (IN)	Factor C_F
Temporary	4	2.0
	6	1.5
	8	1.0
Permanent	All	1.0

Table 11.12.6.2-2 Nomenclature for Facing Reinforcement Area per Unit Width

Direction	Location	Cross-Sectional Area of Reinforcement per Unit Width
Vertical	Nail head (1)	$a_{vn} = a_{VM} + \frac{A_{VH}}{S_H}$
	Mid-span	a_{vm}
Horizontal	Nail head (2)	$a_{hn} = a_{hm} + \frac{A_{HH}}{S_V}$
	Mid-span	a_{hm}

Notes: (1) At the nail head, the total cross-sectional area (per unit length) of reinforcement is the sum of the welded-wire mesh area, a_{vm}, and the area of additional vertical bars, A_{VH}, divided by the horizontal spacing, S_h.

(2) At the nail head, the total area is the sum of the area of the welded-wire mesh, a_{hm}, and the area of additional horizontal bars (i.e., waler bars, A_{HH}) divided by S_v.

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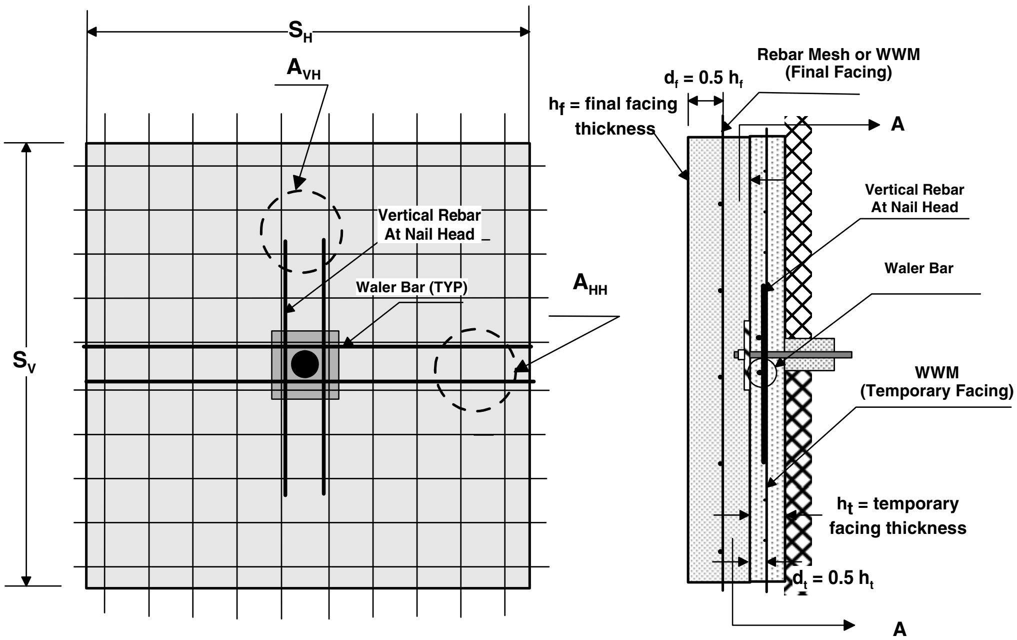
PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**



Total Cross Sectional Area (per unit length)

Vertical		Horizontal	
Mid-span between nails: a_{vm}		Mid-span between nails: a_{hm}	
At nail head:	$a_{vn} = a_{vm} + \frac{A_{VH}}{S_H}$	At nail head:	$a_{hn} = a_{hm} + \frac{A_{HH}}{S_V}$

Figure 11.12.6.2-2 Geometry Used in Flexural Limit State

The minimum and maximum amount of steel reinforcement to be placed in the facing, ρ_{min} and ρ_{max}, respectively, shall be as follows:

$$\rho_{min}[\%] = 0.24 \frac{\sqrt{f'_c}}{f_{y\text{-}f}} \quad (11.12.6.2\text{-}3)$$

$$\rho_{max}[\%] = 0.05 \frac{f'_c}{f_{y\text{-}f}}\left(\frac{90}{90 + f_{y\text{-}f}}\right) \quad (11.12.6.2\text{-}4)$$

where:

f_c' = concrete compressive nominal resistance (PSI)

$f_{y\text{-}f}$ = reinforcement tensile yield nominal resistance (KSI)

The reinforcement ratio, ρ, shall be calculated as:

$$\rho = \frac{a_{ij}}{0.5\,h}\ 100 \quad (C11.12.6.2\text{-}3)$$

where:

a_{ij} = ratio of cross-sectional area of reinforcement per unit width (in "i" direction and "j" location) (PERCENT)

The direction "i" can be vertical or horizontal; the location "j" can be at the nail head or mid-span, giving rise to the four possible cross-sectional areas noted in Figure 11.12.6.2-2.

In addition to the minimum and maximum ratios indicated in this section, the ratios a_{vn}/a_{vm} or a_{hn}/a_{hm} should be limited to values less than 2.5.

11.12.6.3 Punching-Shear Resistance in Facing

For the limit state of punching-shear in the facing, it shall be verified that:

C11.12.6.3

The limit state for punching-shear may involve the formation of a localized, conical slip surface around the nail head. The slip surface may extend behind the

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$$\phi_{FP}\, R_{FP} \geq T_o \qquad (11.12.6.3\text{-}1)$$

where:

ϕ_{FP} = resistance factor for punching-shear in the facing (DIM)

R_{FP} = nominal resistance for punching-shear in facing (KIP)

The factored soil nail tensile force from punching-shear failure shall be calculated as:

$$R_{FP} = C_P V_F \qquad (11.12.6.3\text{-}2)$$

where:

V_F = nominal punching-shear resistance acting through the facing section (KIP)

C_P = correction factor that accounts for the contribution of the support resistance of the soil (DIM)

The nominal punching-shear resistance shall be calculated as:

$$V_F = 0.58\sqrt{f_c'}\ \pi\ D_c' h_C \qquad (11.12.6.3\text{-}3)$$

where:

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

h_C = effective depth of conical surface (FT), as discussed below.

PROPOSED COMMENTARY

bearing plate or headed studs and may punch through the facing thickness at an inclination of about 45 degrees and form two punching limit states (Figure 11.12.6.3-1).

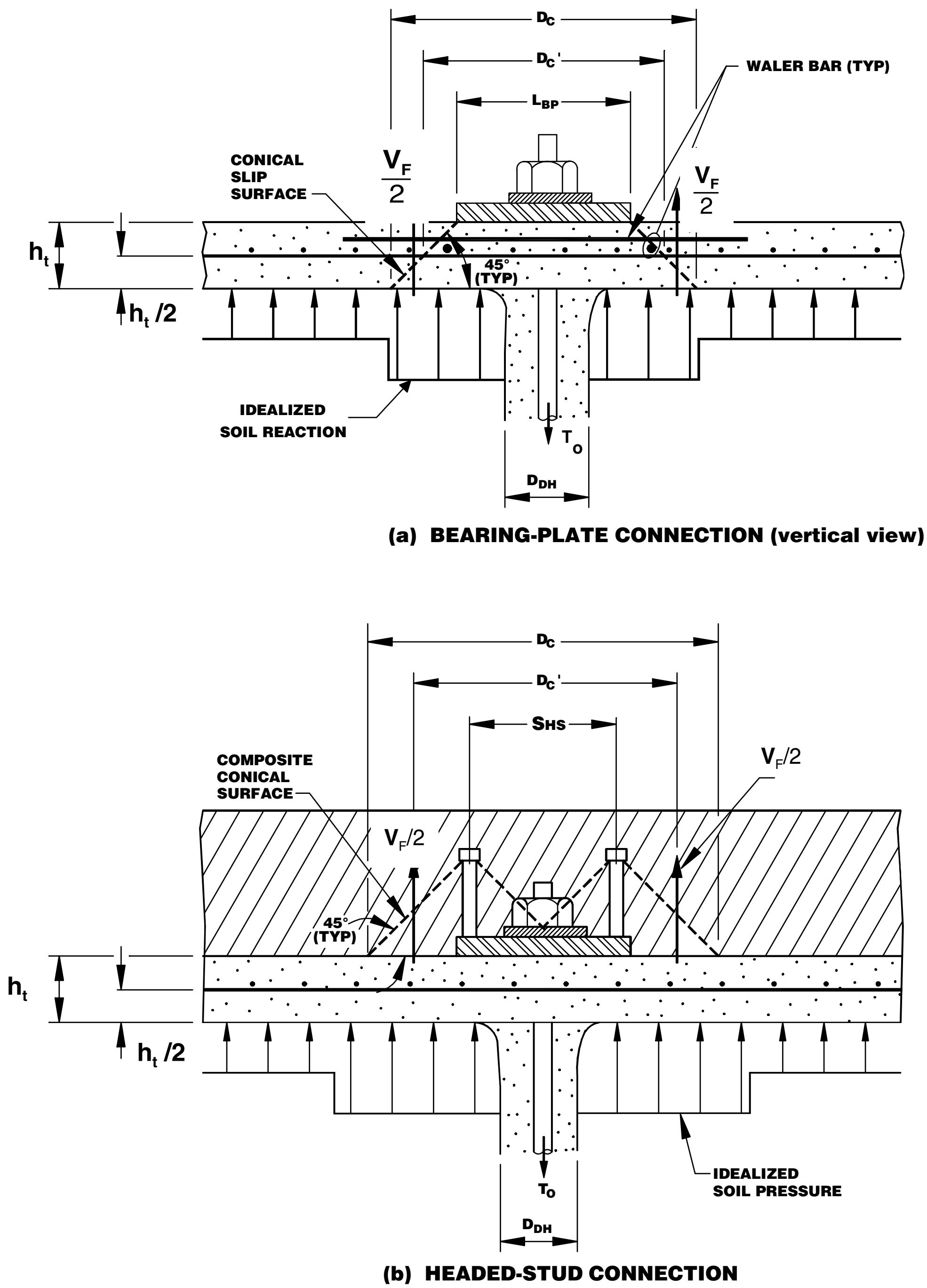
The size of the conical slip surface depends on the facing thickness and the type of the nail-facing connection (i.e., bearing-plate or headed-studs).

Generally, the contribution from the soil support is ignored and C_P = 1.0. If the soil reaction is considered, C_P can assume values up to 1.15.

These equations shall be separately used for temporary and permanent facing. The maximum and average diameters of the slip surface (D_C and D'_C on Figure 11.12.6.3-1), as well as the effective depth of the slip surface (h_C) shall be selected separately for temporary and permanent facings.

For temporary facing, only the dimensions of the bearing plate and facing thickness shall be considered. For permanent facings, the dimensions of headed-studs and bearing plate, and the facing thickness shall be considered.

PROPOSED SPECIFICATIONS | **PROPOSED COMMENTARY**



(Horizontal Cross Sections)

Modified after Byrne et al. (1998)

Figure 11.12.6.3-1 Punching-Shear Limit States

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The effective diameter of the slip surfaces must be considered as follows:

Temporary facing

D'_C = $L_{BP} + h_t$
h_C = h_t

where:

L_{BP} = bearing plate length (FT)
h_t = temporary facing thickness (FT)

Permanent facing

D'_C = minimum of ($S_{HS} + h_C$, or $2h_C$)
h_C = $L_S - t_h + t_P$

where:

S_{HS} = headed-stud spacing (FT)
L_S = headed-stud length (FT)
t_H = headed-stud head thickness (FT)
t_P = bearing plate thickness (FT)

11.12.6.4 Headed-Stud in Tension

For the limit state of facing headed-stud in tension, it shall be verified that:

$$\phi_{FH}\ R_{FH} \geq T_o \qquad (11.12.6.4\text{-}1)$$

where:

ϕ_{FH} = resistance factor for headed-stud in tension (DIM)

R_{FH} = nominal tensile resistance of headed-stud (KIP)

R_{FH} is computed as:

$$R_{FH} = N_H\ A_S\ f_{y\text{-}hs} \qquad (11.12.6.4\text{-}2)$$

where:

N_H = number of headed-studs in the connection (usually 4) (DIM)

A_S = cross-sectional area of the headed-stud shaft (IN^2)

$f_{y\text{-}hs}$ = yield tensile nominal resistance of headed-stud in facing (KSI)

PROPOSED COMMENTARY

C11.12.6.4

To provide sufficient anchorage, the length of the headed-studs shall extend beyond the mid-section of the facing, while maintaining 2 IN minimum cover.

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

$$A_E = \frac{\pi}{4}\left[D_E - \left(\frac{0.9743}{n_t}\right)\right]^2 \qquad (C11.12.6.4\text{-}1)$$

where:

D_E = effective diameter of the bolt core

n_t = number of threads per unit length

PROPOSED SPECIFICATIONS

In addition, the limit state for compression of the concrete behind the head of the headed-stud shall be established by assuring that the following geometric constraints are met (ACI, 1998):

$$A_H \geq 2.5\ A_S \qquad (11.12.6.4\text{-}3)$$

$$t_H \geq 0.5\ (D_H - D_S) \qquad (11.12.6.4\text{-}4)$$

where (see Figure 11.12.6.4-1):

A_H = cross-sectional area of the stud head

t_H = head thickness

D_H = diameter of the stud head

D_S = diameter of the headed-stud shaft

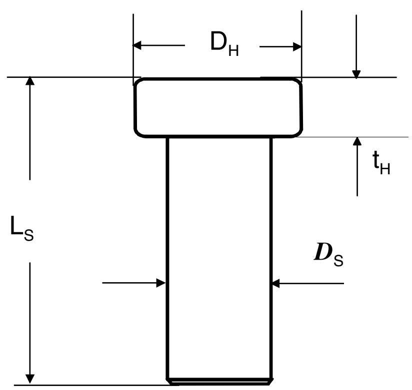

Figure 11.12.6.4-1 Geometry of a Headed-Stud

11.12.7 Drainage

Surface water runoff and groundwater shall be controlled both during and after construction of the soil nail wall. If appropriate performance cannot be achieved, the effect of the groundwater table shall be considered in the analysis.

11.12.8 Corrosion Protection

For all permanent soil nail walls and, in some cases, for temporary walls, the soil corrosion potential shall be evaluated and considered part of the design. See Appendix B, Proposed LRFD Construction Specifications for Soil Nail Walls.

PROPOSED COMMENTARY

C11.12.7 Drainage

Permanent surface and groundwater controls may consist of a combination of the following features: permanent surface water controls, geocomposite drain strips, shallow drains (weep-holes), toe drain, and drain pipes.

Geocomposite drain strips are routinely placed in vertical strips against the excavation face along the entire depth of the wall. The lower end of the strips typically discharges into a pipe drain that runs along the base of the wall or through weep holes at the bottom of the wall.

C11.12.8

A full discussion on corrosion of metallic components and a methodology that assists in selecting the appropriate level of corrosion protection of soil nail walls is presented in Lazarte et al. (2003).

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PROPOSED SPECIFICATIONS **PROPOSED COMMENTARY**

REFERENCES

American Concrete Institute (ACI) (1998) "Code Requirements for Nuclear Safety-Related Concrete Structures (ACI 349-97) and commentary," ACI 349R-97, ACI Committee 349, American Concrete Institute, Farmington Hills, MI, p. 129.

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

Clouterre (1991). "Recommendations Clouterre 1991" (Trans.: Soil Nailing Recommendations 1991), English Translation, Presses de l'Ecole Nationale des Ponts et Chaussées, Paris, France.

Elias, V., Christopher, B.R., and Berg, R. (2001). "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines," Federal Highway Administration, FHWA-NHI-00-043, Washington, D.C., 394 pp.

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

Kavazanjian, E., Jr., Matasović, N., Hadj-Hamou, T., and Sabatini, P.J. (1997). "Design Guidance: Geotechnical Earthquake Engineering for Highways, Volume I, Design Principles," Geotechnical Engineering Circular No. 3, FHWA-SA-97-076, Federal Highway Administration, Washington, D.C.

Lazarte, C.A., V. Elias, R.D. Espinoza, and P.J. Sabatini (2003). "Soil Nail Walls," Geotechnical Engineering Circular No. 7, FHWA0-IF-03-017, Federal Highway Administration, Washington, D.C., 305p.

APPENDIX B

Proposed LRFD Construction Specifications for Soil Nail Walls

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1.1 DESCRIPTION

This work consists of constructing a permanent soil nail wall as specified herein and as shown on the Plans. The Contractor shall furnish all labor, materials, and equipment required to complete the work. The Contractor shall select the excavation, drilling, and grouting methods and the diameter of the drill-holes to meet the performance requirements specified herein or shown on the Plans.

The work shall include excavating in staged lifts in accordance with the approved Contractor's plan; detailing the drilling of the soil nail drill-holes to the diameter and length required to develop the specified resistance; grouting the soil nails; providing and installing the specified drainage features; providing and installing bearing plates, washers, nuts, and other required miscellaneous materials; and constructing the required temporary shotcrete face and constructing the final structural facing.

1.2 MATERIALS

1.2.1 Facing

Facing material shall conform to the following sections and subsections.

1.2.1.1 Cast-in-place Concrete

Cast-in-place (CIP) concrete shall meet the requirements of Section 5 of the AASHTO LRFD Bridge Construction Specifications.

1.2.1.2 Reinforcing Steel

Reinforcing steel shall meet the requirements of Section 6 of the AASHTO LRFD Bridge Construction Specifications.

1.2.1.3 Permanent Shotcrete

Permanent shotcrete shall meet the requirements of Section 24, "Pneumatically Applied Mortar," of the AASHTO LRFD Bridge Construction Specifications.

1.2.1.4 Architectural Surface Finishes

Architectural surface finishes may include textured surfaces or a surface finish with color/stain application.

1.2.2 Soil Nails

1.2.2.1 Solid Soil Nail Bar

Solid nail bars shall meet the requirements of AASHTO M 31/ASTM A 615, Grade 420 or 520, or ASTM A 722 for Grade 1035. Soil nail bars shall be continuous without splices or welds,

PROPOSED COMMENTARY

C1.1

The Owner may choose to have the Contractor design and construct the work at the location shown on the drawings. The Contractor shall furnish all labor, plans, drawings, design calculations, and all other material and equipment required to design and construct the soil nail wall(s) in accordance with this Specification.

Project-specific needs may require a different type of facing including reinforced shotcrete, cast-in-place concrete, and precast concrete panels.

C1.2.2.1

Bars of A 722, Grade 1035, should not be used in conventional soil-nailing applications because the material tends to be more brittle and more susceptible to stress corrosion

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new, straight, undamaged, bare or epoxy coated, or encapsulated as shown on the Plans. Bars shall be threaded a minimum of 150 mm on the wall anchorage end, to allow proper attachment of the bearing plate and nut. If threads are cut into a soil nail bar, provide the next larger bar number designation than what is shown on the Plans, at no additional cost.

1.2.2.2 Bar Coupler

Bar couplers shall develop the full nominal tensile capacity of the soil nail bars as certified by the manufacturer.

1.2.2.3 Fusion-Bonded Epoxy Coating

Fusion-bonded epoxy coating shall meet the requirements of ASTM A 775 and have a minimum thickness of 0.4 mm (0.016 in.) as applied electrostatically. Bend test requirements are waived.

1.2.2.4 Encapsulation

Bar encapsulation shall be a minimum 1-mm (0.04-in.) thick, corrugated, HDPE tube conforming to AASHTO M 252, or corrugated PVC tube conforming to ASTM D 1784, Class 13464-B.

1.2.3 Soil Nail Appurtenances

1.2.3.1 Centralizer

Centralizers shall be manufactured from Schedule 40 PVC pipe or tube, steel, or other material not detrimental to the soil nail steel bar. Wood shall not be used. Centralizers shall be securely attached to the soil nail bar and shall be sized to allow: (a) position the soil nail bar within 25 mm (1 in.) of the center of the drill-hole; (b) tremie pipe insertion to the bottom of the drill-hole; and (c) grout to freely flow up the drill-hole.

1.2.3.2 Grout

Grout shall be a neat cement or sand/cement mixture with a minimum 3-day compressive strength of 10.5 MPa (1,500 psi) and a minimum 28-day compressive strength of 21 MPa (3,000 psi), meeting the requirements of AASHTO T 106/ASTM C 109.

1.2.3.3 Fine Aggregate

Fine aggregate for grout and/or shotcrete shall meet the requirements of AASHTO M 6/ASTM C 33.

1.2.3.4 Portland Cement

Portland cement for grout and/or shotcrete shall meet the requirements of AASHTO M 85/ASTM C 150, Type I, II, III, or V.

PROPOSED COMMENTARY

than the more commonly used lower-grade steels.

Threading may be a continuous spiral, deformed ribbing provided by the bar deformations (continuous thread bars), or may be cut into a reinforcing bar.

C1.2.2.3

The coating at the wall anchorage end of epoxy-coated soil nail bars may be omitted over this length provided for threading the nut against the bearing plate.

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1.2.3.5 Admixtures

Admixtures shall meet the requirements of AASHTO M 194/ASTM C 494. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations.

1.2.3.6 Film Protection

Polyethylene film for moisture loss control shall meet the requirements of AASHTO M 171.

1.2.4 Bearing Plates, Nuts, and Head-Stud Shear Connectors

1.2.4.1 Bearing Plates

Bearing plates shall meet the requirements of AASHTO M 183/ASTM A 36.

1.2.4.2 Nuts

Nuts shall meet the requirements of AASHTO M 291, Grade B, hexagonal, and fitted with beveled washer or spherical seat to provide uniform bearing.

1.2.4.3 Shear Connectors

Shear connectors of the soil nail head may consist of headed-studs, threaded bolts, etc.

1.2.5 Welded-Wire Mesh

Welded wire mesh shall meet the requirements of AASHTO M 55/ASTM A 185 or A 497.

1.2.6 Reinforcing Steel

Reinforcing steel shall meet the requirements of AASHTO M 31/ASTM A 615, Grade 420, deformed.

1.2.7 Geocomposite Sheet Drain

Geocomposite sheet drain shall be manufactured with a drainage core (e.g., geonet) and a drainage geotextile attached to or encapsulating the core. Drainage core shall be manufactured from long-chain synthetic polymers composed of at least 85 percent by mass of polypropylenes, polyester, polyamine, polyvinyl chloride, polyoleofin, or polystyrene and have a minimum compressive strength of 275 kPa (40 psi) when tested in accordance with ASTM D 1621 Procedure A. The drainage core with the geotextile fully encapsulating the core shall have a minimum flow rate of 1 liter per second per meter of width tested in accordance with ASTM D 4716. The test conditions shall be under an applied load of 69 kPa (10 psi) at a gradient of 1.0 after a 100-hour seating period.

PROPOSED COMMENTARY

C1.2.3.5

Admixtures that control bleed, improve flowability, reduce water content, and retard set may be used in the grout subject to review and acceptance by the Engineer. Accelerators are not permitted. Expansive admixtures may only be used in grout used for filling sealed encapsulations.

C1.2.4.1

For nominal resistance of bearing plates refer to Article 5.10.9.7.2 of the AASHTO LRFD Bridge Design Specifications.

C1.2.4.3

See Article 11.3.3.1 of the AASHTO LRFD Bridge Construction Specifications.

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1.2.8 Underdrain and Perforated Pipe

1.2.8.1 Pipe

Underdrain and perforated pipe shall meet the requirements of ASTM 1785 Schedule 40 PVC solid and perforated wall; cell classification 12454-B or 12354-C, wall thickness SDR 35, with solvent weld or elastomeric joints.

1.2.8.2 Fittings

Fittings for underdrain and perforated pipe shall meet the requirements of ASTM D 3034, Cell classification 12454-B or C, wall thickness SDR 35, with solvent or elastomeric joints.

1.2.9 Temporary Shotcrete

All materials, methods, and control procedures for temporary shotcrete shall be submitted to the Owner's Engineer for review and approval.

1.3 CONTRACTOR QUALIFICATIONS

The soil-nailing contractor shall meet the following qualification requirements:

1. Completed at least three permanent soil nail wall projects during the past three years totaling at least 1,000 m^2 (10,000 ft^2) of soil nail wall face area and at least 500 permanent soil nails.
2. Provide a Registered Professional Engineer with experience in the construction of permanent soil nail walls on at least three completed projects over the past three years.
3. Provide on-site supervisors and drill operators with experience installing permanent soil nail walls on at least three projects over the past three years.
4. Submit a brief description of at least three projects, including the owning agency's name, address, and current phone number; location of project; project contract value; and scheduled completion date and completion date for the project.

1.4 SUBMITTALS

1.4.1 Personnel

At least 60 calendar days before starting soil nail work, submit names of the Engineer, on-site supervisors, and drill operators assigned to the project, and a summary of each individual's experience. Only those individuals designated as meeting the qualification requirements shall be used for the project. The Contractor cannot substitute any of these individuals without written approval of the Owner or the Owner's Engineer. The Owner's Engineer shall approve or reject the Contractor qualifications and staff within 15 working days after receipt of the submission. Work shall not be started nor materials ordered until the Contractor's qualifications have been approved by the Owner's Engineer. The Owner's Engineer may suspend the work if the Contractor substitutes unqualified personnel for approved personnel during construction. If work is suspended due to the substitution of unqualified personnel, the Contractor shall be fully liable for all additional costs resulting from the suspension of work, and no adjustment in contract time resulting from the suspension of the work shall be allowed.

1.4.2 Surveys

The Contractor shall be responsible for providing the necessary survey and alignment control during the excavation for each lift, locating drill-holes and verifying limits of the soil nail wall installation.

1.4.3 Construction Plan

At least 30 days before starting soil nail work, the Contractor shall submit a Construction Plan to the Owner's Engineer that includes the following.

1. Project start date and proposed detailed wall construction sequence.
2. Drilling and grouting methods and equipment, including the drill-hole diameter proposed to achieve the specified nominal pullout resistance values shown on the Plans and any variation of these along the wall alignment.
3. Nail grout mix design, including compressive strength test results (per AASHTO T 106/ASTM C 109) supplied by a qualified independent testing lab verifying the specified minimum 3-day and 28-day grout compressive strengths.
4. Nail grout placement procedures and equipment.
5. Temporary shotcrete materials and methods.
6. Soil nail testing methods and equipment setup.
7. Identification number and certified calibration records for each test jack, pressure gauge, dial gauge and load cell to be used. Jack and pressure gauge shall be calibrated as a unit. Calibration records shall include the date tested, the device identification number, and the calibration test results and shall be certified for an accuracy of at least 2 percent of the applied certification loads by a qualified independent testing laboratory within 90 days prior to submittal.
8. Manufacturer Certificates of Compliance for the soil nail ultimate strength, nail bar steel, Portland cement, centralizers, bearing plates, epoxy coating, and encapsulation
9. The Owner's Engineer shall approve or reject the Contractor's Construction Plan within 30 working days after the submission. Approval of the Construction Plan does not relieve the Contractor of his responsibility for the successful completion of the work.

C1.4.3

In a performance type contract, the Contractor must select one of the specialty contractors listed in the documents and shall identify the specialty contractor on his proposal at the bid opening. No substitution will be permitted without written approval of the Owner's Engineer. Substitution after the bid opening will not be grounds for changes in bid prices.

Under a performance type contract, the design of soil nail walls shall be based on geotechnical data and project requirements provided by the Owner including but not limited to soil/rock nominal shear strength parameters, slope and external surcharge loads, seismic design coefficient, type of wall facing, architectural treatment, corrosion protection requirements, easements, and rights-of-way.

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At least 45 days before the planned start of the wall excavation, the Contractor shall submit complete design calculations and working drawings to the Owner's Engineer for review and approval. Include all details, dimensions, quantities, ground profiles and cross-sections necessary to construct the wall. The Contractor shall verify the limits of the wall and ground survey data before preparing the drawings. The working drawings shall be prepared to the (Agency) standards. The Owner's Engineer will approve or reject the Contractor's submittals within 30 calendar days after the receipt of the complete submission. The Contractor shall not begin construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Owner's Engineer.

1.5 STORAGE AND HANDLING

Soil nail bars shall be stored and handled in a manner to avoid damage or corrosion. Soil nail bars exhibiting abrasions, cuts, welds, weld splatter, corrosion, or pitting shall be replaced. Bars exhibiting damage to encapsulation or epoxy coating shall be repaired or replaced at no additional cost. Repaired epoxy coating areas shall have a minimum 0.3-mm (0.012-in.) thick coating.

1.6 EXCAVATION

The height of exposed unsupported final excavation face cut shall not exceed the vertical nail spacing plus the required reinforcing lap or the short-term stand-up height of the ground, whichever is less. Excavation to the final wall excavation line and shotcrete application shall be completed in the same work shift, unless otherwise approved by the Owner's Engineer.

Excavation of the next-lower lift shall not proceed until soil nail installation, reinforced shotcrete placement, attachment of bearing plates and nuts, and nail testing have been completed and accepted in the current lift. Nail grout and shotcrete shall have cured for at least 72 hours or attained at least their specified 3-day compressive strength before excavating the next underlying lift.

PROPOSED COMMENTARY

C1.6

For construction on side hills, a minimum 5-m (15-ft) wide working bench is required for adequate drill rig access.

Shotcrete application may be delayed up to 24 hours if the contractor can demonstrate that the delay will not adversely affect the excavation face stability.

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1.7 SOIL NAIL INSTALLATION

The soil nail length and drill-hole diameter necessary to develop the load capacity and to satisfy the acceptance criteria for the design load required shall be provided, but they shall be not less than the lengths or diameters shown in the Plans.

Drill-holes for the soil nails shall be drilled at the locations, elevations, orientations, and lengths shown on the Plans. The drilling equipment and methods shall be selected to be suitable for the ground conditions and in accordance with the accepted installation methods submitted by the Contractor. The use of drilling mud or other fluids to remove cuttings will not be allowed. If caving ground is encountered, cased drilling methods shall be used to support the sides of the drill-holes. Soil nail bars shall be provided as shown in the Plans.

Centralizers shall be provided and sized to position the soil nail bars to within 25 mm (1 in.) of the center of the drill-hole. Centralizers shall be positioned as shown on the Plans so that their maximum center-to-center spacing does not exceed 2.5 m (8.2 ft) and shall be located to within 0.5 m (1.5 ft) from the top and bottom of the drill-hole.

1.8 GROUTING

The drill-hole shall be grouted after installation of the soil nail bar and within 2 hours of completion of drilling. The grout shall be injected at the lowest point of each drill-hole through a grout tube, casing, hollow-stem auger, or drill rods. The outlet end of the conduit shall deliver grout below the surface of the grout as the conduit is withdrawn to prevent the creation of voids. The drill-hole shall be filled in one continuous operation. Cold joints in the grout column shall not be allowed except at the top of the test bond length of proof tested production nails.

Grout shall be tested in accordance with AASHTO T 106/ASTM C 109 at a frequency of one test per mix design and a minimum of one test for every 40 m^3 (52 cy) of grout placed. Grout cube test results shall be provided to the Owner's Engineer within 24 hours of testing.

1.9 SOIL NAIL TESTING

1.9.1 Tests

The Contractor shall perform both verification and proof testing of designated test soil nails. Verification tests on sacrificial test nails shall be conducted at locations shown on the Plans. Proof tests on production nails shall be conducted at locations selected by the Owner's Engineer. Testing of any nail shall not be performed until the nail grout and shotcrete facing have cured for at least 72 hours or attained at least their specified 3-day compressive strength.

1.9.2 Equipment

Testing equipment shall include 2 dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a

PROPOSED COMMENTARY

C1.7

The use of self-drilling soil nail bars (also known as hollow, self-grouting or pressure-grouted nail bars) are not allowed for permanent construction, unless approved by the Owner.

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reaction frame. The pressure gauge shall be graduated in 500 kPa (75 psi) increments or less. Nail head movement shall be measured with a minimum of 2 dial gauges capable of measuring to 0.025 mm (0.001 in.).

1.10 VERIFICATION TESTING

Verification testing shall be conducted prior to installation of production soil nails on sacrificial soil nails to confirm the appropriateness of the Contractor's drilling and installation methods, and verify the required nail pullout resistance.

1.10.1 Methods

The verification tests must be conducted on nails of the same design and constructed with the same construction methods to be used on production nails for meaningful results.

Verification test nails shall have both bonded and unbonded lengths. The nail bar shall not be grouted along the unbonded length. The unbonded length of the test nails shall be at least 1 m (3 ft). The bonded length of the soil nail during verification tests, $L_{B\,VT}$, shall be the smaller value of the following range:

$$L_{B\,VT} = \text{greater of} \begin{cases} 3\,\text{m}\ (10\,\text{ft}) \\ L_{B\,VT\,max} \end{cases}$$

The maximum length, $L_{B\,VT\,max}$, is defined as:

$$L_{B\,VT\,max} = \frac{C_{RT} \times A_t \times f_Y \times \phi_{T-VT}}{r_{PO} \times \phi_{PO}}$$

where:

- C_{RT} = reduction coefficient;
- A_t = cross-sectional area of soil nail bar;
- f_Y = nominal yield resistance of soil nail bar;
- r_{PO} = nominal pullout resistance (per unit length) of soil nail, as specified herein or in Plans;
- $\phi_{T\text{-}VT}$ = resistance factor for soil nail in tension in verification tests; and
- ϕ_{PO} = resistance factor for soil nail pullout.

The maximum bonded length shall be preferably based on production nail maximum bar grade. Larger bar sizes shall be provided at no additional cost, if required, to meet the 3-m (10-ft) minimum test bonded length requirement.

The Design Load during the verification test, *DL*, shall be calculated based on as-built bonded lengths, as follows:

$$DL = L_{B\,VT} \times r_{PO} \times \phi_{PO}$$

1.10.2 Schedule

Verification tests shall be conducted by incrementally loading the verification test nails to failure or a maximum test load of

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C1.10.1

At least two verification tests should be conducted in each soil strata in which it is anticipated that nail bond zone will be grouted.

Where possible, verification tests should be conducted to failure to establish a maximum resistance with respect to pullout.

The maximum length for verification tests, $L_{B\,VT\,max}$, is selected so that the nail load does not exceed 90 percent of factored nominal tensile resistance of the soil nail bar during the verification test.

Use C_{RT} = 0.9 for 420 and 520 MPa (Grade 60 and 75) bars. If 1,035 MPa (Grade 150) soil nail bars are allowed in the job, use C_{RT} = 0.8.

In verifications tests, select $\phi_{T\text{-}VT}$ = 0.4 or, preferably, 0.33. Select ϕ_{PO} based on Table 11.5.6-1, "Resistance Factors," of AAHTO LRFD Bridge Design Specifications. For preliminary values, use ϕ_{PO} = 0.5.

The selection of $\phi_{T\text{-}VT}$ during verification tests should be consistent with the maximum test load that has been selected for the verification test. The maximum load depends on the selected ϕ_{PO} and must be selected such that the test bars are not overstressed during the test.

C1.10.2

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200 percent of the *DL* in accordance with the following loading schedule. The Contractor shall record soil nail movements at each load increment.

Verification Test Loading Schedule

Load	Hold Time
0.05 DL max. (= alignment load, AL)	1 minute
0.25 DL	10 minutes
0.50 DL	10 minutes
0.75 DL	10 minutes
1.00 DL	10 minutes
1.25 DL	10 minutes
1.50 DL (Creep Test)	60 minutes
1.75 DL	10 minutes
2.00 DL	10 minutes
0.05 DL max. (AL)	1 minute (record permanent set)

Load levels beyond 200 percent of *DL* are optional.

Dial gauges shall be set to "zero" after the alignment load has been applied. Following application of the maximum load ($3.0 \times DL$), the load shall be reduced to the alignment load ($0.05 \times DL$ maximum) and the permanent set shall be recorded.

Each load increment shall be held for at least 10 minutes. The verification test nail shall be monitored for creep at the $1.50 \times DL$ load increment. Nail movements shall be measured and recorded during the creep portion of the test in increments of 1, 2, 3, 5, 6, 10, 20, 30, 50, and 60 minute(s). The load shall be maintained during the creep test to within 2 percent of the intended load by use of a load cell.

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In soils that are susceptible to creep, extended creep tests beyond the tests required by these specifications should be conducted based on PTI (2005) methods. The alignment load, AL, should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the DL.

In projects, for which there is no local experience in soil nailing or the degree of uncertainty in the pre-selected nominal pullout resistance (per unit length) values is significant, the Owner or the Owner's Engineer may require the Contractor to perform verification tests up to a maximum test load of 300 percent of the DL in the tests. For these situations, the loading shall be performed according to the schedule below:

Additional Verification Test Loading Schedule

Load	Hold Time
From AL to 2.00 DL (or Failure)	as shown previously
2.50 DL (optional)	10 minutes
3.0 DL (optional)	10 minutes
0.05 DL max. (AL)	1 minute (record permanent set)

1.11 PROOF TESTING

Successful proof testing shall be demonstrated on at least 5 percent of production soil nails in each nail row or a minimum of one per row. The Owner's Engineer shall determine the locations and number of proof tests prior to nail installation in each row.

C1.11

1.11.1 Methods

Production proof test nails shall have both bonded and temporary unbonded lengths. The unbonded length of the test nail shall be at least 1 m (3 ft). The bonded length of the soil nail during proof production tests, $L_{B\,PT}$, shall be at least 3 m (10 ft) but not longer than a maximum length, $L_{B\,PT\,max}$. Therefore, the following requirements shall be met:

$$L_{B\,PT} = \text{greater of} \begin{cases} 3\,\text{m}\ (10\,\text{ft}) \\ L_{B\,PT\,max} \end{cases}$$

C1.11.1

The unbonded length is temporary because this length is grouted after the proof test is completed.

The maximum length for proof tests, $L_{B\,PT\,max}$, is selected so that the nail load does not exceed 90 percent of factored nominal tensile resistance of the soil nail bar during the proof test.

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The maximum length $L_{B\,PT\,max}$ is defined as:

$$L_{B\,PT\,max} = \frac{C_{RT} \times A_t \times f_Y \times \phi_{T-PT}}{r_{PO} \times \phi_{PO}}$$

where:

C_{RT} = reduction coefficient;
A_t = cross-sectional area of soil nail bar;
f_Y = nominal yield resistance of soil nail bar;
r_{PO} = nominal pullout resistance (per unit length) of soil nail, as specified herein or in Plans;
$\phi_{T\text{-}PT}$ = resistance factor for soil nail in tension in proof tests; and
ϕ_{PO} = resistance factor for soil nail pullout.

The Design Load during the verification test, *DL*, shall be calculated based on as-built bonded lengths, as follows:

$$DL = L_{B\,PT} \times r_{PO} \times \phi_{PO}$$

1.11.2 Schedule

Proof tests shall be conducted by incrementally loading the proof test nail to 150 percent of *DL* in accordance with the following loading schedule. Soil nail movements shall be recorded at each load increment.

Proof Test Loading Schedule

Load	Hold Time
0.05 DL max. (*AL*)	Until Movement Stabilizes
0.25 *DL*	Until Movement Stabilizes
0.50 *DL*	Until Movement Stabilizes
0.75 *DL*	Until Movement Stabilizes
1.00 *DL*	Until Movement Stabilizes
1.25 *DL*	Until Movement Stabilizes
1.50 *DL* (Max. Test Load)	Creep Test (see below)

Dial gauges shall be set to “zero” after the alignment load has been applied.

The creep period shall start as soon as the maximum test load ($1.50 \times DL$) is applied and the nail movement shall be measured and recorded at 1, 2, 3, 5, 6, and 10 minute(s). Where the nail movement between 1 minute and 10 minutes exceeds 1 mm (0.04 in.), the maximum test load shall be maintained for an additional 50 minutes and nail movements shall be recorded at 20, 30, 50, and 60 minutes. All load increments shall be maintained to within 5 percent of the intended load.

PROPOSED COMMENTARY

Use C_{RT} = 0.9 for 420 and 520 MPa (Grade 60 and 75) bars. If 1,035 MPa (Grade 150) soil nail bars are allowed in the job, use C_{RT} = 0.8.

In proof tests, select $\phi_{T\text{-}PT}$ = 0.67. Select ϕ_{PO} based on Table 11.5.6-1, “Resistance Factors,” of AAHTO LRFD Bridge Design Specifications. For preliminary values, use ϕ_{PO} = 0.5.

The selection of $\phi_{T\text{-}PT}$ during proof tests should be consistent with the maximum test load that has been selected for these tests. The maximum load depends on the selected ϕ_{PO} and must be selected such that the test bars are not overstressed during the test. This is usually 1.5 DTL. Avoiding bar overstressing during the test allows using the test bars as production bars after the test.

Production proof test nails shorter than 4 m (12 ft) may be constructed with less than the minimum 3-m (10-ft) bond length.

C1.11.2

The alignment load, *AL*, should be the minimum load required to align the testing apparatus and should not exceed 5 percent of the DL.

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1.12 ACCEPTANCE CRITERIA OF TEST SOIL NAILS

A test nail shall be considered acceptable when all of the following criteria are met:

1. For verification tests, the total creep movement is less than 2 mm (0.08 in.) between the 6- and 60-minute readings, and the creep rate is linear or decreasing throughout the creep test load hold period.

2. For proof tests, the total creep movement is less than 1 mm (0.04 in.) during the 10-minute readings or the total creep movement is less than 2 mm (0.08 in.) during the 60-minute readings, and the creep rate is linear or decreasing throughout the creep test load hold period.

3. For verification and proof tests, the total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length of the test nail.

4. A pullout limit state does not occur at $2.0 \times DL$ under verification testing and $1.5 \times DL$ test load under proof testing. Pullout limit state is defined at a load level at which the test load cannot be further increased while there is continued pullout movement of the test nail. The load at the pullout limit state shall be recorded as part of the test data.

5. Maintaining stability of the temporary unbonded test length for subsequent grouting is the Contractor's responsibility. If the unbonded test length of production proof test nails cannot be satisfactorily grouted after testing; the proof test nail shall become sacrificial and shall be replaced with an additional production nail installed at no additional cost to the Owner.

1.13 REJECTION OF TEST SOIL NAILS

1.13.1 Verification Test Soil Nails

The Owner's Engineer will evaluate the results of each verification test. Installation methods that do not satisfy the nail testing requirements shall be rejected. The Contractor shall propose alternative methods for review by the Owner's Engineer and shall install replacement verification test nails. Replacement test nails shall be installed and tested at no additional cost.

1.13.2 Proof Test Soil Nails

For proof test nails, the Owner's Engineer may require the Contractor to replace some or all of the installed production nails between a failed proof test soil nail and the adjacent passing proof test nail. Alternatively, the Owner's Engineer may require

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the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient nominal pullout resistance. Installation and testing of additional proof test nails or installation of additional or modified nails as a result of proof test nail failure(s) shall be at no additional cost.

1.14 WALL DRAINAGE NETWORK

All elements of the soil nail wall drainage network shall be installed and secured as shown on the Plans. The drainage network shall consist of geocomposite drain strips, PVC connection pipes, soil nail wall footing drains, and weepholes, as shown on the Plans. Exclusive of the wall footing drains, all elements of the drainage network shall be installed prior to shotcreting.

1.14.1 Geocomposite Drain Strips

Geocomposite drain strips shall be centered between the columns of soil nails, as shown on the Plans. Drain strips shall be at least 300 mm (12 in.) wide and placed with the geotextile side against the ground. Strips shall be secured to the excavation face. Contamination of the geotextile with shotcrete shall be prevented. Drain strips shall be vertically continuous. Splices shall be made with a 300 mm (12 in.) minimum overlap such that the flow of water is not impeded. Drain plate and connector pipe shall be installed at the base of each strip. Damage to the geocomposite drain strip shall be repaired so that water flow is not interrupted.

1.14.2 Footing Drains

Footing drains shall be installed at the bottom of the wall, as shown on the Plans. The drainage geotextile shall envelope the footing drain aggregate and pipe and shall conform to the dimensions of the trench. The drainage geotextile shall overlap on top of the drainage aggregate as shown on the Plans. Damaged or defective drainage geotextile shall be repaired or replaced.

1.15 SHOTCRETE FACING

Shotcrete facing and permanent shotcrete facing shall be provided as required. Where shotcrete is used to complete the top ungrouted zone of the soil nail drill-hole near the face, the nozzle shall be positioned into the mouth of the drill-hole to completely fill the void.

1.15.1 Final Face Finish

Shotcrete finish shall be either an undisturbed gun finish as applied from the nozzle or a rod, broom, wood float, rubber float, steel trowel, or rough screeded finish as shown on the Plans.

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1.15.2 Attachment of Nail Head Bearing Plate and Nut

A bearing plate, washers, and nut shall be attached to each nail head as shown on the Plans. While the shotcrete construction facing is still plastic and before its initial set, the plate shall be uniformly seated on the shotcrete by hand-wrench tightening the nut. Where uniform contact between the plate and the shotcrete cannot be provided, the plate shall be set in a bed of grout. After grout has set for 24 hours, hand-wrench tighten the nut. The bearing plates with headed studs shall be located within the tolerances shown on the Plans.

1.15.3 Shotcrete Facing Tolerances

Construction tolerances for the shotcrete facing from plan location and plan dimensions shall be as follows:

Horizontal location of welded wire mesh; reinforcing bars, and headed studs: 10 mm (0.4 in.)

Location of headed-studs on bearing plate: 6 mm (1/4 in.)

Spacing between reinforcing bars: 25 mm (1 in.)

Reinforcing lap: 25mm (1 in.)

Complete thickness of shotcrete:
- If troweled or screeded: 15 mm (0.6 in.)
- If left as shot: 30 mm (1.2 in.)

Planeness of finish face surface-gap under 3-m (10-ft) straightedge:
- If troweled or screeded: 15 mm (0.6 in.)
- If left as shot: 30 mm (1.2 in.)

Nail head bearing plate deviation from parallel to wall face: 10 degrees

1.16 REINFORCING STEEL

The Contractor shall submit all order lists and reinforcement bending diagrams to the Owner's Engineer and shall fabricate reinforcing steel; ship and protect material; and place, fasten, and splice reinforcing steel as required by the Plans.

1.17 STRUCTURAL CONCRETE

The Contractor shall design the concrete mix; store, handle, batch, and mix material and deliver concrete; provide quality control; and construct concrete facing.

1.18 ARCHITECTURAL SURFACE FINISHES

Textured form liners shall be furnished, form liners installed, and a surface finish (color/stain application) applied that will duplicate the architectural surface finish shown on the Plans.

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The Contractor shall submit detailed drawings of the form liner for approval by the Owner's Engineer at least 7 days before form liner work begins. Before production work begins, a 1-m (3-ft) high by 0.5-m (1.5-ft) wide by 3-m (10-ft) long test panel shall be constructed on site using the same forming methods, procedures, form liner, texture configuration, expansion joint, concrete mixture and color/stain application proposed for the production work.

1.19 BACKFILLING BEHIND WALL FACING UPPER CANTILEVER

If backfilling is required behind an extension of the wall facing at the top of a soil nail wall, compaction of the soil backfill within 1 m (3 ft) shall be limited to light mechanical tampers.

Backfill shall be relatively free-draining granular soil meeting the requirement of Article 7.3.5 (of the AASHTO LRFD Bridge Construction Specifications).

1.20 ACCEPTANCE

Material for soil nail retaining walls will be accepted based on the manufacturer production certification or from production records. Construction of soil nail retaining walls will be accepted based on visual inspection and the examination of relevant production testing records by the Owner's Engineer.

2.0 MEASUREMENT AND PAYMENT

2.1 SOIL NAILS

Production soil nails shall be measured by the linear meter (or foot). The length to be paid will be the length measured along the soil nail bar centerline from the back face of shotcrete to the bottom tip end of the 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.

Verification test nails shall be measured by each test meeting the acceptance criteria of Article 1.10. Failed verification test nails or additional verification test nails installed to verify alternative nail installation methods proposed by the Contractor will not be measured.

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2.2 STRUCTURE EXCAVATION

Structure excavation for the soil nail wall shall be measured as the theoretical plan volume in cubic meters (cubic feet) within the structure excavation limits shown on the Plans. This will be the excavation volume within the zone measured from top to bottom of shotcrete wall facing and extending out 2 m (6 ft) horizontally in front of the plan wall final excavation line. Additional excavation beyond the Plan wall final excavation line resulting from irregularities in the cut face, excavation overbreak or inadvertent excavation will not be measured. No measurement will be made for using temporary stabilizing berms. General roadway excavation, including hauling, will not be a separate wall pay item but will be measured and paid as part of the general roadway excavation.

2.3 WALL FACE

The wall face of soil nail walls shall be measured by the square meter (square foot) of wall face. Measurement will be made on the vertical plane of front face accepted in the final work. No measurement or payment will be made for additional shotcrete or CIP concrete needed to fill voids created by irregularities in the cut face, excavation overbreak or inadvertent excavation beyond the Plan final wall face excavation line, or failure to construct the facing to the specified line and grade and tolerances. The final pay quantity shall include all structural shotcrete, admixtures, reinforcement, welded wire mesh, wire holding devices, wall drainage materials, bearing plates and nuts, test panels and all sampling, testing and reporting required by the Plans and this Specification. The final pay quantity shall be the design quantity increased or decreased by any changes authorized by the Owner's Engineer.

2.4 PAYMENT

The accepted quantities, measured as provided in Articles 2.1, 2.2 and 2.3, will be paid for at the contract unit price per unit of measurement for the pay items listed below that are shown on the bid schedule. Payment will be full compensation for the work prescribed in this section. Payment will be made under:

2.4.1 Pay Items

Pay Item	Pay Unit
Permanent Soil Nails. No. _ Bar (Grade _)	Linear meter (or linear foot)
Verification Test Nails	Each
Structure Excavation-Soil Nail Wall	Cubic meter (or cubic foot)
Soil Nail Wall	Square meter (or square foot)

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C2.4

Under a performance type contract, payment may be made on a lump-sum basis to include all materials, labor and design costs.

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3 CORROSION PROTECTION

Soil nails and soil nail head components shall be protected against corrosion consistent with the ground and groundwater conditions at the site. The level and extent of corrosion protection shall be a function of the ground environment and the potential consequences of a soil nail failure.

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Corrosion protection shall be applied in accordance with the provisions of AASHTO LRFD Bridge Construction Specifications, Section 6, "Ground Anchors."

A full discussion on corrosion of metallic components and a methodology that assists in selecting the appropriate level of corrosion protection of soil nail walls is presented in Lazarte et al. (2003).

REFERENCES

Lazarte, C.A., V. Elias, R.D. Espinoza, and P.J. Sabatini (2003). "Soil Nail Walls," Geothechnical Engineering Circular No. 7, FHWAO-IF-03-017, Federal Highway Administration, Washington, D.C., 305p.

PTI (2005). "Recommendations for Prestressed Rock and Soil Anchors," 4th ed. Post-Tensioning Institute, Phoenix, Arizona.

APPENDIX C

Soil Nail Test Pullout Resistance Database

Introduction

The pullout resistance database is presented in this appendix. The information consulted to build the pullout resistance database included the following:

1. Soil Nail Test Results
   - Load applied to the soil nail (P);
   - Total measured elongation (Δ_{tot});
   - Observations made during test (e.g., premature failure, proximity to failure); and
   - Design Load (DL).
2. Soil Nail Data
   - Diameter of drill-hole (D_{DH});
   - Nail total length and bonded length (L_{tot}, L_B); and
   - Nail bar diameter (D_B).
3. Geotechnical Data
   - Site location;
   - Soil type description;
   - Geotechnical reports including boring logs;
   - Blow count (N) or other field test results;
   - Groundwater table location;
   - Plans with SNW and boring locations;
   - Description of nail installation method; and
   - Drawings and specifications of soil nails.

Sources of Soil Nail Load-Test Data

Soil nail load-test results were obtained from numerous sources including: the project team's database; company members of ADSC: The International Association of Foundation Drilling; soil nail specialty contractors; state departments of transportation; and published data. A summary of the available data organized according to the material type, number of projects, and number of tests used is presented in Table C-1.

The soil nail load-test data was derived from proof load and verification tests. Over 95 percent of the data considered was derived from proof tests. For most cases, the maximum load applied to the nails was 150 percent of DL or less. An unexpected pullout failure, occurring before the intended load test level was achieved, was observed in only two proof tests. No unexpected pullout failure was observed in the verification tests before the intended load test level was achieved. The nominal bond resistance was established for the selected load tests using methods that are presented in the following subsection.

Limitations noted in some of the tests listed in Table C-1 included inadequate or missing information related to (i) project features (e.g., tested nail not identified in plan or elevation views or correlated to a soil condition); (ii) geotechnical data (e.g., no geotechnical report, no boring logs, inadequate soil description); (iii) characteristics of test bars (e.g., missing information on D_{DH}, bonded and unbonded lengths, bar diameter); and (iv) installation technique (e.g., information on drilling, casing, or grout strength characteristics were missing). When items listed in (i) through (iii) were missing, tests were excluded from the database.

Additional results of soil nail testing may be used to increase design reliability. In theory, conducting more verification (possibly testing nails to higher loads) should produce a higher degree of reliability in the design.

Interpretation of Results

The database was organized according to soil type (i.e., predominantly sand, clay, and weathered rock). The number of cases pertaining to sandy/gravelly soils was small (i.e., only eight cases); therefore, these data points were combined with those pertaining to sandy soils. In all cases, the bond stress was calculated based on the load (usually expressed in tons), bonded length, and drill-hole diameter. Alternatively, the pullout load per unit length, Q, (also previously referred to as load transfer, r_{PO}) was calculated. The elastic elongation of the unbonded bar section was calculated and deducted

Table C-1. Summary of available soil nail tests considered for database.

Predominant Material Type	Number of Projects	Number of Available Load Tests	Number of Used Load Tests
Sand	10	168	74
Sand/Gravel	3	31	8
Clay	8	92	45
Weathered Rock	5	67	26
Other	6	88	0
Total	**32**	**446**	**153**

from the total elongation to calculate the net elongation of the bonded length. The net elongation was then divided by the bonded length and the result expressed as a percentage. Load test results were plotted as mobilized bond stress, q, and expressed as a function of the total elongation, net elongation, or net elongation/bonded length (defined as the net elongation divided by the bonded length, and expressed as a percentage). The data was plotted against the total, net, or normalized net elongations.

On average, the curves tended to flatten and exhibited the onset of ultimate conditions for a normalized net elongation of $\epsilon_B = 0.1$ to 0.5 percent (sands), 0.01 to 0.05 percent (clays), and greater than 0.5 percent (gravel and weathered rock). These trends are consistent with typical soil-strain response of these soil types. The data for sand tended to exhibit less variability when the load data was plotted as a function of the normalized net elongation.

The interpretation of load-test results included the estimation of an "ultimate" nail load (equivalently, nominal bond resistance). Several procedures were used to estimate the nominal bond resistance, including: (a) field observations of "near" or imminent failure; (b) evaluation of test curves; (c) analyses of creep test data; and (d) analyses of loads using a maximum deflection criteria. The adequacy of each of these approaches is discussed below.

Field Observations

The success of this approach was limited because the great majority of tests were proof tests, which were loaded up to 150 percent of *DL*, and did not exhibit imminent failure. Contractors' notes during load tests, if available, were reviewed.

Evaluation of Test Curves

This approach was helpful to estimate the elongation at which the test curve flattened and to establish an ultimate load. Observations provided better estimates of an ultimate condition when the soil nail test was performed in clays and clayey sands, when compared to tests in gravel, dense sands, and weathered rock. In the latter cases, soil nails typically required a significant deformation to mobilize their resistance.

Analysis of Creep Test Data

The usefulness of this approach was limited because none of the tests showed an excessive deformation rate that indicated an imminent load failure (or even a nail rejection in the U.S. practice). In French soil-nailing practice (Clouterre, 2002), deformation rates observed during creep tests at increasing loads are analyzed to estimate a "yield" pullout load. However, the amount of creep data that was available for this research project was insufficient for the Clouterre approach to be used.

Analysis of Load-Elongation Curves

Several criteria were used to analyze the load curves and establish an "ultimate" load. Techniques similar to those used to estimate the ultimate compression and tension loads in deep foundations were considered. Some of the techniques considered included the well-known Davisson (1972) method (graphical estimation of an ultimate load from a load-settlement curve), the De Beer (1967and 1968) method (graphical estimation of ultimate loads based on the graphical representation of the logarithms of loads and settlements), and the Brinch-Hansen (1963) method (graphical estimation of ultimate loads based on a parabolic approximation of the load-settlement curve). Only in a few cases were these methods helpful to identify clearly the ultimate pullout resistance.

Methods commonly used in tension tests of piles were also considered to estimate the ultimate pullout load. In these methods (e.g., Hirany and Kulhawy, 2002; Koutsoftas, 2000), the ultimate load is achieved when the soil/nail interface shows 0.4 to 0.5 in. of movement.

When the ultimate pullout resistance was not evident from the methods mentioned in items (a) through (d), the maximum load was considered to be achieved when the net is at least 1 in. This criterion is consistent with the practice adopted by some SNW contractors to stop a load test.

Measured and Predicted Values of Pullout Resistance

Measured values of pullout resistance were obtained based on the various criteria described above and are presented for each soil type.

For each of these soil types, the predicted pullout resistance was defined as 200 percent of the design load as is common in U.S. practice (see Byrne et al., 1998 and Lazarte et al., 2003). These estimations are also provided in Tables C-2 through C-4 for each soil type. Note that the predicted pullout resistance values are not directly related to any specific design equation but, instead, represent the values selected by design engineers possibly based on a combination of recommended ranges (e.g., Elias and Juran, 1991) and values based on local experience. Values predicted using correlations with PMT or SPT values were not used because PMT data was unavailable and because SPT information was incomplete or not directly associated to the soil nail test location.

The mean, standard deviation, and COV of the bias were obtained for the lognormal distribution for each of the soil types. In establishing these parameters, the lognormal distribution was adjusted to match the lognormal distribution with the lower tail of the resistance bias data points. The statistical parameters for these curves are summarized in Table C-5. These factors are to perform the calibration of the pullout resistance factors.

Table C-2. Summary of estimation and prediction of nominal bond resistance—sands.

No.	Type of Natural Material	Soil/RockType	Project Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B (in.)	Design Load, DL (kip)	Test Design Load, DL (kip)	Estimated Pullout Resistance, Q (kip/ft)	Predicted Resistance (kips)	Measured Resistance (kips)
1	Cohesionless	Sand	Milledgeville,GA	4	12	3	NA	1	24	24.0	2.0	48	29
2	Cohesionless	Sand	Milledgeville,GA	1	12	3	NA	1.25	24	24.0	2.0	48	31
3	Cohesionless	Sand	Milledgeville,GA	6	12	3	NA	1	24	24.0	2.0	48	33
4	Cohesionless	Sand	Milledgeville,GA	Proof #1	5.2	9.3	6	0.75	9.8	9.8	1.88	19.6	14.3
5	Cohesionless	Sandy Silt	San Diego, CA	11	11	20	6	1.24	22	22	2.0	44	33
6	Cohesionless	Sand	Milledgeville,GA	H-1-7	9	11	6	1	13.5	13.5	1.5	27	20.5
7	Cohesionless	Sandy Silt	San Diego, CA	8	11	18.5	6	1.24	22	22	2.0	44	34
8	Cohesionless	Sandy Silt	San Diego, CA	12	11	20	6	1.24	22	22	2.0	44	34.5
9	Cohesionless	Sandy Silt	San Diego, CA	9	11	20	6	1.24	22	22	2.0	44	35
10	Cohesionless	Sandy Silt	San Diego, CA	5	11.4	20	6	1.24	22.8	22.8	2.0	45.6	38
11	Cohesionless	Sand	Milledgeville,GA	2	12	3	NA	1	24	24.0	2.0	48	40
12	Cohesionless	Sand	Milledgeville,GA	H-1-5	7.5	7.5	6	1	11.3	11.3	2	22.6	19.2
13	Cohesionless	Sand	Milledgeville,GA	H-1-4	8	7	6	1	12	12	1.5	24	20.4
14	Cohesionless	Sand	Milledgeville,GA	5	12	3	NA	1	24	24.0	2.0	48	41
15	Cohesionless	Sandy Silt	San Diego, CA	7	11	20	6	1.24	22	22	2.0	44	38
16	Cohesionless	Sand	Milledgeville,GA	H-1-2	5	10	6	1	7.5	7.5	1.5	15	13
17	Cohesionless	Sandy Silt	San Diego, CA	16	11.4	19	6	1.24	22.8	22.8	2.0	45.6	40
18	Cohesionless	Sandy Silt	San Diego, CA	21	11	20	6	1.24	22	22	2.0	44	39
19	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-2	16	4	3.5	0.875	15.8	25.28	1.6	50.56	45
20	Cohesionless	Sandy Silt	San Diego, CA	20	11.4	20	6	1.24	22.8	22.8	2.0	45.6	41
21	Cohesionless	Sand	Milledgeville,GA	H-1-1	10	15	6	1	15	15	1.5	30	27
22	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-1	14	6	3.5	0.875	15.8	22.12	1.6	44.24	40
23	Cohesionless	Sandy Silt	San Diego, CA	18	11.4	19	6	1.24	22.8	22.8	2.0	45.6	41.5
24	Cohesionless	Sand	Roseville, CA	D-2-1	10	12	6	0.875	18.1	18.1	1.8	36.2	33
25	Cohesionless	Sandy Silt	San Diego, CA	19	11.4	20	6	1.24	22.8	22.8	2.0	45.6	42
26	Cohesionless	Sandy Silt	San Diego, CA	17	11.4	19	6	1.24	22.8	22.8	2.0	45.6	42.5
27	Cohesionless	Sand	Milledgeville,GA	3	12	3	NA	1	24	24.0	2.0	48	45
28	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-3	10	10	3	1.181	29.23	29.23	2.9	58.46	55
29	Cohesionless	Sand	Milledgeville,GA	H-2-1	5.2	9.3	6	0.75	9.8	7.8	1.5	15.6	14.8

Table C-2. (Continued).

No.	Type of Natural Material	Soil/RockType	Project Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B (in.)	Design Load, DL (kip)	Test Design Load, DL (kip)	Estimated Pullout Resistance, Q (kip/ft)	Predicted Resistance (kips)	Measured Resistance (kips)
30	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-3	10	10	3.5	0.875	15.8	15.8	1.6	31.6	30
31	Cohesionless	Sandy Silt	San Diego, CA	15	11	19	6	1.13	27.5	27.5	2.5	55	53
32	Cohesionless	Sandy Silt	San Diego, CA	10	11	14	6	1.00	22	22	2.0	44	43
33	Cohesionless	Sandy Silt	San Diego, CA	14	11	19	6	1.13	27.5	27.5	2.5	55	54
34	Cohesionless	Sand	Milledgeville,GA	H-1-3	7	13	6	1	10.5	10.5	1.5	21	21
35	Cohesionless	Sandy Silt	San Diego, CA	6	11.4	20	6	1.24	22.8	22.8	2.0	45.6	46
36	Cohesionless	Sand	Roseville, CA	D-2-2	10	12	6	0.875	18.1	18.1	1.8	36.2	37
37	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-2	10	10	3	1.181	29.23	29.23	2.9	58.46	60
38	Cohesionless	Sand	Milledgeville,GA	7	12	3	NA	1	24	24.0	2.0	48	50
39	Cohesionless	Sandy Silt	San Diego, CA	13	11	19	6	1.00	22	22	2.0	44	46
40	Cohesionless	Sand	Milledgeville,GA	H-1-6	4	16	6	1	6	6	1.5	12	13
41	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-4	10	10	6	1	15.8	15.8	1.6	31.6	35
42	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-6	10	10	3	1.181	29.23	29.23	2.9	58.46	65
43	Cohesionless	Sandy Silt	San Diego, CA	2	11.5	18.5	6	1.24	23	23	2.0	46	52
44	Cohesionless	Sandy Silt	San Diego, CA	22	11	6	6	1.24	22	22	2.0	44	50
45	Cohesionless	Sandy Silt	San Diego, CA	4	10.5	19.5	6	1.24	21	21	2.0	42	48
46	Cohesionless	Sand	Cobb, GA	D-3-20	14.4	8.5	8	1.41	36.2	25.92	1.8	51.84	60
47	Cohesionless	Sandy Silt	San Diego, CA	23	11	6	6	1.24	22	22	2.0	44	51
48	Cohesionless	Sandy Silt	San Diego, CA	3	10.5	19.5	6	1.24	21	21	2.0	42	49
49	Cohesionless	Sandy Silt	San Diego, CA	1	10.5	18	6	1.24	21	21	2.0	42	50
50	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-6	10	24	6	1	15.8	15.8	1.6	31.6	38
51	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-8	10	25	6	1	15.8	15.8	1.6	31.6	39
52	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-8	10	10	2.5	1.181	20	20	2.0	40	50
53	Cohesionless	Sand	Cobb County, GA	D-3-21	14.3	8.5	8	1.41	36.2	25.74	1.8	51.48	66
54	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-1	10	10	3	1.181	29.23	29.23	2.9	58.46	77
55	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-4	10	10	3	1.181	29.23	29.23	2.9	58.46	79
56	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-5	10	10	3	1.181	29.23	29.23	2.9	58.46	80
57	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-5	10	10	6	1	15.8	15.8	1.6	31.6	44
58	Cohesionless	Gravelly Sand	Squaw Valley, CA	D-4-7	10	10	2.5	1.181	20	20	2.0	40	57

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Table C-2. (Continued).

No.	Type of Natural Material	Soil/RockType	Project Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B (in.)	Design Load, DL (kip)	Test Design Load, DL (kip)	Estimated Pullout Resistance, Q (kip/ft)	Predicted Resistance (kips)	Measured Resistance (kips)
59	Cohesionless	Clayey Sand	San Luis Obispo, CA	D-1-7	10	10	3.5	0.875	15.8	15.8	1.6	31.6	46
60	Cohesionless	Sand	Cobb County, GA	D-3-27	15.7	7.8	8	1.41	36.2	28.26	1.8	56.52	85
61	Cohesionless	Sand	Cobb County, GA	D-3-30	17.4	6	8	1.41	36.2	31.32	1.8	62.64	95
62	Cohesionless	Sand	Cobb County, GA	D-3-26	14.75	8.5	8	1.41	36.2	26.55	1.8	53.1	83
63	Cohesionless	Sand	Cobb County, GA	D-3-17	14.8	15.2	8	1.41	36.2	26.64	1.8	53.28	84
64	Cohesionless	Sand	Cobb County, GA	D-3-16	14.5	12.5	8	1.41	36.2	26.1	1.8	52.2	84
65	Cohesionless	Sand	Cobb County, GA	D-3-10	15.9	7.3	8	1.41	36.2	28.62	1.8	57.24	94
66	Cohesionless	Sand	Cobb County, GA	D-3-28	14.2	8.5	8	1.41	36.2	25.56	1.8	51.12	86
67	Cohesionless	Sand	Cobb County, GA	D-3-22	11	4	8	1.41	36.2	19.8	1.8	39.6	68
68	Cohesionless	Sand	Cobb County, GA	D-3-18	14	8.5	8	1.41	36.2	25.2	1.8	50.4	89
69	Cohesionless	Sand	Cobb County, GA	D-3-24	12.3	4.5	8	1.41	36.2	22.14	1.8	44.28	79
70	Cohesionless	Sand	Cobb County, GA	D-3-19	15.3	9.7	8	1.41	36.2	27.54	1.8	55.08	100
71	Cohesionless	Sand	Cobb County, GA	D-3-9	15	7.5	8	1.41	36.2	27	1.8	54	100
72	Cohesionless	Sand	Cobb County, GA	D-3-23	14.8	8	8	1.41	36.2	26.64	1.8	53.28	100
73	Cohesionless	Sand	Cobb County, GA	D-3-33	11	4	8	1.41	36.2	19.8	1.8	39.6	75
74	Cohesionless	Sand	Cobb County, GA	D-3-4	14.5	7.3	8	1.41	36.2	26.1	1.8	52.2	100
75	Cohesionless	Sand	Cobb County, GA	D-3-25	14.2	11.5	8	1.41	36.2	25.56	1.8	51.12	100
76	Cohesionless	Sand	Cobb County, GA	D-3-32	14	9	8	1.41	36.2	25.2	1.8	50.4	100
77	Cohesionless	Sand	Cobb County, GA	D-3-14	12.4	16.7	8	1.41	36.2	22.32	1.8	44.64	90
78	Cohesionless	Sand	Cobb County, GA	D-3-13	13.5	3.2	8	1.41	36.2	24.3	1.8	48.6	99
79	Cohesionless	Sand	Cobb County, GA	D-3-6	13.5	7	8	1.41	36.2	24.3	1.8	48.6	100
80	Cohesionless	Sand	Cobb County, GA	D-3-12	13.2	3.6	8	1.41	36.2	23.76	1.8	47.52	99
81	Cohesionless	Sand	Cobb County, GA	D-3-11	12.2	4.5	8	1.41	36.2	21.96	1.8	43.92	93
82	Cohesionless	Sand	Cobb County, GA	D-3-29	9.1	6	8	1.41	36.2	16.38	1.8	32.76	70

Table C-3. Summary of estimation and prediction of nominal bond resistance—fine-grained soils.

No.	Type of Natural Material	Soil Type	Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B (in.)	Design Load, DL (kips)	Test Design Load, DL (kips)	Estimated Pullout Resistance, Q (kips/ft)	Predicted Resistance (kips)	Measured Resistance (kips)
1	Fine-grained	Sandy Clay	San Luis Obispo, CA	D-5-1	11	18	6	1(6)	15.8	17.6	1.6	35.2	31
2	Fine-grained	Sandy Clay	San Luis Obispo, CA	D-5-2	13	13	6	0.875	15.8	20.8	1.6	41.6	37
3	Fine-grained	Clay	Solana Beach, CA	D-6-1	15.3	6.5	8	1	22	16.83	1.1	33.66	31
4	Fine-grained	Clay	Solana Beach, CA	D-6-2	17	4	8	1	22	18.7	1.1	37.4	35.7
5	Fine-grained	Clay	Solana Beach, CA	D-6-3	16	7.5	8	1	22	17.6	1.1	35.2	33.8
6	Fine-grained	Clay	Solana Beach, CA	D-6-4	16.75	6.5	8	1	22	18.425	1.1	36.85	35.6
7	Fine-grained	Clay	Solana Beach, CA	D-6-5	16.8	6.5	8	1	22	18.48	1.1	36.96	35.9
8	Fine-grained	Clay	Solana Beach, CA	D-6-6	15.4	6.5	8	1	22	16.94	1.1	33.88	33.0
9	Fine-grained	Clay	Solana Beach, CA	D-6-7	16.4	12.5	8	1	22	18.04	1.1	36.08	35.4
10	Fine-grained	Clay	Solana Beach, CA	D-6-8	15.25	13.5	8	1	22	16.775	1.1	33.55	33.0
11	Fine-grained	Clay	Solana Beach, CA	D-6-9	13	14	8	1	22	14.3	1.1	28.6	28.3
12	Fine-grained	Clay	Guadalupe River, CA	D-10-20	10	15	8	0.875	13.6	13.6	1.4	27.2	27
13	Fine-grained	Clay	Solana Beach, CA	D-6-10	13	8	8	1	22	14.3	1.1	28.6	28.5
14	Fine-grained	Clay	Solana Beach, CA	D-6-11	14.5	12	8	1	22	15.95	1.1	31.9	31.9
15	Fine-grained	Clay	Solana Beach, CA	D-6-12	14.2	8.8	8	1	22	15.62	1.1	31.24	31.4
16	Fine-grained	Clay	Solana Beach, CA	D-6-13	14.2	9.3	8	1	15.6	15.62	1.1	31.24	31.6
17	Fine-grained	Clay	Solana Beach, CA	D-6-14	15	8.2	8	1	22	16.5	1.1	33	33.5
18	Fine-grained	Clay	Solana Beach, CA	D-6-15	15.4	17.8	8	1	22	16.94	1.1	33.88	34.6
19	Fine-grained	Clay	Solana Beach, CA	D-6-16	16.75	6.5	8	1	22	18.425	1.1	36.85	37.8
20	Fine-grained	Clay	Solana Beach, CA	D-6-17	12	10.5	8	1	22	13.2	1.1	26.4	27.2
21	Fine-grained	Clay	Solana Beach, CA	D-6-18	15.5	7.7	8	1	22	17.05	1.1	34.1	35.3
22	Fine-grained	Clay	Solana Beach, CA	D-6-19	15.5	8	8	1	22	17.05	1.1	34.1	35.5
23	Fine-grained	Clay	Solana Beach, CA	D-6-20	17.8	5	8	1	22	19.58	1.1	39.16	40.9
24	Fine-grained	Clay	Solana Beach, CA	D-6-21	17.3	5.7	8	1	22	19.03	1.1	38.06	40.0
25	Fine-grained	Clay	Solana Beach, CA	D-6-22	16.8	6.25	8	1	22	18.48	1.1	36.96	39.0
26	Fine-grained	Clay	Solana Beach, CA	D-6-23	17.25	5.7	8	1	22	18.975	1.1	37.95	40.2

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Table C-3. (Continued).

No.	Type of Natural Material	Soil Type	Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B (in.)	Design Load, DL (kips)	Test Design Load, DL (kips)	Estimated Pullout Resistance, Q (kips/ft)	Predicted Resistance (kips)	Measured Resistance (kips)
27	Fine-grained	Clay	Solana Beach, CA	D-6-24	16.8	6	8	1	22	18.48	1.1	36.96	39.4
28	Fine-grained	Clay	Guadalupe River, CA	D-10-8	7.5	15	8	0.875	13.6	10.2	1.4	20.4	22
29	Fine-grained	Clay	Guadalupe River, CA	D-10-2	10	20	6	0.875	13.6	13.6	1.4	27.2	30
30	Fine-grained	Clay	Guadalupe River, CA	D-10-9	10	15	8	0.875	13.6	13.6	1.4	27.2	31
31	Fine-grained	Clay	Guadalupe River, CA	D-10-19	10	15	8	0.875	13.6	13.6	1.4	27.2	32
32	Fine-grained	Silty Clay	Chattanooga, TN	1	8	NA	6	1	16	16	2.0	32	38
33	Fine-grained	Clay	Guadalupe River, CA	D-10-1	10	20	6	0.875	13.6	13.6	1.4	27.2	33
34	Fine-grained	Clay	Guadalupe River, CA	D-10-5	10	15	8	0.875	13.6	13.6	1.4	27.2	33.5
35	Fine-grained	Clay	Guadalupe River, CA	D-10-6	10	15	8	0.875	13.6	13.6	1.4	27.2	34
36	Fine-grained	Clay	Guadalupe River, CA	D-10-3	10	20	6	0.875	13.6	13.6	1.4	27.2	35
37	Fine-grained	Clay	Guadalupe River, CA	D-10-13	10	15	8	0.875	13.6	13.6	1.4	27.2	36
38	Fine-grained	Clay	Guadalupe River, CA	D-10-4	10	15	8	`	13.6	13.6	1.4	27.2	37
39	Fine-grained	Clay	Guadalupe River, CA	D-10-7	10	15	8	0.875	13.6	13.6	1.4	27.2	38
40	Fine-grained	Sandy Lean Clay	San Luis Obispo, CA	D-5-4	10	10	6	0.875	15.8	16	1.6	32	46
41	Fine-grained	Clay	Guadalupe River, CA	D-10-10	10	15	8	0.875	13.6	13.6	1.4	27.2	40
42	Fine-grained	Clay	Guadalupe River, CA	D-10-11	10	20	8	0.875	13.6	13.6	1.4	27.2	41
43	Fine-grained	Clay	Guadalupe River, CA	D-10-14	10	15	8	0.875	13.6	13.6	1.4	27.2	42
44	Fine-grained	Clay	Guadalupe River, CA	D-10-17	10	15	8	0.875	13.6	13.6	1.4	27.2	43
45	Fine-grained	Clay	Guadalupe River, CA	D-10-16	10	15	8	0.875	13.6	13.6	1.4	27.2	44

Table C-4. Summary of estimation and prediction of nominal bond resistance—rock.

No.	Type of Natural Material	Soil Type	Location	Test ID	Bonded Length, L_B (ft)	Unbonded Length, L_U (ft)	Drill-Hole Diameter, D_{DH} (in.)	Nail Bar Diameter, D_B [5] (in.)	Design Load, DL (kips)	Test Design Load, DL (kips)	Estimated Pullout Resistance, Q (kips/ft)	Predicted Resistance (kips)	Measured resistance (kips)
1	Rock	Mélange	Marin County, CA	D-8-10	15	5	6	NA	27.1	40.5	2.7	81	55
2	Rock	Mélange	Marin County, CA	D-7-5	10	10	6	1	34	34	3.4	68	47
3	Rock	Mélange	Marin County, CA	D-7-4	10	10	6	1	34	34	3.4	68	50
4	Rock	Mélange	Marin County, CA	D-7-3	10	10	6	1	34	34	3.4	68	53
5	Rock	Mélange	Marin County, CA	D-8-1	9	15	6	NA	27.1	24.3	2.7	48.6	40
6	Rock	Mélange	Marin County, CA	D-8-3	10	10	6	NA	27.1	27	2.7	54	47
7	Rock	Mélange	Marin County, CA	D-7-6	10	10	6	1	34	34	3.4	68	62
8	Rock	Mélange	Marin County, CA	D-7-1	10	10	6	1.27(6)	34	34	3.4	68	65
9	Rock	Mélange	Marin County, CA	D-7-2	10	10	6	1.27(6)	34	34	3.4	68	67
10	Rock	Shale	Pike County, KY	P-1-7	9.8	26.2	4	1.27	42.15	42.14	4.3	84.28	84
11	Rock	Mélange	Marin County, CA	D-8-12	10	19	6	NA	27.1	27	2.7	54	54
12	Rock	Shale	Pike County, KY	P-1-2	9.8	29.5	4	1.27	42.15	42.14	4.3	84.28	85
13	Rock	Mélange	Marin County, CA	D-8-5	10	10	6	NA	27.1	27	2.7	54	55
14	Rock	Shale	Pike County, KY	P-1-8	9.8	19.7	4	1.27	42.15	42.14	4.3	84.28	86
15	Rock	Shale	Pike County, KY	P-1-1	9.8	26.2	4	1.27	42.15	42.14	4.3	84.28	88
16	Rock	Shale	Pike County, KY	P-1-5	9.8	19.7	4	1.27	42.15	42.14	4.3	84.28	89
17	Rock	Shale	Pike County, KY	P-1-3	9.8	31.2	4	1.27	42.15	42.14	4.3	84.28	90
18	Rock	Shale	Pike County, KY	P-1-6	9.8	31.2	4	1.27	42.15	42.14	4.3	84.28	91
19	Rock	Shale	Pike County, KY	P-1-4	9.8	14.8	4	1.128	42.15	42.14	4.3	84.28	94
20	Rock	Shale	Pike County, KY	P-1-10	9.8	4.9	4	1.27	42.15	42.14	4.3	84.28	95
21	Rock	Mélange	Marin County, CA	D-8-6	9	17	6	NA	27.1	24.3	2.7	48.6	55
22	Rock	Shale	Pike County, KY	P-1-9	9.8	29.5	4	1.27	42.15	42.14	4.3	84.28	99
23	Rock	Shale	Pike County, KY	P-1-12	9.8	19.7	4	1.27	42.15	42.14	4.3	84.28	102
24	Rock	Mélange	Marin County, CA	D-8-4	9	11	6	NA	27.1	24.3	2.7	48.6	60
25	Rock	Shale	Pike County, KY	P-1-11	9.8	4.9	4	1.27	42.15	42.14	4.3	84.28	105
26	Rock	Mélange	Marin County, CA	D-8-2	7	13	6	NA	27.1	18.9	2.7	37.8	48

Table C-5. Statistics of bias for nominal bond strength.

Material	Resistance Parameters						
	Number of Points in Database	Distribution Type	Mean of Bias	Standard Deviation	Coefficient of Variation	Log Mean of Bias	Log Standard Deviation
	N		λ_R	σ_R	COV_R	μ_{ln}	σ_{ln}
Sand and Sand/Gravel	82	Lognormal	1.050	0.25	0.24	0.02	0.24
Fine-Grained	45	Lognormal	1.033	0.05	0.05	0.03	0.05
Rock	26	Lognormal	0.920	0.18	0.19	-0.10	0.19
All	153	Lognormal	1.050	0.22	0.21	0.03	0.21

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APPENDIX D

Comparison of ASD- and LRFD-Based Designs of Soil Nail Walls

D.1 Introduction

This appendix presents a comparison of SNW designs based on the ASD and LRFD approaches. The comparison was between designs of identical cases and conditions of SNWs for both the ASD and LRFD approaches. Designs were performed using the computer programs SNAILZ (Caltrans, 2007) and GOLDNAIL (Golder, 1993), the two most commonly used computer programs for SNW design in the United States. Section D.2 provides a brief description of these two computer programs. Section D.3 provides an overview of the comparisons. Section D.4 contains results of a parametric study conducted to assess the design sensitivity to various factors. Section D.5 presents results of a comparison based on a design example presented in the FHWA Geotechnical Engineering Circular (GEC) 7 (Lazarte et al., 2003) of a SNW using the ASD method and designs for the same wall using the LRFD method.

D.2 Computer Programs Used in Comparative Analyses

D.2.1 SNAILZ

Basic Features

SNAILZ, developed by the California Department of Transportation (Caltrans, 2007), is an updated version of the program SNAIL (Caltrans, 1991) and is currently the most widely used program in the United States for the design of SNWs. The program is available through the public domain and can be downloaded free-of-charge from http://www.dot.ca.gov/hq/esc/geotechlrequest.htm. Technical support is limited. SNAIL was originally in a Microsoft® DOS platform. SNAILZ runs within a Microsoft Windows® environment. SNAILZ is versatile as it allows the design engineer to consider various design scenarios and the most common elements that participate in the design of a SNW. The user can input nail bond and tensile resistances, as well as the facing resistance.

Program Capabilities

SNAILZ can model only two-dimensional wall geometries. It is based on the limit-equilibrium method and only achieves force equilibrium. Moment equilibrium is generally not achieved in this program; therefore, results from SNAILZ are only approximate but are considered acceptable for design purposes.

SNAILZ uses two-part planar wedges. It can model slip surfaces with one wedge exiting the SNW toe and the other to the ground surface behind the modeled wall [Figure D-1(a)]. This is the most common scenario for SNWs. The program can also model approximately a slip surface extending behind and below the wall using a simplified passive earth pressure formulation for the section below the wall toe [Figure D-1(b)]. However, this solution approach is only approximate. Therefore, the sliding and basal heave limit states can be modeled only approximately with this program.

SNAILZ can model up to seven soil layers. Up to three points define the water table location, which for some groundwater conditions may not be sufficient. SNAILZ allows a maximum of two uniform surcharge distributions behind the face of the wall. Therefore, the program may have limited capabilities to model complex stratigraphy and load conditions. For complex wall geometries, stratigraphy distributions, or load conditions, the design engineer may need to simplify actual conditions due to the program limitations. However, for most common conditions encountered in SNW design practice, this program produces acceptable results, even in relatively complex design situations.

Input Parameters

Parameters selected for input in SNAILZ include those related to reinforcement, loads, and soil. Reinforcement parameters include nail head depth on the wall face, nail diameter, nail inclination, vertical and horizontal nail spacing, bar cross-sectional area, and nail tensile resistance. These parameters can

Figure D-1. Slip surfaces used in SNAIL: (a) two wedges through toe and (b) two wedges under toe.

be assigned either to individual nails or globally to all nails. Up to two uniform surcharge distributions can be input. Pseudo-static seismic loads can be considered in SNAILZ by entering horizontal and vertical seismic coefficients. Soil parameters include soil unit weight, soil cohesion, friction angle, and bond resistance. Soil nominal resistance is modeled in SNAILZ using the Mohr-Coulomb failure envelope model. Another input parameter that must be included is the facing resistance. Input data can be entered in the English or SI unit systems.

Use of Computer Programs for LRFD Method Analyses

SNAILZ is ASD based; therefore, SNAILZ strictly provides calculated global factors of safety, FS_G, for overall stability. The program cannot be used to perform an analysis using LRFD methodologies unless simplifying assumptions are made and intermediate calculations are performed. The user may manually input reduced values of nail tensile, pullout, and facing resistances (i.e., nominal values multiplied by the corresponding resistance factors) before the program executes any computations. The user must use the "pre-factored" option available in SNAILZ for reduced values of nail tensile, pullout, and facing resistances. By selecting this option, only soil parameters (cohesion and tangent of friction angle) are affected by FS_G, while the other resistances remain constant throughout the analysis. External loads (i.e., two uniform loads available in SNAILZ) can be entered pre-multiplied by a resistance factor. Earth loads cannot be entered pre-multiplied by a resistance factor.

When the "pre-factored" option is selected and factored values for resistance are entered, SNAILZ can provide equivalent results in ASD format or in a format resembling LRFD. However, this is limited to the condition of load factor $\gamma = 1.0$. When SNAILZ is used to perform an LRFD-equivalent analysis for $\gamma = 1.0$, factored values of the nominal resistances must be entered. For this step, the nominal resistances of soil cohesion, c_s, and the friction angle, φ_s, must be affected by multiplying manually these values by soil resistance factors. Note that in SNAILZ, φ_s, not the tangent of the angle ($\tan \varphi_s$) is input. Therefore, an equivalent reduced friction angle ($\varphi_{s\ red}$) is computed as $\varphi_{s\ red} = \tan^{-1} [\tan(\varphi_s) \times \phi]$ and entered. With these factored nominal resistances entered, the condition $FS_G = 1.0$ in SNAILZ would represent a limit state for global stability.

The ASD and "LRFD" modes would be equivalent in SNAILZ only for $\gamma = 1.0$. If load factors different than 1.0 were used in the "LRFD" format in SNAILZ, inconsistent results between the ASD and LRFD "modes" would be obtained. In addition, affecting soil loads with load factors different than 1.0 is not possible in SNAILZ. For example, an attempt to affect the soil unit weight by earth load factors (in general > 1.0) would also affect earth load effects on the resistance side and would ultimately produce inconsistent results between the ASD and the LRFD-equivalent analyses in SNAILZ.

In summary, the only practical way to use SNAILZ with a LRFD format is to set all load factors equal to 1.0, which is consistent with a service limit state for overall stability as is currently adopted in the *LRFD Bridge Design Specifications* (AASHTO, 2007). For, load factors > 1.0, inconsistent results are obtained.

D.2.2 GOLDNAIL

Basic Features

GOLDNAIL is a Windows-based proprietary program developed by Golder Associates (Golder, 1993). Although GOLDNAIL is not as commonly used as SNAILZ, the program offers more advanced analysis capabilities and options that allow considering a wider range of scenarios and material properties than SNAILZ. The program is commercialized and some technical support can be obtained for a fee.

Program Capabilities

This program is two-dimensional and satisfies moment and force equilibriums. GOLDNAIL uses circular failure surfaces and analyzes SNWs as a series of slices instead of wedges. In GOLDNAIL, the sliding soil mass is divided into vertical slices, like is typically done in most slope-stability methods. The program iteratively modifies the normal stresses distribution at the base of the slices until force and moment equilibriums are obtained. The program constrains circular slip surfaces to pass through or above the SNW toe. Input data can be entered in the English or SI unit systems, or any other compatible unit system. Sliding and basal heave cannot be assessed using this program. GOLDNAIL may also be used to analyze unreinforced slopes and anchored walls.

GOLDNAIL allows analyzing SNWs using either an ASD-equivalent method or the LRFD method. For each of these methods, the program works in one of the three following calculation modes: (i) Design Mode; (ii) Factor of Safety Mode; and (iii) Nail Service Load Mode. In the Design Mode, the program is executed by modifying some of the factors controlling stability (e.g., nail length) until a target safety factor (ASD method) is calculated or the limit condition (LRFD method) is met. In the Factor of Safety Mode, a global factor of safety, FS_G, is calculated using the ASD method or the limit condition is met (LRFD method) for a specified set of input parameters, including soil nail length. In the Nail Service Load Design Mode, the program provides the maximum in-service tensile forces in the soil nails that are used for the design of the nail bar diameter and facing characteristics resistances.

Input Parameters

Nail and soil parameters are similar to those entered in SNAILZ with a few exceptions. The program can model up to 13 soil layers, complex slopes and subsurface geometries, horizontal and vertical surcharge distributions, groundwater, and pseudo-static, horizontal seismic coefficients. The program only considers uniform spacing and inclination of the nails. Although this scenario is typical for most designs, this assumption may be too restrictive for some cases. Soil strength is modeled using a linear Mohr-Coulomb envelope with the option of using a bi-linear strength envelope. Therefore, if the bi-linear Mohr-Coulomb model option is used, additional sets of cohesion and friction values are needed. In addition, the program allows the input of both vertical and horizontal surcharge loads.

Use of Program in the LRFD Method

For the LRFD method, GOLDNAIL allows the user to input load and resistance factors directly into the program, and there is no need to pre-calculate manually factored resistances. The user can input load factors separately for soil weight, water weight, surcharge and seismic load. Reduction strength factors (i.e., equivalent to the inverse of safety factors) are also entered for other resistance components (i.e., facing or nail head resistance, nail tensile resistance, and bond or pullout resistance). When the ASD method is used in GOLDNAIL, safety factors are entered separately for cohesion and friction.

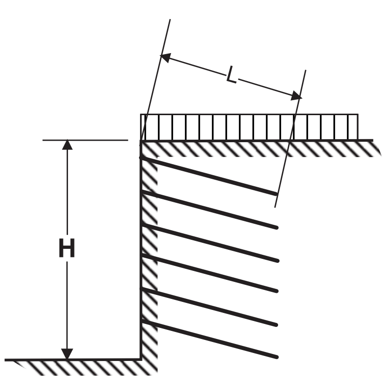
D.3 Comparison of LRFD- and ASD-Based Designs

Designs of SNWs using the LRFD and ASD approaches are compared in two manners in this section. First, a parametric analysis was performed in GOLDNAIL in the ASD and LRFD modes. The objective of this analysis was to assess differences of key design parameters (i.e., nail length, cross sectional area, facing resistances) using the ASD and LRFD modes in the same software to avoid potential inconsistencies. Several wall conditions were inspected and various factors that may influence results were considered. Second, designs of a LRFD design example using GOLDNAIL and SNAILZ (with modified input to emulate a LRFD mode) were compared to the ASD-based design made for the same design example presented in Lazarte et al. (2003). The comparisons are presented in the following subsections.

D.4 Parametric Study

D.4.1 Description

The influence of several factors that may affect the required nail length was evaluated using the LRFD and the ASD in a parametric study. To facilitate the comparisons, a uniform nail pattern and homogeneous soil profile were assumed. A wall of height *H* (Figure D-2) is reinforced with six rows of nails (inclination of 15 degrees) of uniform length, *L*. The parameters analyzed included the wall height, soil friction angle, nail bond resistance, and surcharge. All results were compared against the results of a baseline case, whose parameters are indicated on Figure D-2 and Table D-1. In these analyses, the pullout limit state was assured by selecting an artificially high



Baseline Case

H = 30 ft, φ_s = 35 deg, q_u = 15 psi, Q = 0

Other Cases

Variable Analyzed

Wall height,	H = 20, 30, and 40 ft
Soil friction angle,	φ_s = 28, 32, 35, and 38 deg
Bond resistance,	q_u = 10, 15, 20, and 25 psi
Surcharge,	Q = 0, 250, and 1,000 psf

For all cases: c = 0, γ_s = 120 pcf

Figure D-2. Geometry of SNW in comparative analyses.

Table D-1. Soil nail wall input parameters (baseline case).

Description	Quantity
Wall height (ft)	30
Wall batter (deg)	0
Number of soil nail levels	6
Nail diameter (in.)	1.128
Diameter of grouted hole (in.)	6
Nail inclination (deg)	15
Nail vertical spacing (ft)	5
Nail horizontal spacing (ft)	5
Soil unit weight (pcf)	120
Soil friction angle (deg)	35
Bond stress (psi)	15

nail yield resistance and facing resistance. However, this is not a typical manner of analyzing SNWs.

Because the calibrated pullout resistance factors are values that are close to 0.5 (a value that would have been derived through a calibration with safety factors), the results between the LRFD and ASD methods are expected to be similar.

D.4.2 Results

Results for the over 30 analyzed design cases are summarized in Table D-2. Results confirm what was expected: using the reliability-calibrated resistance factors of Chapter 3, the calculated nails length that are required to satisfy design criteria are comparable using both the LRFD and ASD methods. For all cases considered, the calculated nail length is, on average, approximately 4 percent larger in the LRFD method. No single factor appears to have a significantly greater influence on the results. Slightly larger differences were obtained for large loads and for high nominal pullout or bond resistances. The largest difference obtained for nail length was approximately 8 percent.

The soil nail loads calculated via the ASD or LRFD modes were similar, with differences on average less than 3 percent. As a result, it is expected that the differences in calculating the necessary nail cross sectional area and required facing resistance would be almost identical using either the ASD or LRFD methods for $\gamma = 1.0$.

D.5 Example Design of a SNW

D.5.1 Design Conditions

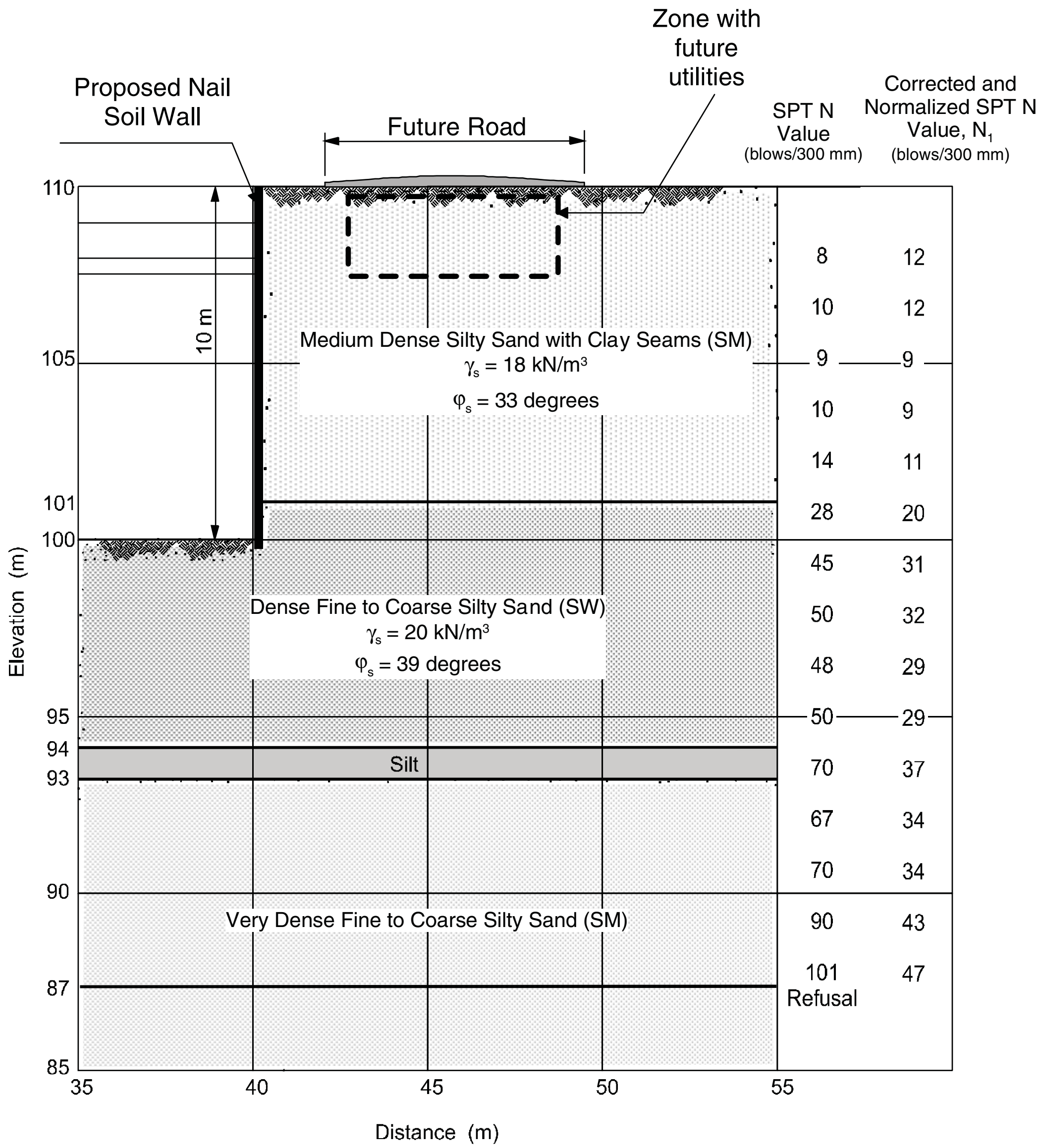
In this design example (Figure D-3), the soil profile behind the proposed SNW and the project requirements are similar to those of the design example presented in Appendix D of FHWA GEC No. 7 (Lazarte et al., 2003). The objective of this exercise is to compare the results obtained from the two most common SNW software programs in the design of a wall with realistic conditions.

The wall conditions are as follows. A 10-m (33-ft) high SNW is to be constructed as part of a roadway project. The road adjacent to the proposed wall is of low-to-medium

Table D-2. Comparison of required nail length using ASD and LRFD approaches.

Variable Compared	Case	Variable Value	Required Length, L (ft)			LRFD to ASD Percent Difference (w/ respect to baseline case, %)	
			ASD	LRFD			
			$FS_G = 1.5$	$\phi_{PO} = 0.49$	$\phi_{PO} = 0.47$	$\phi_{PO} = 0.49$	$\phi_{PO} = 0.47$
–	Baseline	$H = 30$ ft, $\varphi_s = 35°$ $q_u = 15$ psi, $Q = 0$	23.43	24.14	24.48	3.03	4.48
Wall Height	1	$H = 40$ ft	31.24	32.18	32.63	3.01	4.45
	2	$H = 20$ ft	15.62	16.16	16.31	3.46	4.42
Friction Angle	1	$\varphi_s = 28°$	27.59	28.43	28.99	3.04	5.07
	2	$\varphi_s = 32°$	25.22	25.99	26.51	3.05	5.11
	3	$\varphi_s = 38°$	21.64	22.29	22.74	3.00	5.08
Bond Resistance	1	$q_u = 10$ psi	26.28	27.59	28.42	4.98	8.14
	2	$q_u = 20$ psi	18.93	19.39	19.78	2.43	4.49
	3	$q_u = 25$ psi	17.14	17.67	17.83	3.09	4.03
Surcharge	1	$Q = 250$ psf	36.09	37.18	38.69	3.02	7.20
	2	$Q = 500$ psf	40.67	41.91	43.61	3.05	7.23

Note: ϕ_{PO} were calibrated for a reliability factor of 2.33



Source: Lazarte et al. (2003)

Figure D-3. Subsurface stratigraphy and design cross section.

traffic volume and is considered non-critical. A 7.3-m (24-ft) wide road will be constructed 3 m (9.8 ft) behind the wall. The wall is to be constructed in medium-dense silty sand with clay seams with the soil nails shown in Figure D-4. The parameters used for the SNW design are as follows:

A. Wall Layout
   Wall height, $H = 10$ m (33 ft);
   Wall length Wall Length >> H; and
   Face batter, $\alpha = 0$.

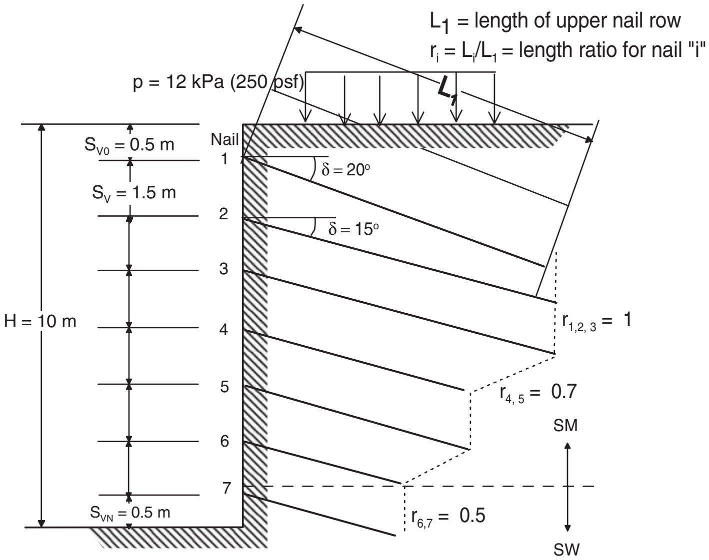
B. Soil Nail Vertical and Horizontal Spacing, $S_H = S_V = 1.5$ m (5 ft).

C. Soil Nail Inclination, i
   $i = 20$ degrees (for top row of nails to avoid utilities); and
   $i = 15$ degrees (for other nail rows).

D. Soil Nail Length Distribution
   The soil nail length is variable as indicated by length ratios r_i (see Figure D-4)

E. Nail Yield Tensile Resistance, $f_y = 520$ MPa (75 ksi)

F. Soil Properties and Ground Conditions
   1. Upper Silty Sand Deposit:
      $\varphi_s = 33$ degrees
      $c' = c_s = 0$ (conservative for long-term design conditions)
      $\gamma_s = 18$ kN/m³ (115 pcf)
   2. Lower Silty Sand Deposit:
      $\varphi_s = 39$ degrees
      $c' = c_s = 0$
      $\gamma_s = 20$ kN/m³ (125 pcf)
   3. Groundwater: absent.



Source: Lazarte et al. (2003)

Figure D-4. Non-uniform nail length pattern.

G. Drill-Hole Diameter, D_{DH} = 150 mm (6 in.)

H. Bond Resistance:

Upper Silty Sand: q_u = 100 kPa (14.5 psi); and

Lower Silty Sand: q_u = 150 kPa (21.8 psi).

I. Load Combination and Load Resistance Factors

The combination of loads for the project conditions is adopted from AASHTO (2007) recommendations. The load combination considered is Service Limit I. The load combinations and load factors based on AASHTO (2007) recommendations are $\gamma = 1.0$.

J. Facing Features

See Table D-3.

For a mesh 152 × 152 – MW19 × MW19 (6 × 6 – W2.9 × W2.9 mesh in English units) and using Table A.2 of Lazarte et al. (2003), the total reinforcement area per unit length at midspan is:

$$a_{vm} = a_{hm} = 123\ \text{mm}^2/\text{m} = 1.23 \times 10^{-4}\ \text{m}^2/\text{m}\left(0.058\ \text{in}^2/\text{ft}\right)$$

At the nail, there are two No. 13 (No. 4) vertical and horizontal (waler) bars. Using Table A.3 of Lazarte et al. (2003), the total nominal area in each direction is:

$$A_{VW} = A_{HW} = 2 \times 129 = 258\ \text{mm}^2\left(0.4\ \text{in.}^2\right)$$

The total reinforcement area per unit length around the nails is:

$$a_{vn} = a_{vm} + \frac{A_{VW}}{S_H} =$$

$$a_{vn} = \frac{123 \times 1.5 + 258}{1.5} = 295\ \text{mm}^2/\text{m}$$

$$= 2.95 \times 10^{-4}\ \text{m}^2/\text{m}\left(0.14\ \text{in.}^2/\text{ft}\right)$$

The reinforcement ratio at the nail head and at the midspan, and the total ratio are calculated as:

$$\rho_n = \frac{a_{vn}}{b\,h/2} \times 100 = \frac{a_{hn}}{b\,h/2} \times 100$$

$$\rho_m = \frac{a_{vm}}{b\,h/2} \times 100$$

$$\rho_{TOT} = \frac{\left(a_{vn} + a_{vm}\right)}{b\,h/2} \times 100$$

$$\rho_{TOT} = \frac{\left(295 + 123\right)}{1{,}000 \times 50} \times 100 = 0.84\%$$

Table D-3. Facing features.

Element	Description	Temporary Facing	Permanent Facing
General	Thickness (*h*)	100 mm (4 in.)	200 mm (8 in.)
	Facing Type	Shotcrete	CIP Concrete
	Comp. Strength, f'_c	21 MPa (3,000 psi)	28 MPa (4,000 psi)
Reinforcement	Type	WWM	Steel Bars Mesh
	Grade	420 (Grade 60)	420 (Grade 60)
	Denomination	152 × 152 MW 19 × MW 19 (6 × 6 - W2.9 × W2.9)	No. 13 @ 300 mm (each way) [No. 4 @ 12 in. (each way)]
Other Reinf.	Type	Waler Bars 2 × 13 mm (2 × #8)	–
Bearing Plate	Type	4 Headed-Studs $^1/_2 \times 4^1/_8$	–
	Steel	250 MPa (Grade 420)	–
	Dimensions	Length; L_P = 225 mm (9 in.)	–
		Thickness: t_P = 25 mm (1 in.)	–
Headed-Studs	Dimensions	–	Nominal Length: L_s = 105 mm (4 in.)
		–	Head Diameter: D_H = 25.4 mm (1 in.)
		–	Shaft Diameter: D_S = 12.7 mm ($^1/_2$ in.)
		–	Head Thickness: t_H = 7.9 mm (0.3 in.)
		–	Spacing: S_S = 150 mm (6 in.)

Table D-4. Resistance factors for overall stability.

Resistance Factor	Value
Soil Shear Resistance ϕ_s	0.65
Nail Pullout Resistance, ϕ_{PO}	0.49
Nail Tendon Resistance, ϕ_T	0.56
Nail Head Resistance (flexure and punching shear), ϕ_{FF} **(controls)**	0.67
Nail Head Resistance (headed-stud in tension), ϕ_{FH}	0.50

D.5.2 Design Procedures

Based on the recommendation of AASHTO (2007), the overall stability of the SNW is assessed using the load combination for service limit state.

The resistance factors in Table D-4 are used based on the recommendation of AASHTO (2007). For the pullout resistance factor, the values calibrated in Chapter 3 for sand are used.

The overall stability of the SNW system is evaluated using SNW design software. The following are examples of design analysis results obtained using the programs SNAILZ and GOLDNAIL.

GOLDNAIL

1) Define geometry of wall. Trial nail lengths are selected as follows:

Nail Layers	Trial Nail Length (ft)
1 through 3	30
4 and 5	21
6 and 7	15

Due to the limitations in the program, the nail inclination of nail layer #1 is selected to be the same as other layers (i.e., 15° instead of 20° as shown in Figure D-4).

2) Input the following parameters:
To ensure that pullout failure controls over tensile or punching-shear failure, artificially large values of nail diameter and nail head resistance can be entered in GOLDNAIL. For consistency with the example in GEC No. 7, the following nail bar and head resistances are selected:
   - Threaded bar: No. 8, 25 mm diam., cross-sect. area = 510 mm^2 (0.79 in.^{2})
   - Nail nomin. tensile resist. = 0.79 in.^{2} × 75 ksi = 59.3 kips
   - Nail head nominal resistance (for permanent facing) = 92 kip, from page D-28 (Lazarte et al., 2003)
   - Nail pullout nominal resistance (per linear ft):
     Upper silty sand: π × 6 in. × 1 ft × 12 in./ft × 14.5 psi = 3,280 lbs; and
     Lower silty sand: π × 6 in. × 1 ft × 12 in./ft × 21.8 psi = 4,931 lbs.
   - Soil Design Parameters:

     Upper Silty Sand Deposit:
     φ_s = 33 degrees
     c' = 0
     γ_s = 115 pcf

     Lower Silty Sand Deposit:
     φs = 39 degrees
     c' = 0
     γs = 125 pcf

3) Input the following load and resistance factors:
In the safety factor screen, select LRFD mode. The load factors for water weight, soil weight surcharge, and seismic load are all selected to be 1.0. Input the resistance factors as shown in Table D-4.

4) Compute the necessary nail length and head resistance:
In GOLDNAIL, run analysis using the design analysis mode. The required nail lengths to achieve a resistance-to-load ratio greater than 1.0 are calculated.

Nail Layers	Required Nail Length (ft)
1 through 3	32.7
4 and 5	22.9
6 and 7	16.3

The maximum force occurs in the lowermost nail at 32,070 kip (as obtained from the nail service mode in GOLDNAIL).

Figure D-5 shows the calculated critical failure surface.

SNAILZ

In order to perform an analysis that resembles the LRFD format in SNAILZ, resistances must be modified. Note that in this example, the service limit state is analyzed and all load factors are equal to 1.0. Below is a summary of the modified input parameters using a FS = 1.0 (i.e., an equivalent of the LRFD):

Upper Silty Sand Deposit:
φ_s = tan^{-1}(0.65 tan33°) = 22.9°
c' = 0 × 0.65 = 0
γ_s = 115 × 1.0= 115 pcf
q_u = 14.5 psf (nominal value)
BSF = 0.49 (Bond Stress Factor, equivalent to pullout resistance factor)
q = 14.5 psf × 0.49 = 7.11 psf (factored value)

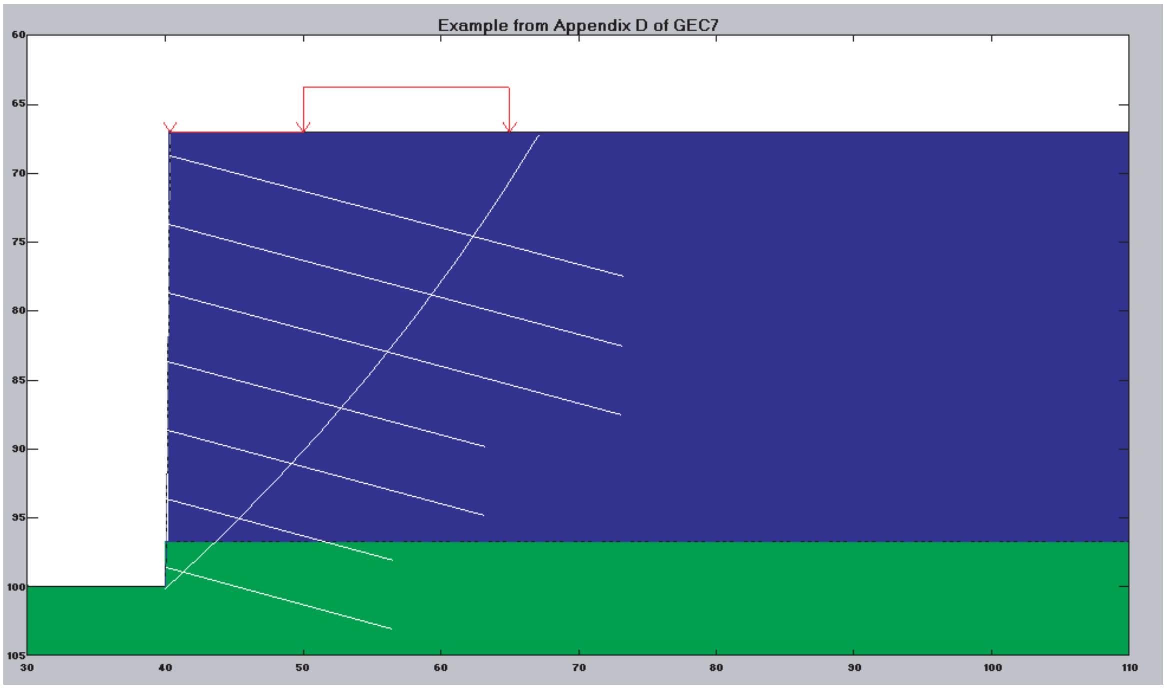


Figure D-5. Critical failure surface calculated using GOLDNAIL.

Lower Silty Sand Deposit:

$\varphi_s \ = \tan^{-1}(0.65 \tan 39°) = 27.8°$

$c' \ = 0 \times 0.65 = 0$

$\gamma_s \ = 125 \times 1.0 = 125$ pcf

$q_u \ = 21.8$ psf (nominal value)

$BSF = 0.49$ (Bond Stress Factor)

$q \ = 21.8$ psf $\times$ $0.49 = 10.7$ psf (factored value)

Nail Head and Nail Tensile Resistances:

Facing resistance = 92 (nominal) × 0.67 = 61.3 (kips); and

Tensile resistance (force) = 59.3 (nominal) × 0.56 = 32.9 (ksi).

Tensile resistance (stress) = 75 (nominal) × 0.56 = 41.7 (ksi).

Nail lengths need to be computed in SNAILZ iteratively in different runs until a target factor of safety of 1.0 (i.e., a condition equivalent to the limit state) is achieved. Figure D-6 shows the critical failure surface calculated by SNAILZ. The required nail lengths as calculated with this procedure are listed below.

Nail Layers	Required Nail Length (ft)
1 through 3	34.1
4 and 5	23.9
6 and 7	17

The maximum calculated nail force is 32.7 kip (in the lowermost nail).

Note that these values are almost identical to those obtained using the ASD method according to GEC 7 (and using the prefactored mode in SNAILZ).

The comparison indicates that SNAILZ requires nails that are approximately 4 percent longer than those obtained using GOLDNAIL. The maximum nail forces in SNAILZ are approximately 2 percent larger than with GOLDNAIL.

D.6 Discussion of Results

Comparative analyses show that both the LRFD and ASD method provide comparable design values for soil nail walls under various conditions. Overall, the comparisons indicate that the required soil nail length calculated using the LRFD method and the proposed resistance factors are comparable with those obtained with the ASD method. For all cases considered, the length difference is on average approximately 4 percent larger in the LRFD method. No factor appears to have greater influence than others do. Slightly larger differences were obtained for large loads and for high nominal pullout or bond resistances. The largest difference obtained in the comparative analysis was approximately 8 percent. In all cases, soil-nail loads calculated using either method are comparable, with a difference of less than about 3 percent.

The analyses using the LRFD method with SNAILZ and GOLDNAIL show that the differences and nail loads are very small, 4 and 2 percent, respectively.

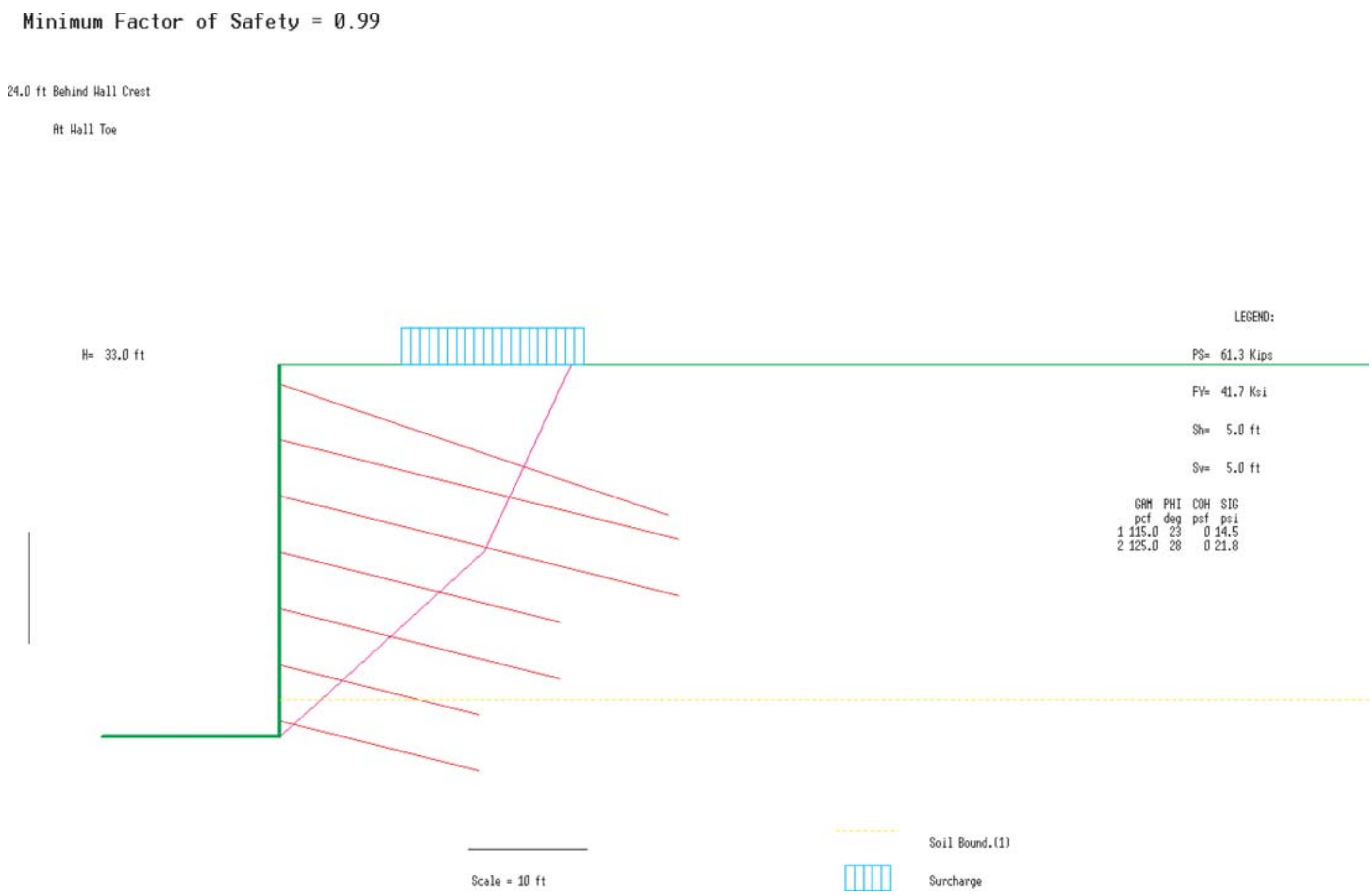


Figure D-6. Critical failure surface calculated using SNAILZ.

D.7 Summary

The comparative analyses confirm that the calculated quantities, including soil nail lengths and cross-sectional areas (as a function of the maximum soil nail force), as obtained using the LRFD and ASD methods are very similar. The reason for these similar trends, which were already apparent in Chapter 3, stem from the fact that the calibrated resistance factors for pullout are very similar to those that could have been obtained directly from a calibration using factors of safety. The differences were small between LRFD and ASD methods using the same program (i.e., GOLDNAIL) and between different programs using LRFD and ASD methods. Therefore, the calibration and comparison demonstrate that the parameters currently used in practice should not be altered. Adopting the LRFD method and the calibrated resistance factors used herein would only result in a change of design format. However, the design would result in essentially the same quantities. A limitation of these comparisons is that analyses have been performed for load factors equal to 1.0, per the current AASHTO LRFD practice of overall stability. However, it is expected that slightly different results and design quantities would be obtained for conditions other than load factors = 1.0.

References

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Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation