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New Directions in LRFD for Soil Nailing Design and Specifications

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ABSTRACT: In the last decade, the use of soil nailing in the United States has increased significantly as a method to construct retaining structures in cut applications (top-down construction). Under favorable soil conditions, soil nail walls present advantages over traditional retaining structures in slope cuts and excavations including lower construction costs and design redundancy. Soil nail walls have been commonly constructed as temporary structures; however, the use of soil nail walls as permanent structures has increased considerably for highway and other applications. The current direction for soil nail design is moving toward a load and resistance factor specification. To be more than just a restatement of current practice, the calibration of new LRFD norms needs to be soundly based on statistical reliability analysis.

1 INTRODUCTION – LRFD FOR SOIL NAILING

The Federal Highway Administration (FHWA) has developed state-of-the-practice documents related to the analysis, design, and construction of soil nail walls for highway applications. In 1989, FHWA commissioned the first comprehensive U.S. study of soil nailing design and construction practice published as FHWA RD-89-198 (Elias and Juran 1991), which contained the first construction and material specification. In 1993, as a result of a FHWA-sponsored scanning tour to Europe (FHWA 1993a), FHWA translated and published the French practice in soil nailing "Recommendations Clouterre" FHWA-SA-93-026 (FHWA 1993b). In 1994, FHWA initiated Demonstration Project 103 (Demo 103) to disseminate the use of soil nail walls, which evolved into design manual FHWA-SA-96-69R (FHWA 1998). The 1998 manual presents two separate design methodologies; one based on the Allowable Stress Design (ASD) method, and the other based on the Load and Resistance Factor Design (LRFD) method. FHWA is currently completing a Geotechnical Engineering Circular (GEC) on the selection, analysis, design, and construction of soil nail walls for highway applications (Lazarte et al., in preparation) using the ASD method.

LRFD methodologies related to the design of earth structures have emerged in the last decade (e.g., Barker et al. 1991). The current version of the American Association of State Highway and Transportation Officials (AASHTO) LRFD Specifications (AASHTO 2002) are gradually replacing ASD methodologies by engineers for the design of highway structures. Refinements to LRFD are being implemented as a result of recent efforts including National Highway Cooperative Research Program (NCHRP)-sponsored research on foundations [e.g., NCHRP Project 12-35, "Recommended Specifications for the Design of Foundations, Retaining Walls and Substructures," (Withiam et al. 1989)], retaining structures [e.g., NCHRP Project 20-7, Task 88, "AASHTO LRFD Specifications for Retaining Walls" (D'Appolonia 1999) and NCHRP Project 12-55, "AASHTO LRFD Specifications for Bridge substructures and Retaining Walls" (Withiam et al. 2002)], deep foundations [NCHRP Project 24-17, "LRFD for Deep Foundations" (Paikowsky et al. 2002)], LRFD for highway bridge substructures (Withiam et al. 1991, 1995, 1997), and other studies, such as for mechanically stabilized earth (MSE) walls (Chen 2000a, Chen 2000b).

Soil nail wall provisions were not contained in the AASHTO Standard Specifications; additionally, information on statistics of soil nail walls was insufficient when the AASHTO LRFD Specifications were first promulgated in 1994 (AASHTO 1994). Consequently, no guidelines for the design and construction of soil nail walls based on the LRFD method were included in AASHTO (1994). Although the 1998 FHWA manual on soil nailing contained an early LRFD procedure, soil nail walls were not included in the AASHTO LRFD Specifications because the load and resistance factors presented in 1998 were not based on statistical calibrations. Instead, the 1988 load and resistance factors were developed by fitting to the then current version of the AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) based on ASD (i.e., the LRFD factors were fit to ASD factors of safety). As statistical validation of LFRD factors for soil nail walls remains missing, guidelines related to soil nailing are similarly missing from current AASHTO LRFD specifications (AASHTO 2002).

Some state transportation agencies have not widely accepted soil nailing due to the lack of AASHTO soil nailing guidelines. Consequently, the advantages of construction with soil nailing technology for highway applications have not been fully realized. Without a technically acceptable soil nailing LRFD specification, the advantages of construction with soil nailing technology may not be fully realized.

2 BASIS OF LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

The LRFD method presents the condition for the satisfactory design of a system, for which it is necessary to evaluate the safety margin for each potential limit state (e.g., resistance or service). In the LRFD method, this condition is expressed in a condensed form as:

$$\phi R_n \geq \sum \gamma_i Q_i \tag{1}$$

The left side of Equation 1 is the resistance term and contains the nominal (ultimate) resistance, R_n, multiplied by the resistance factor, ϕ, which is always less or equal to unity. The nominal resistance of the system is thereby reduced by ϕ to account for uncertainties in resistances. The right side of Equation 1 represents load effects and consists of the sum of the effects of load components, Q_i, multiplied by associated load factors, γ_i. The load factors account for uncertainties in loads derived from the load type, variability, and predictability associated with a particular limit state.

Because the load effect at a particular limit state involves a combination of different load types, Q_i, each of which has a different degree of predictability, load factors differ in magnitude for various load types. Therefore, the total load effect is represented by a summation of $\gamma_i \times Q_i$ products. Additionally, current AASHTO LRFD-based procedures also consider load modifiers, η (not shown on the right side of Equation 1), that modify the loads to account for the effects of redundancy, ductility, and operational importance of the structure. Current practice using LRFD for substructure design assumes η = 1.0 because data are insufficient to account for these specific effects for the design of geotechnical features.

In the LRFD method, both resistance and load factors are selected using statistical procedures to satisfy the condition that the joint probability that the actual loads are larger than design values and the actual system capacity is lower than the design capacity is low. In other words, in an adequate design it is intended that the probability that the capacity (possibly overestimated) of the system is larger than the loads (possibly underestimated) is high.

3 SOIL NAILING

The condition stated above represents a design target criterion that must be met for each plausible limit state for the system being considered. Limit states of soil nail walls are described in the following sections.

3.1 Soil Nail Wall Limit States

Like other structures, soil nail walls have two primary types of limiting conditions: *Service Limit States* and *Strength Limit States*. Service limit states refer to conditions that do not involve collapse but can impair the normal and safe operation and function of the structure. The major service limit state associated with soil nail walls is excessive horizontal wall deformation. Other service limit states include total or differential settlements and cracking of concrete facing. Strength limit states refer to failure or collapse modes that result when applied loads are larger than the overall strength or the strength of individual components, and the structure becomes unstable. Strength limit states arise when failure mechanisms develop as a whole or in resisting elements. These limiting states are more critical in the design of soil nail walls.

Because soil nail walls include various elements, ranging from soil to structural (i.e., soil nail bar, nail head, concrete facing), numerous strength limit states, each associated with a potential failure, are feasible. Failure mechanisms associated with strengths limit states can be classified as: external failure mechanisms, internal failure mechanisms, and facing failure mechanisms. Each of these modes is succinctly described in the following sections.

3.2 External failure mechanisms

External failure mechanisms refer to mechanisms in which relatively large failure surfaces develop in the soil while soil nails contribute to stability (e.g., global stability failure). The soil always contributes to the stability of the soil nail wall along the schematic failure surfaces indicated in Figure 1. If the failure surface does not intersect the soil nails, they do not contribute to stability (e.g., sliding stability under and behind the soil nails and bearing failure in soft soils). The evaluation of the external stability is an important aspect in the design of soil nail walls because the magnitude of the failure can be significant and cause major consequences.

3.3 Internal failure mechanisms

The most common internal failure mechanisms related to soil nails include (Figure2): nail pullout failure (i.e., failure along the soil-grout interface due to insufficient intrinsic bond strength or insufficient nail length), tensile failure of the soil nail (i.e., inadequate tensile strength of the soil nail bar), slippage of the bar-grout interface, and bending and shear of nails.

As the common and recommended design practice is to use threaded bars and relatively high-strength grout, the potential slippage between nail and grout can be avoided and therefore disregarded. The shear and bending failure modes of the soil nails are conservatively disregarded in most current design methods.

3.4 Facing Failure Modes

The most common potential failure mechanisms at the facing-nail head connection are: failure due to excessive bending beyond the facing flexural capacity, failure due to punching shear, and failure of headed studs in tension.

For each of the potential failure mechanisms shown in Figures 1 through 3, equations similar to Equation 1 must be derived. For example, for the external potential failure mechanisms shown in Figure 1, the resistance terms contain the soil strength and the contribution of the soil nails to stability and the load terms contain the driving action of the soil and wall weight and other external loads. For the internal potential failure mechanisms shown in Figure 2, the soil nail pullout and tensile capacities must be considered in the resistance terms, while the soil nail tensile force (derived from global stability analysis) must be taken into consideration. For the facing potential failure mechanisms shown in Figure 3, the resistance terms include the capacity of the temporary and permanent shotcrete facing sections and the nail head capacity. The load terms include the value of the nail tensile force at the head, or alternatively a value of an equivalent lateral earth pressure acting behind the facing. In addition, when service limit states are considered, equations that portray a limiting condition (e.g., maximum tolerable wall movement) and contain load effects and "resistance" parameters on either side must be derived.

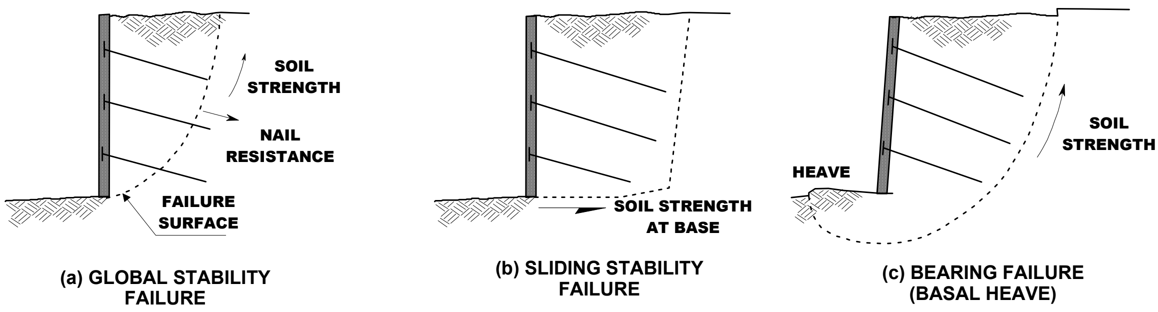

FIGURE 1: EXTERNAL FAILURE MODES

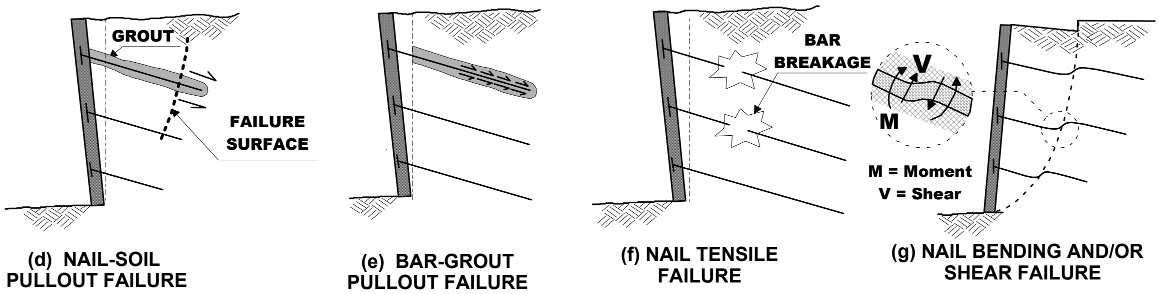

FIGURE 2: INTERNAL FAILURE MODES

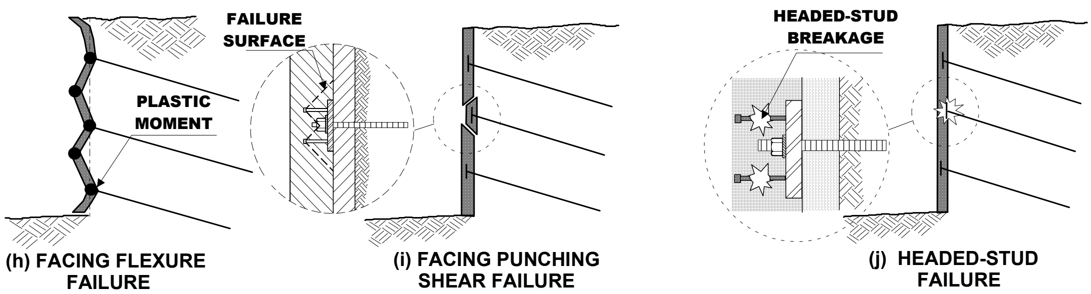

FIGURE 3: FACING FAILURE MODES

3.5 Resistance and LRFD Resistance Factors in Soil Nail Walls

Structural Components: The capacities associated with structural components of a soil nail wall system are: nail bar tensile capacity, flexural capacity of the facing sections, punching shear capacity of facing sections, and capacity of nail-head connectors and hardware. In general, the resistance of individual structural components and connections controls the capacity of a structural system, and the nominal capacity depends on material properties (e.g., strength, stiffness), component geometry, and method of analysis (Ellingwood et al. 1980). Numerous studies have been conducted to develop statistical parameters for various structural elements. For example, reinforced concrete has been studied by Nowak (1999); Nowak et al. (1994); and Vecchio and Collins (1986).

Geotechnical Components: Geotechnical components involve soil internal cohesive and frictional shear strength, and bond shear strength along the soil nail/soil interface that affects the soil nail pullout capacity. The variability of soil parameters has been extensively documented (e.g., Harr 1987). Other significant sources of information are reports conducted to assess reliability of various in situ test methods, laboratory testing, and site characterization in connection with foundations (Kulhawy and Phoon 1996). In addition, the NCHRP Project 12-55 (Withiam et al. 2002) will summarize available statistics of earth loads.

One of the most critical parameters in the design of soil nail walls is bond strength along the nail-soil interface. No studies on the variability of bond strength have been conducted. The bond strength depends on numerous factors including the type of soil in which the soil nail is installed, the drilling method used to create the nail hole, the characteristics of the grout. Presently, there is no viable theoretical relationship that can accurately predict nail pullout capacity. Typically, the soil test data related to pullout resistance are related to STP data and classification index test results. However, no definitive correlations exist. When nail load test data are not available, design engineers use estimates of bond strengths. The most widely used set of presumptive values of bond strength are those developed originally by Elias and Juran (1991) and updated by Lazarte et al. (in preparation). These values are based on data compiled from numerous load tests and have been used extensively by U.S. practitioners. Typical values of ultimate bond strength in grouted nails for various soil conditions and drilling methods are presented in Table 1.

The variability of the bond strength for a particular soil type can be significant. However, when specific knowledge of the subsurface conditions and drilling method is available, the variability of the bond strength decreases considerably. This trend illustrates the importance of a test load database to bring the trends into narrower ranges and decrease the variability.

In contrast with LRFD design for structural elements, the number of factors contributing to the uncertainty associated with geotechnical resistance parameters is large. When selecting LRFD resistance factors for geotechnical applications, the ϕ values affecting soil properties contributing to the resistance in a particular limit state must take into account various sources of uncertainty. These sources include: spatial variability of material properties, extent and applicability of site characterization, validity of the selected resisting model to predict failure, inherent variability and reliability of the individual testing methods, uncertainty in estimating loads when these contribute to stability (i.e., soil weight), construction workmanship, and consequence(s) of failure. Ideally, the selection of ϕ should consider all sources described above. However, current LRFD methodology can consider implicitly the failure consequence and provides a mechanism for systematically including the effects of engineering uncertainties.

TABLE 1: BOND STRENGTH ESTIMATES FOR NAILS IN SOIL AND ROCK
[MODIFIED FROM ELIAS AND JURAN (1991)]

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

3.6 Loads and LRFD Load Factors in Soil Nail Walls

For soil nail walls, the principal loads to consider are: soil weight, dead load, horizontal earth pressure, live load, and earthquake loads (in regions with seismic risk). When high groundwater or perched water exists in the soil mass behind the wall, water pressure (horizontal and uplift) must be also considered. Most commonly, soil nail walls in cut slopes are acted upon by a few of these loads. When the soil nail wall is part of a bridge abutment, numerous other loads including traffic, vehicular impact, temperature, creep, may need to be considered. Loads in soil nail walls can be permanent (e.g., dead load, earth pressures, or permanent internal load such as post-tensioning, as done in a few cases) and transient (live and seismic). In the current LRFD Specifications, seismic loading is a special strength limit state referred to as Extreme Event Limit State I.)

To consider generic loads that are similar to other retaining structures, it is advantageous to consider loads already included in the LRFD Specifications (AASHTO 2002). Another important data source for developing load factors for soil nail walls are statistical parameters of dead and live load obtained from studies conducted in relation to LRFD-based bridge design (e.g., Nowak 1999). For example, the LRFD Specifications (AASHTO 2002) prescribe the live load (LS) as an equivalent soil surcharge, h_{eq}, for retaining walls, as indicated in Table 2. Overburden (vertical) earth pressures (EV) are caused by the weight of earth behind a wall and depend on the depth of soil, density of the existing soil, and the natural water content. Obtaining the horizontal (lateral) earth pressures (EH) acting on any retaining structure presents more difficulties than with EV because horizontal earth pressures depend on the distribution of EV with depth, the nature and stiffness of the wall, the magnitude and distribution of engineering properties of the soil behind the wall, presence and location of water behind the wall, and surcharge loads on the surface. In the case of soil nail walls, the magnitude and distribution of EH also depends on the particulars of the top-down construction method, nail spacing (typically 5 ft), and other aspects affecting the soil-structure interaction. Studies on earth loads for retaining structures are nearing completion for project NCHRP 12-55.

Load values prescribed in the LRFD Specifications are based on the assumption that the exposure of a new structure has a 75-year design life. However, it is important to note that many soil nail walls continue to be built as temporary structures (i.e., structures with a service life typically of 18 months). Therefore, to consider appropriately the shorter service life of temporary soil nail walls, modifications to some of the values of transient loads factors may be necessary. NCHRP Project 2-55 will result in modifications for LL for temporary structures based on load statistics already developed by Nowak (1999).

The load factors γ_i to be applied to loads affecting earth structures like a soil nail wall must consider the type and nature of load, likelihood of various load combinations, and reliability of the field and laboratory procedures used to obtain soil properties if they affect loads (e.g., soil weight). The current AASHTO LRFD Specifications uses the load combinations and load factors presented in Table 3. These load combinations and load factors typically are used in retaining structures used for bridges and appurtenant structures. Note that only the shaded rows of Table 2 are typical limit states and load combination for soil nail applications.

TABLE 2: EQUIVALENT SOIL SURCHARGE REPRESENTING LIVE LOADS IN RETAINING WALLS (AASHTO 2002)

Wall Height (ft)	**h_{eq} (ft)**
5	4
10	3
> 20	2

TABLE 3: LRFD AASHTO LOAD COMBINATIONS AND LOAD FACTORS

Load Combination Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	WA	WS	WL	FR	TU, CR, SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	γ_p	**1.75**	**1.00**	-	-	**1.00**	**0.50/1.20**	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV (EH, EV, ES, DWDC only)	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME EVENT-I	γ_p	γ_{EQ}	**1.00**	-	-	**1.00**	-	-	-	**1.00**	-	-	-
EXTREME EVENT-II	γ_p	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	-	-	-	-	-	-	-	-	-	-	-

Where:

Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = accumulated locked-in effects resulting from the construction process, including the secondary forces from post-tensioning
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
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 structure

Table 4 shows the load factors (γ_p) for permanent loads (routinely the most critical for soil nail wall stability) in the current version of the LRFD AASHTO Specifications. The routine cases of load factors combination for soil nail applications are shown shaded.

4 LRFD METHODS IN SLOPE STABILITY ANALYSIS

Soil nail walls are typically analyzed and designed with methods based on limit–equilibrium principles. These methods follow similar procedures to those used in conventional global stability of slopes. However, there exists limited information on the application of the LRFD methodology to design of systems where global stability if of concern, according to the AASHTO LRFD Specifications (AASHTO 2002). Resistance factors currently recommended are $\phi = 0.85$ for slopes not supporting a structure, and $\phi = 0.65$ for slopes supporting or containing a structure. These values were simply derived as the inverse of minimum recommended factors of safety against stability. A reliability-based calibration of slope stability methods has not yet been performed. In slope stability problems, the weight of the soil acts as the driving force (for which a load factor $\gamma = 1$ is commonly adopted, which is conservative) and it also acts as a component of the resisting forces (for which the resistance factors described above are used).

TABLE 4: LOAD FACTORS FOR PERMANENT LOADS

Type of Load	Case	Load Factor γ_p	
		Maximum	**Minimum**
DC: Dead Load		1.25	0.90
DD: Downdrag		1.80	0.45
DW: Wearing Surface and Utilities		1.50	0.65
EH: Horizontal Earth Pressure	Active At-Rest	1.50 1.35	0.90 0.90
EL: Locked-In Load Effects	Locked-in Erection Stresses	1.00	1.00
EV: Vertical Earth Pressure	Overall Stability Retaining Walls and Abutment	1.00 1.35	N/A 1.00
	Rigid Buried Structure Rigid Frame Flexible Buried Structure other than Metal Box Culvert Flexible Metal Box Culvert	1.30 1.35 1.95 1.50	0.90 0.90 0.90 0.90
ES: Earth Surcharge		1.50	0.75

In soil nail walls, the mechanics are more complex because of the interaction between the nails and the soil. Additionally, as various failure modes involving different materials may occur, the consideration of simultaneous resistance factors is necessary. The calibration of the analysis method is necessary and requires that multiple resistance and load factors are evaluated.

4.1 LRFD Calibration

The process of assigning values to ϕ and γ is called *calibration*. This calibration can be done a number of ways. The simplest way is to "fit" LRFD factors to pre-existing safety factor used in the ASD practice. This ensures consistency with earlier design, but also fails to address explicitly the involved uncertainties. Another way is to estimate factors using the subjective opinion of experts. Depending on the experts, this may increase the confidence one has in resulting designs, but the estimates are not empirically validated against data. Today, LRFD factors are usually calibrated using empirical data and reliability-based design. This approach has two advantages: first, the approach is explicit and, second, resulting load and resistance factors are directly related to quantified levels of uncertainty.

Calibration using reliability-based design involves probabilistic concepts. Withiam et al. (1997) describe that basic calibration process in the LRFD methodology consists of the following four steps: Step 1: Evaluate the level of reliability in the existing ASD method and characterize it with a reliability index, β, Step 2: Assess the variation of reliability with different load combinations, structure geometry, and methods of predicting resistance, Step 3: Select a target reliability index, β_T, based on the margin of safety existing in current designs, and Step 4: Calculate the resistance factor ϕ consistent with β_T. Figure 4 suggests the uncertainties inherent in estimates of load, Q, and resistance, R, can be summarized as probability distributions. Load and resistance each have a mean or best estimate value, $\overline{Q}$ and $\overline{R}$, respectively, and for each there is some level of uncertainty about what the actual values of load and resistance are with respect to those best estimates. This uncertainty is reflected in a standard deviation of values about the mean σ. Thus, because the actual load can be larger than the average load, and the actual resistance lower than the average resistance, there is some probability that $R < Q$, and therefore that failure ensues. This is shown in Figure 5, which illustrates the corresponding probability distribution of the safety margin, $M = R - Q$. The area under this probability distribution in the interval $M \leq 0$ is the probability of failure, since $M = 0$ is the limiting state. This probability of failure is summarized in a *reliability index,* β, which is the number of standard deviations, in this case of safety margin, separating the mean value of the safety margin, $\overline{M} = \overline{R} - \overline{Q}$ from the limiting state $M = 0$.

In LRFD practice, Equation 1 is used as the design criterion, in which for geotechnical applications, the nominal values of load and resistance are taken as the means. The question for calibration is then, how to choose γ and ϕ_i such that an agreed upon level of reliability, reflected in an agreed upon value of the reliability index, β, can be achieved.

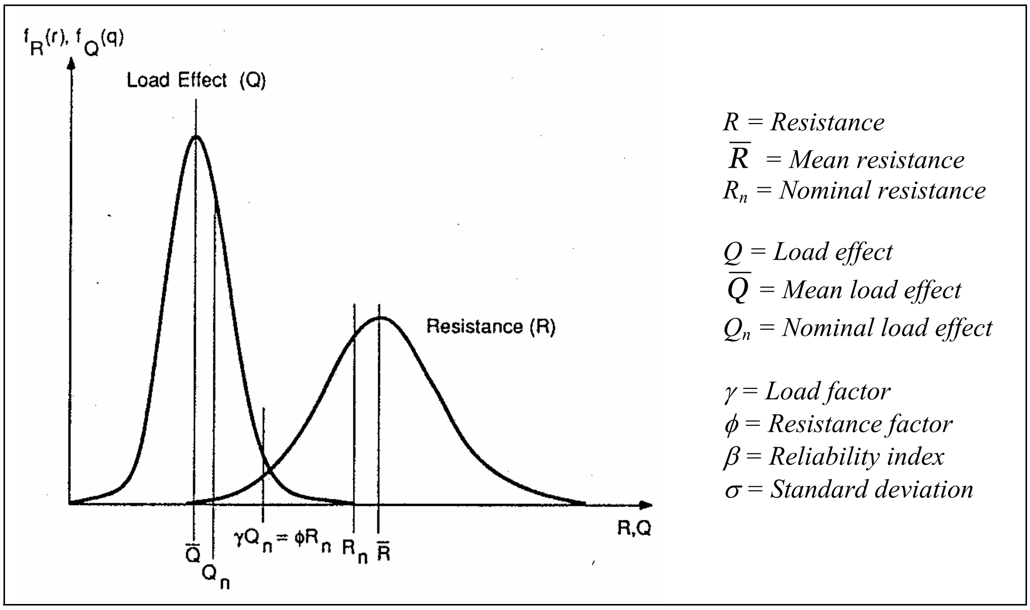

FIGURE 4: PROBABILITY DENSITY FUNCTION FOR Q AND R

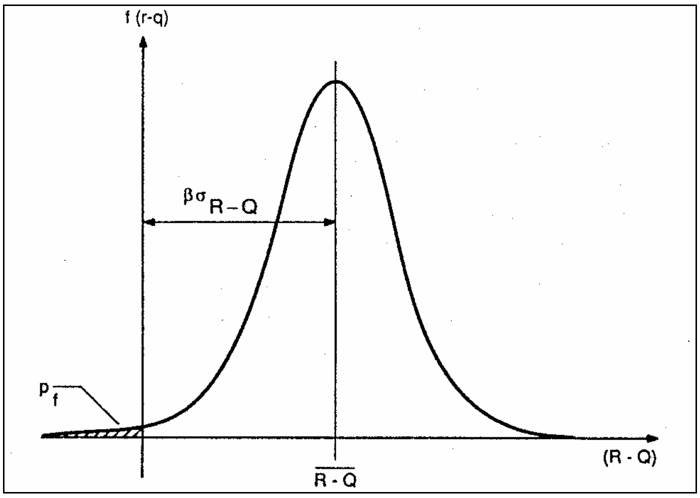

FIGURE 5: COMBINED PROBABILITY DENSITY FUNCTION

The simpler, and historically early approach to this question is to expand the expression for β as a function of the means and standard deviation of R and Q_i in a linear approximation about the means. This is called a *first-order second-moment* approach (FOSM), which uses a first-order or linear approximation at the means, and uses only first and second moment information about the uncertain quantities (means and variances). The advantage is that this approach results in a closed-form approximation for the special, but common, case of logNormal variation,

$$\phi = \frac{\left(\sum \gamma_i Q_i\right)\sqrt{\dfrac{1 + COV_Q^{\,2}}{1 + COV_R^{\,2}}}}{Q \exp\left\{\beta \sqrt{\ln[(1 + COV_R^{\,2})(1 + COV_Q^{\,2})]}\right\}} \tag{2}$$

in which COV is the *coefficient of variation*, defined as COV = (standard deviation)/(mean). The disadvantage of this approach is that the approximation may be poor for some cases, and the values

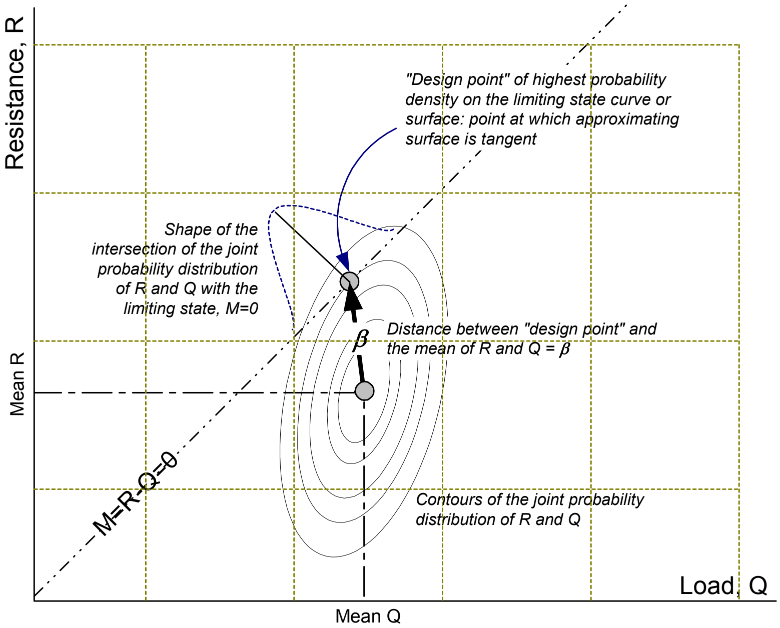

FIGURE 6: LIMITING STATE SURFACE.

calculated are not invariant to mechanically equivalent transformations of the limiting state (e.g., factor of safety FS = 1 vs. margin of safety M = 0) (Hasofer and Lind 1974). For example, previous NCHRP-sponsored work has found that the FOSM value of ϕ is systematically about 10 percent too low for deep foundations (Paikowsky et al. 2002).

Today, the accepted approach to calibration is using the so-called *first-order reliability method* (FORM) approach (Hasofer and Lind 1974). This method also approximates the expression for β as a function of the means and standard deviation of R and Q_i in a linear approximation, but does so about the most likely failure point on the limiting state surface rather than at the means of the uncertainty quantities (Figure 6). This approach has the advantage of being nearly invariant to transformations, but the disadvantage is that it needs an iterative solution.

5 CONCLUSIONS

The current direction for soil nail design is moving toward a load and resistance factor specification. This will provide added capability in setting specifications that are coherent with other geotechnical design procedures for shallow and deep foundations, and earth structures. Unlike those other uses of LRFD in geotechnical design, however, soil nail systems have varied and complex failure conditions, and thus require careful review and analysis of historical performance data. Nonetheless, to be more than just a restatement of current practice, the calibration of new LRFD norms for soil nail walls needs to be soundly based on statistical reliability analysis. The ongoing NCHRP Project 24-21 **"**LRFD Soil-Nailing Design and Construction Specifications,**"** is expected to contribute in this direction.

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