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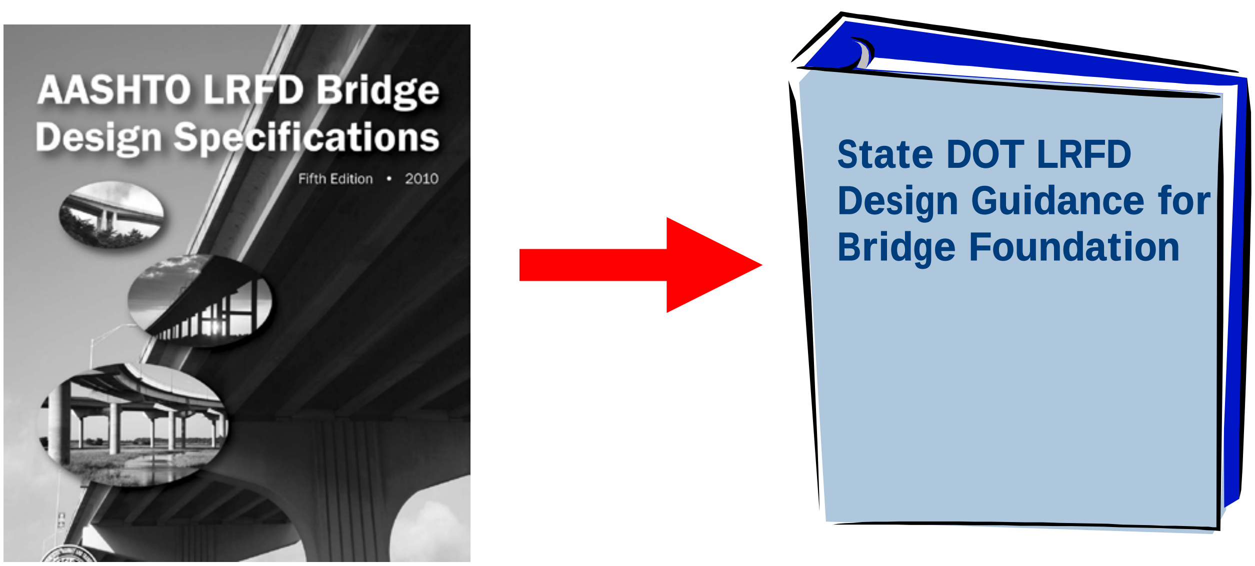
Implementation of LRFD Geotechnical Design for Bridge Foundations

Reference Manual



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16. ABSTRACT. This document is the reference manual for NHI training course No. 132083, both developed to assist State Departments of Transportation (DOTs) in the successful development of LRFD Design Guidance for bridge foundations based on the 2010 AASHTO LRFD Bridge Design Specifications and their local experiences. Initially, an LRFD Implementation Plan of six consecutive steps is discussed and the remainder of the manual presents recommendations to assist DOTS with implementation of these steps. The principal changes in the AASHTO design specifications from ASD to LRFD are presented. The DOTs have three options for selection of LRFD geotechnical design methods: adoption of AASHTO's LRFD methods or development of local LRFD methods by fitting to ASD or through reliability analysis of information collected at load test sites. The calibration methods and conditions for these three options are discussed to assist DOTs in implementation of these options. Procedures are furnished to evaluate and address the project site variability. Then, the three implementation options are evaluated and compared to assist DOTs with selection of the most appropriate option. The advantages of development of local reliability-based LRFD design methods over other implementation options are demonstrated. The implementation of AASHTO LRFD for geotechnical design for bridge foundations will lead to savings or to equivalent foundation costs compared with ASD methods. At the end, a roadmap for development of LRFD design guidance that consists of LRFD design specifications and delivery processes for bridge foundations is discussed.

17. KEY WORDS: AASHTO, LRFD, ASD, limit states, foundations, safety factor, resistance factor, calibration, conditions, reliability, load tests, design, construction, NCHRP Report 507

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		LENGTH		
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		AREA		
mm^2 m^2 m^2 ha km^2	square millimeters square meters square meters hectares square kilometers	0.0016 10.764 1.195 2.47 0.386	square inches square feet square yards acres square miles	in^2 ft^2 yd^2 ac mi^2
		VOLUME		
ml l m^3 m^3	millimeters liters cubic meters cubic meters	0.034 0.264 35.71 1.307	fluid ounces gallons cubic feet cubic yards	fl oz gal ft^3 yd^3
		MASS		
g kg tonnes	grams kilograms tonnes	0.035 2.202 1.103	ounces pounds tons	oz lb tons
		TEMPERATURE		
EC	Celsius	1.8 C + 32	Fahrenheit	EF
		WEIGHT DENSITY		
kN/m^3	kilonewton / cubic meter	6.36	poundforce / cubic foot	pcf
	FORCE and PRESSURE or STRESS			
N kN kPa kPa	newtons kilonewtons kilopascals kilopascals	0.225 225 0.145 20.9	poundforce poundforce poundforce / square inch poundforce / square foot	lbf lbf psi psf

FORWARD

State departments of transportation (DOTs) are at various stages of implementing the AASHTO Load and Resistance Factor Design (LRFD) specifications (AASHTO LRFD, 2007) for the design of bridge foundations. Some DOTs still use the allowable stress design (ASD) method (AASHTO Standard Specifications, 2002) for the geotechnical design of foundations. Other DOTs have developed LRFD design manuals that heavily refer to AASHTO LRFD without fully understanding the impact of these specifications on their design practices or the conditions they should adhere to when using them. Engineers from several DOTs have expressed an interest in having some form of guidance on how to implement the AASHTO LRFD design specifications with consideration of their local experiences. Consequently, the Federal Highway Administration (FHWA) has developed a new web-based National Highway Institute (NHI) training course called "Implementation of LRFD Geotechnical Design for Bridge Foundations." The goal of this training course is to assist DOTs in the successful development of LRFD Design Guidance for bridge foundations based on the 2010 AASHTO LRFD Bridge Design Specifications and with the consideration of their local experience. This manual is the reference manual for this course and includes seven chapters, with each chapter serving as a reference to a lesson in the training course. Note that this course supplements NHI Course 130082 "LRFD for Highway Bridge Substructures and Earth Retaining Structures" as it covers additional and more specific guidance to assist DOTs with implementation of LRFD.

It is hoped that that this course will help DOTs with their successful implementation of AASHTO LRFD and address their specific local LRFD implementation issues. Improvements to the AASHTO LRFD platform will continue in the future based on applied research studies, the results of additional load tests, and the experience of the highway community with implementation of LRFD. The LRFD reliability calibration process is a dynamic process: it allows for continued refinement of the resistance factors with more data. DOTs will need to keep updating their own LRFD design specifications to keep pace with future refinements and updates of AASHTO LRFD.

This course will continue to be updated in the future to address any errata, include new improvements, and to reflect new revisions in the AASHTO LRFD Geotechnical design specifications for bridge foundations.

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LIST OF SYMBOLS

AASHTO	American Association of State Highway and Transportation Officials
ASD	Allowable stress design
A_s	Shaft base area
A_s	Steel cross-sectional area
BOR	Beginning of redrive conditions
Bpf	Number of blows to drive 1 foot in the Standard Penetration Test
bpi	Number of hammer blows to drive the pile 1 inch
DD	Downdrag load
DOT	State Department of Transportation
D_{scour}	Scour depth due to combined effect of degradation scour, contraction scour, and local scour
COV	Coefficient of variation of bias resistance data and bias loads data in the reliability analysis; and coefficient of variation of the measured soil and rock properties across the site in the assessment of site variability.
$COVi_{nhernt}$	COV of measured soil and rock properties due to inherent (natural) variability in subsurface materials and testing methods
CPT	Cone penetration test
DSC	Differing site conditions
E_d	Developed hammer energy
EN dynamic formula	Engineering News dynamic formula
EOD	End of pile driving conditions
FHWA	Federal Highway Administration
FORM method	First Order Reliability method
FOSM method	First Order Second Moment method
FS	Factor of safety
f_y	Steel yield strength
GL	Geotechnical resistance losses
GWL	Groundwater level
IGM	Intermediate geomaterial
L, $L_{\mathbf{max}}$	Pile penetration length and the maximum length a pile can be safely to without damage
LL	Live load
P_f	Probability of failure
LFD	Load factor design
LRFD	Load resistance factor design
N_b	Number of hammer blows for 1 inch of pile permanent set
n	Number of data
N	Uncorrected SPT blow count
N_{60}	SPT blow count value corrected for hammer efficiency
$N1_{60}$	SPT blow count value corrected for both overburden and hammer efficiency effects
NHI	National Highway Institute
NCHRP	National Cooperative of Highway Research Program

P_n	Nominal structural resistance
QC and QA	Quality control and Quality Assurance
Q	Force effect on foundation or a foundation design (service) load
Q_f and Q_{fmax}	Factored load applied to a footing (excluding xowndrag loads) and the maximum factored load a footing can support
Q_s and Q_{smax}	Service load applied to a footing (excluding xowndrag loads) and the maximum service load a footing can support
q_u	Unconfined compressive strength of rock
RQD	Rock quality designation
RMR	Rock mass rating
R_m	Measured geotechnical resistance on a test foundation from load test
R_n	Nominal geotechnical resistance of a footing; predicted geotechnical resistance for a load test foundation
TRB	Transportation Research Board
SPT	Standard penetration test
S_u	Undrained shear strength of soils
s	Pile permanent set in inches per 1 hammer blow
WEAP	Wave equation analysis program
β	Reliability index
λ	Mean of bias resistance or load data
γ, γ_p, **and** γ_{ave}	Load factor, load factor for downdrag load, and average load factor
ϕ	Resistance factor
ϕ_{sta}	Resistance factor for the piles static analysis methods
ϕ_{dyn}	Resistance factor for the piles for the dynamic analysis and static load test methods
η	Load modifier

CHAPTER 1

INTRODUCTION

In 2000, the American Association of State Highway and Transportation Officials (AASHTO) recommended, and the Federal Highway Administration (FHWA) concurred, that all State departments of transportation (DOTs) should follow Load and Resistance Factor Design (LRFD) principles in the design of all new highway bridges by October 2007. To implement AASHTO LRFD design and construction specifications for bridges, each DOT should develop:

- An LRFD Design Manual based on the most updated version of the AASHTO LRFD Bridge Design Specifications and on the DOT's own design practices and specifications.

- An LRFD Construction Manual consistent with the DOT's LRFD Design Manual and based on the most updated version of the AASHTO LRFD Bridge Construction Specifications and on the DOT's construction practices and specifications.

- LRFD-based design and construction delivery processes consistent with the DOT's LRFD Design and Construction Specifications. These processes are often presented in the DOTs Design and Construction Manuals.

To assist DOTs in implementing the LRFD design platform, FHWA has developed several National Highway Institute (NHI) training courses and LRFD-based technical manuals, and has provided direct technical support to DOTs. Most DOTs have widely and quickly accepted and implemented the AASHTO LRFD Bridge Design Specifications (AASHTO, 2010a) for the design of bridge superstructures. This is not the case for the design of bridge foundations.

DOTs are at various stages of implementing the AASHTO LRFD Bridge Design Specifications for the design of bridge foundations. Some DOTs still use the allowable stress design (ASD) method for the geotechnical design of foundations (AASHTO Standards, 2002). Other DOTs have developed LRFD design manuals that heavily refer to AASHTO LRFD (AASHTO 2010a) without fully understanding the impact of these specifications on their design practices or the conditions they should adhere to when using them. Engineers from several DOTs have expressed an interest in having some form of guidance on how to implement LRFD. Consequently, the FHWA has developed a new web-based National Highway Institute (NHI) training course called "Implementation of LRFD Geotechnical Design for Bridge Foundation." The goal of this training course is to assist DOTs in the successful development of LRFD Design Guidance for bridge foundations based on the 2010 AASHTO LRFD Bridge Design Specifications and with consideration of their local experience. It is important to realize that this course supplements

NHI Course 130082 “LRFD for Highway Bridge Substructures and Earth Retaining Structures” (NHI, 2005) as it covers additional and more specific guidance to assist DOTs with implementation of LRFD.

This document is the reference manual for the training course described above and includes seven chapters, with each chapter serving as a reference to a lesson in the training course.

- **Chapter 2: LRFD Implementation Plan.** This chapter presents the steps needed for the development of LRFD Design Guidance for bridge foundations and briefly describes how these steps are addressed in Chapters 3 to 7 of this manual.

- **Chapter 3: Changes in the AASHTO Design Specifications from ASD to LRFD.** This chapter provides an overview of the content of AASHTO LRFD Section 10 and other sections in AASHTO LRFD that are referenced within Section 10, with emphasis on the following three principal changes between AASHTO ASD and LRFD design platforms: i) Incorporation of Limit State Designs; ii) Load and Resistance Factors to Account for Uncertainties; and iii) “New and improved Methods to Determine Foundations Loads, Displacements, and Resistances.”

- **Chapter 4: Calibration Methods for Geotechnical Resistance Factors.** This chapter describes the two calibration methods—by fitting to ASD, and through reliability analysis—to the extent needed by DOTs for the local calibration of their resistance factors, and then describes the methods employed to develop all the geotechnical resistance factors in the AASHTO LRFD Design Specifications.

- **Chapter 5: Calibration Conditions and Assessment of Site Variability.** This chapter describes conditions that the DOTs need to adhere to when they adopt AASHTO’s LRFD design methods and provides recommendations to the DOTs to consider when they develop local LRFD design methods. It concludes with recommendations to evaluate and address the project site variability.

- **Chapter 6: Selection of LRFD Geotechnical Design Methods.** This chapter evaluates and compares the options for selection of LRFD geotechnical design methods in order to assist DOTs in selecting and finalizing the most appropriate option. This chapter demonstrates the advantages of development of local reliability-based LRFD design methods over other implementation options.

- **Chapter 7: Development of the LRFD Design Guidance.** The manual concludes with general recommendations for development of LRFD Design Guidance that consists of LRFD design specifications and delivery processes for bridge foundations.

CHAPTER 2

LRFD IMPLEMENTATION PLAN

This chapter presents the sequence of steps recommended for the development of LRFD design guidance for bridge foundations and briefly describes how these steps are addressed in Chapters 3 to 7 of this manual. These steps constitute the LRFD implementation plan.

2.1 STEP 1: FORM LRFD IMPLEMENTATION COMMITTEE

Formation of this committee needs sponsorship from within the DOT. It should include members from all DOT offices involved with the design and construction of foundations (structural, geotechnical, and hydraulic), and possibly from the FHWA and academia, with defined roles and responsibilities and a schedule with commitment. This Committee will be responsible for:

- Development of state-specific LRFD Design Guidance for Bridge Foundations. To achieve this goal, the Committee needs to develop LRFD implementation plan based on the steps furnished in this chapter, and execute this plan based on the recommendations presented in Chapters 3 to 7 of this manual.
- Monitoring the implementation of LRFD. It is very important to capture the impact of LRFD on DOT practices, in terms of time, cost, and design procedures and practices. Lessons learned during implementation of the LRFD platform need to be documented and used in future refinements and improvements to the LRFD platform.
- Sponsoring and providing LRFD training and technical support. LRFD training with real LRFD examples should be offered to all personnel involved with the use of LRFD. Require that consultants and local agencies attend LRFD training before they perform LRFD design.
- Sponsoring needed LRFD research studies, and tracking results of relevant LRFD research studies conducted by others. Research needs can be identified during implementation of LRFD and by monitoring the implementation.

2.2 STEP 2: REVIEW KEY LRFD DESIGN REFERENCES

At a minimum, the following LRFD references should be reviewed to provide adequate background and direction for the implementation committee.

- AASHTO LRFD Section 10 and related sections referenced within Section 10 (e.g., AASHTO LRFD Sections 2 to 8 covering hydraulic issues, loads, and structural design). It is important to review the most updated and recent version of AASHTO LRFD (AASHTO 2007 with 2008, 2009, and 2010 Interims). The 2006 Interim Revisions to the AASHTO

LRFD Bridge Design Specifications contained the most significant changes to foundation design as compared with the AASHTO Standard Specifications and previous LRFD editions. A new article on micropiles (Article 10.9) was added in the 2008 AASHTO Interim Revisions. In the 2009 AASHTO LRFD interims, a new Article 10.5.4.2 "Liquefaction Design Requirements" was added with valuable information in the corresponding commentary article. In 2009, AASHTO released guide specifications for LRFD seismic bridge design (AASHTO 2009a) and guide specification and commentary for vessel collision design of highway bridges (AASHTO 2009b). The 2010 Interim Revisions to the AASHTO LRFD Bridge Design Specifications included many changes to AASHTO LRFD Section 10 that will be discussed more in subsequent chapters.

- References for development of AASHTO's resistance factors (Allen, 2005; Paikowsky et al., 2004) as will be discussed in subsequent chapters.
- Reference Manual for NHI Course 130082 (NHI, 2005), "LRFD for Highway Bridge Substructures and Earth Retaining Structures." Note that this course is currently based on the 2006 AASHTO LRFD interim revisions.
- FHWA's LRFD Design Examples (see http://www.fhwa.dot.gov/bridge/lrfd/examples.htm). Note that these examples are based on the 2004 AASHTO LRFD platform.
- The FHWA LRFD drilled shaft foundation manual (Brown et. al, 2010).
- The FHWA Soils and Foundations Manual (Samtani and Nowatzki, 2006).
- FHWA is the final stages of preparing two NHI courses on the seismic design of bridges and other structures. The reference manuals for these should be reviewed if available.
- The experiences of other DOTs that have developed LRFD Design Manuals (such as Florida, Arizona and Washington States). The experiences and efforts of some DOTs in implementing LRFD for foundation design are detailed in the 2009 Transportation Research Board (TRB) publication "Implementation Status of Geotechnical Load and Resistance Factor Design in State Departments of Transportation," which can be accessed online at http://onlinepubs.trb.org/onlinepubs/circulars/ec136.pdf.

2.3 STEP 3: IDENTIFY CHANGES NEEDED TO TRANSITION TO LRFD

The DOTs should review their current ASD design specifications against AASHTO Section 10 LRFD design specifications, and then identify changes needed to transition to LRFD. It is possible to begin by identifying the common foundation types, the typical geomaterials (sand, clays, soft and hard rocks) that support each foundation type, and the applicable extreme event limit states (seismic event, vessel impact, and check flood). Then continue with review of current ASD foundation design methods for the applicable limit states and compare them to those furnished in Section 10 of AASHTO LRFD design specifications. This review will result in the changes needed to transition to LRFD, which can be either:

- In full accordance with AASHTO LRFD Section 10. These changes will be either adopted from or be based on AASHTO LRFD specifications. This would be the case if the DOT decides to adopt AASHTO's LRFD design methods. Chapter 3 covers the changes in the AASHTO design specifications from ASD to LRFD. The AASHTO LRFD geotechnical resistance factors were developed based on reliability analysis of data collected at load test sites, calibration by fitting to ASD, and engineering judgment.
- Exceptions (deviations) from AASHTO Section 10 (deletions, additions or modifications to the contents of AASHTO Section 10). This would be the case if the DOT decides to develop LRFD design methods with locally calibrated resistance factors. Exception from AASHTO (e.g., Local calibration of resistance factors) is recognized by AASHTO. For example, Article 10.5.5.2.1 of AASHTO LRFD (2010a) allows the use of higher ϕ values developed locally if they are based on substantial successful experience. AASHTO LRFD Article 10.5.5.2.1 allows the use of higher ϕ developed locally if they are based on substantial successful experience OR statistical data with calibration (Load tests). The DOTs need to provide justifications for these exceptions from AASHTO LRFD based on their own long-term successful experience and engineering judgment, research results, and, or to address local issues not addressed by AASHTO.

A change in design platform is an excellent opportunity for moving away from 'business as usual' and making other improvements to practice. The DOTs should consider improvements of their geotechnical design practices at the same time as they're transitioning to LRFD.

2.4 STEP 4: SELECT GEOTECHNICAL LRFD DESIGN METHODS

Based on the previous step, DOTs have three options for the selection of LRFD geotechnical design methods: adopt AASHTO's LRFD methods; develop local LRFD methods by fitting to local ASD methods that have track records of long-term success, or to develop local LRFD methods through reliability analysis of information collected at load test sites.

Chapters 4, 5, and 6 are developed to assist the DOTs with development, evaluation, and comparison of these options. This would help the DOTs to select and implement the most appropriate LRFD geotechnical design methods. What is most appropriate may change through time. For example, if few load tests are initially available, a local reliability analysis is not appropriate. However, if more tests are obtained through time, local reliability analysis could become most appropriate and lead to the most efficient designs.

2.5 STEPS 5 AND 6: DEVELOP LRFD DESIGN SPECIFICATIONS AND DELIVERY PROCESSES

Based on the results of Steps 1 through 4, the DOTs will be ready for development of LRFD Guidance consisting of specifications and delivery processes for each type of their bridge foundations. The development of specifications should be based on Section 10 of the 2010 AASHTO LRFD Bridge Specifications and with consideration of the DOT’s ASD specifications. The delivery processes should be consistent with the developed LRFD specifications and with consideration of the DOT’s ASD delivery processes. Chapter 7 presents a roadmap to develop the contents of this guidance and describe roles and responsibilities of various groups in the DOT in the development.

CHAPTER 3

CHANGES IN THE AASHTO DESIGN SPECIFICATIONS FROM ASD TO LRFD

This chapter is developed to assist DOTs with identification of the required changes to move their practice from ASD to LRFD for the design of bridge foundations. It provides an overview of the content of AASHTO LRFD Section 10 and other sections in AASHTO LRFD that are referenced within Section 10, with emphasis on the following three principal changes between AASHTO ASD and LRFD design platforms:

- Incorporation of limit states designs
- Load and resistance factors to account for design uncertainties and assure desired safety
- New and improved methods to determine foundations loads, displacements, and resistances.

3.1 INCORPORATION OF LIMIT STATE DESIGNS

In contrast to the ASD platform, the concept of "limit state design" is an explicit and integral component of the AASHTO LRFD platform.

3.1.1 Overview of the AASHTO LRFD Limit State Design for Bridge Foundations

All possible structural and geotechnical failure modes for foundations present during the design life of the bridge are grouped into three distinct structural and geotechnical limit states. Note that these failures do not necessary correspond to "true failure" but rather to when a certain criterion is met or exceeded (per Eqs. 1 and 2 discussed later). The various limit states for bridge foundations are discussed in AASHTO (2010a) Sections 1 and 3, and in Articles 10.5.1 to 10.5.4:

- **Service limit states.** Failure modes are related to the function and performance of the bridge due to foundation under regular operating conditions. For example, bridge settlement caused by foundation settlement that exceeds the bridge tolerable settlement. In this case, failure means generated displacements exceeding the tolerable displacements. In the LRFD design, the bridge/foundation displacements under regular service conditions must be kept below the tolerable values.
- **Strength limit states.** Failure modes are the collapse or damage of the bridge or its foundation under loads applied continuously or frequently during its design life. In the LRFD design, the foundations must have adequate structural and geotechnical resistances to resist the loads the bridge is expected to experience during its life with an adequate margin of safety against damage or collapse.
- **Extreme event limit states.** Failure modes are the collapse of the bridge or its foundation

due to events that have a return period greater than the design life; for example, a major earthquake or flood, or vessel or vehicle collision. In the LRFD design, the foundations must have adequate structural and geotechnical resistances to withstand the extreme events the bridge may experience during its life without causing collapse of the bridge. The concern here is survival of the bridge and protection of life safety (some damage to the structure is allowable).

LRFD design for foundations requires that the summation of factored force effects is kept equal to or below the summation of factored geotechnical resistances for all applicable geotechnical limit states, and summation of factored force effects is kept equal or below the summation of factored structural resistances for all applicable structural limit states, as illustrated in the following equations:

$$\Sigma\, \eta_i\, \gamma_i\, Q_i \leq \sum \phi_i\, R_{ni} \text{ for all applicable geotechnical limit states} \tag{3.1}$$

and

$$\Sigma\, \eta_i\, \gamma_i\, Q_i \leq \sum \phi_i\, P_{ni} \text{ for all applicable structural limit states} \tag{3.2}$$

Where:

- $\sum$: summation for a failure mode (e.g., axial compression failure, excessive settlement, overturning) identified in the limit state
- $\mathbf{Q_i}$: is the force effect on the foundation (e.g., axial compression load) from a load applied on the bridge (e.g., dead load) and $\boldsymbol{\gamma_i}$ is the load factor for that load (e.g., dead load).
- $\mathbf{R_{ni}}$ = geotechnical resistance available to resist the force effect (e.g., shaft side or resistances) and $\boldsymbol{\phi_i}$ is its resistance factor
- $\mathbf{P_{ni}}$ = structural resistance available to resist the force effect (e.g., moment resistance) and $\boldsymbol{\phi_i}$ is its resistance factor
- $\boldsymbol{\gamma_i}$ **and** $\boldsymbol{\phi_i}$ will account for the uncertainties in the computation of each load component and resistance component.
- $\boldsymbol{\eta}$ is a load modifier relating to ductility, redundancy, and operational importance. In AASHTO LRFD, η values for superstructure design are furnished, but no specific guidance on the application of this factor to foundations is provided. For deep foundations, redundancy is addressed by reducing the geotechnical resistance factor, as will be discussed later.

AASHTO (2010a) Section 10 provides the design specifications for design of bridge foundations at all structural and geotechnical limit states:

- The structural and geotechnical resistances (or failure modes) that should be evaluated (e.g., bearing and sliding resistances for spread foundations at the strength limit) and their design requirements are discussed.
- Methods to compute foundation nominal geotechnical resistances (R_{ni}) (or displacements) and their resistance factors (ϕ_i) are furnished, which are needed to evaluate the foundation

factored geotechnical resistance ($\sum\phi_i R_{ni}$).

- Earth loads (e.g., downdrag, uplift) are discussed. Other loads are referred to Section 3.
- The load combinations are referred to Section 3, and the computation of the effect of these load combinations on foundations (Q_i in equations 3.1 and 3.2) is referred to Section 4. Note here that the effect of load, Q_i, needs to be computed in the design and then factored NOT to compute the effect of factored load.
- Factored structural resistances are briefly discussed, and the reader is referred (in most cases) to AASHTO (2010a) Section 5 for concrete foundation elements, Section 6 for steel foundation elements, and Section 8 for wood foundation elements.
- The influence of scour on foundation design is discussed (addressed more in Section 3.3.2.8 of this report). Scour at foundations should be investigated for two types of floods, as discussed in AASHTO LRFD Article 2.6.4.4.2:
    - **Design flood:** the flood of a 100-year event or an overtopping flood of a lesser recurrence interval. This type of flood needs to be considered under the service and strength limit states.
    - **Check flood:** a flood not to exceed a 500-year event or an overtopping flood of a lesser recurrence interval. This type of flood needs to be considered under the extreme event limit state.

3.1.2 Addressing All Applicable Structural and Geotechnical Limit States

Article 1.3.2.1 of AASHTO LRFD (2010a) reads: “All limit states shall be considered of equal importance.” This means that the service, strength, and the applicable extreme event limit states must always be checked during foundation design. Some of the implications of this requirement on DOTs practices are discussed next.

The DOTs should ensure close interaction and communication among the entire design and construction team (e.g., structural, geotechnical, hydraulic, and construction). The construction personnel should ensure that the design requirements for construction of foundations are met during construction. The roles of the structural and geotechnical engineers in the design phase should complement each other, as both structural and geotechnical resistances should be evaluated in the LRFD design of foundations, and the structural engineer is responsible for the development of foundation factored loads at various limit states needed in the evaluation of various geotechnical resistances. While the foundation factored loads and tolerable displacements are finalized by the structural engineer, the geotechnical engineer often computes the foundation displacements. Hence, close communication between structural and geotechnical engineers is needed to address this limit state and avoid costly overdesigns. There are several other common design issues that should be jointly addressed by the structural and geotechnical engineers, such as earth loads, use of the same design methods to evaluate both the structural and geotechnical limit states under lateral loading, and the service limit state.

The DOTs should explicitly address the service limit state in the foundation design to ensure that the generated displacements are less than the tolerable displacements. As discussed by Samtani (2008), overstress allowances (larger loads than estimated) provided in the AASHTO Standard Specifications (2002) were used by the structural engineers as a safety umbrella for not performing deformation-based analysis. The 2002 AASHTO Standard Specifications also allowed for some arbitrary reduction factors for the computed base resistance of drilled shafts to avoid conducting a settlement analysis. Geotechnical engineers often employed conservative ASD strength limit design methods with large safety factors to avoid conducting foundation settlement analysis. However, there are no overstress allowances or reduction factors in the AASHTO LRFD specifications (2010a) since the service limit is addressed explicitly.

The DOTs should explicitly address the drivability limit state for driven piles to ensure that the pile can be driven safely to the required depth or resistance without damage. Inclusion of a drivability analysis using wave equation analysis and dynamic testing is now prescribed in the LRFD specifications (Article 10.7.8). Failure to evaluate pile drivability is one of the most common deficiencies in driven pile design practice. Drivability analysis is not specifically addressed in ASD because the maximum allowable pile design loads, Q_{smax}, that a pile can support, were developed based only on pile structural capacity and by applying a large safety factor (FS) to maintain a low Q_{smax}. For example, the AASHTO Standard Specifications' recommendation for H-piles is $Q_{smax} = 0.25\ fyA_s$, where fy is the steel yield strength, A_s is the cross sectional area of the steel, and the safety factor is 4. The traditional conservative ASD approach evolved many years ago when tools were unavailable to estimate and measure pile driving stresses. The LRFD approach recognizes the now mainstream application of tools such as wave equation analysis and dynamic measurements for drivability analysis. In contrary to AASHTO Standard Specifications that recommends Q_{smax} values for different pile types, the AASHTO LRFD does not provide specific values for the maximum axial factored load a pile can support, Q_{fmax}. The Q_{fmax} in the LRFD can be determined in the design by meeting all axial compression strength limit states: geotechnical, structural under static loading, and drivability.

The DOTs should explicitly address the AASHTO's LRFD extreme limit applicable to them. When compared to AASHTO Standards, there have been significant changes in the AASHTO LRFD design specifications for the extreme event design methods that would impact the design of foundations. In 2007 and 2008, AASHTO approved significant changes to the LRFD seismic design specifications. In the 2009 AASHTO LRFD interims, a new Article 10.5.4.2 "Liquefaction Design Requirements" was added with valuable information in the corresponding commentary article. The return period was increased from 500 years to 1,000 years, which led to changes in the seismic zones. Current AASHTO LRFD seismic design specifications are considered to be "force based," where the bridge is designed to have adequate strength (capacity) to resist earthquake forces. In 2009, AASHTO released the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2009a) which are displacement-based specifications developed to address the seismic deflection of the bridge rather than forces, and a guide specification and commentary for vessel collision design of highway bridges (AASHTO 2009b).

This change to explicitly address all limit states has the potential to lead to more accurate and economical design when compared to ASD practices. A few examples are discussed next.

- AASHTO LRFD (2010a) does not promote or endorse using conservative strength limit analysis methods to compensate for not addressing the service limit. As a result, greater geotechnical capacity for foundations can be used. For structures where the service limit does not control the design, more economical foundations are the result.
- LRFD allows piles to accept greater axial loads than ASD if drivability is investigated in the design phase. The upper limit for Q_{fmax} is the pile's factored axial structural resistance, for example for H-piles and assuming severe driving conditions, Q_{fmax} is $0.5fyA_s$. With this Q_{fmax}, H-piles support larger design loads when structural capacity controls, as it might the case for H-piles seated on hard rocks or with friction piles when static load tests are considered.
- AASHTO LRFD (2010a) allows for consideration of both the side and tip resistances of shafts installed in rocks as long as the requirements for construction methods and quality control assumed in the design are met. It has been a common practice for DOTs to ignore or use a smaller portion of the end-bearing resistance in the design of drilled shafts to help ensure that settlement does not exceed tolerable values. For high quality shafts, this results in underutilization of their capacity, and less economical foundations.

3.2 LOAD AND RESISTANCE FACTORS TO ACCOUNT FOR DESIGN UNCERTAINTIES

In the ASD design equation, the uncertainties or in the calculations of loads and resistances are accounted for through a single factor of safety. In the LRFD, the load factor (γ) accounts for uncertainties associated with the calculated design load, and the resistance factor (ϕ) accounts for uncertainties associated with the calculated nominal geotechnical resistance. The sources of uncertainties are separated to make it simpler and more rational to evaluate load and resistance factors based on scientific methods. In reliability-based calibrations, the load and resistance factors are tied together through a target reliability index, β, which is a quantification of the probability of failure, P_f, which is equivalent to not meeting the performance intended by the limit state as defined in equations 3.1 and 3.2.

Thus, the safety factor in ASD is replaced in LRFD with the reliability index, β, and load and resistance factors; design loads in the ASD are replaced with factored loads; and allowable capacities are replaced in the LRFD with factored resistance. Some DOTs employed both factored loads in the load factor design (LFD) *structural* design of foundations and unfactored loads in the ASD *geotechnical* design of foundations. With LRFD, factored loads should be employed in both the structural and geotechnical design of foundations.

3.2.1 Load Combinations, Load Factors, and Resistance Factors at Various Limit States

Section 3 of AASHTO (2010a) presents 12 load combinations that could act on the bridge during its design life and describe various types of loads. For each load combination a unique combination of loads is specified for each limit state and a load factor is assigned to each load type (see Tables 3.1 and 3.2). These combinations of loads and their load factors are needed to compute the maximum effects of factored loads ($\Sigma\gamma_i Q_i$ in Equations 3.1 and 3.2) on all components of the bridge structure. The load combinations relevant to bridge foundations are:

- **Strength Limit State.** The Strength I, II, III, IV, and V load combinations need to be considered.
- **Service Limit State.** Only the Service I load combination is needed to evaluate this limit for foundations.
- **Extreme Event Limit States.** Extreme events are considered one at a time. The following events may or may not be relevant:
  - Extreme Event I. Load combination including earthquake (EQ)
  - Extreme Event II. Load combinations including ice load (IC), collision with vehicle (CV), or collision by vessel (CT), Or
    - Check flood. Consider the strength limit load combination in the design, but with a load factor of 1 assigned to all loads.

Overall stability should theoretically be addressed under the strength limit state because it is the shear strength that is being evaluated and the consequence of failure is global instability. However, it is investigated under the service limit state (Article 11.6.2.3) because soil weight appears on both the load and resistance sides of the equation and the analytical consequence is complex. As a result, current slope stability design methods and programs do not allow the loads to be factored. Therefore, for overall stability, ϕ is computed through calibration by fitting as 1/FS since the load factor is 1 for the service limit state.

Table 3.1. AASHTO LRFD Load Combinations and Load Factors at Various Limit States (after 2010 AASHTO LRFD, Table 3.4.1-1; see AASHTO LRFD Section 3 for definitions of all abbreviations)

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*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	γ_p	—	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	γEQ	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}	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

Table 3.2. AASHTO LRFD Maximum/Minimum Load Factors for Permanent Loads (after AASHTO 2007, Table 3.4.1-2)

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.50	0.90
ES: Earth Surcharge		1.50	0.75

For the service and extreme event limit states, the changes to design procedures are minor since FS = 1 in the ASD design equation, and $\phi = \gamma = 1$ in the LRFD design equation for most resistances and loads considered with these two limit states. This means that the design procedures for these limit states under ASD and LRFD would be similar. The most significant changes introduced with the LRFD platform are for the strength limit as discussed next.

3.2.2 Resistance Factors at the Strength Limit

The AASHTO LRFD geotechnical resistance factors at the strength limit (2010a) were developed based on reliability analysis of data collected at load test sites, calibration by fitting to ASD, and engineering judgment. Reliability calibration of AASHTO's resistance factors is considered mainly for the axial compression resistance determination methods of a driven pile and a drilled shaft at the strength limit so these methods could be impacted by the transition from ASD to LRFD and Chapters 4 to 6 is focused on.

3.2.3 Load Combinations and Factors at the Strength Limit

For the strength limit, five load combinations with maximum and minimum load factors (Tables 3.1 and 3.2) need to be considered in the design. Hence, the procedure in which the loads are combined in the LRFD equation for the strength limit and compared to resistances is significantly different from the ASD design procedure as illustrated next.

Table 3.3 describes the conditions for which the five strength limit load combinations should be considered in LRFD design. The Strength I load combination is the most common and will be applied for most routine short-span bridges, while the Strength IV load combination should be considered for long-span bridges (span length exceeding 200 feet). Strength I or IV load combinations must be always considered in the design. Additionally, if the bridge may be exposed to wind up to 55 mph during bridge design life, then Strength V load combination should be considered in the design, and if wind may exceed 55 mph during bridge design life, Strength III load combination need to be also considered in the design.

The load factors for the same load type vary among the different load combinations. For example, the load factor for live loads (LL) varies from zero in the Strength III and Strength IV load combinations to 1.35 in the Strength II and V load combinations to 1.75 in the Strength I load combination. As will be discussed in chapter 4, Strength I load factors are considered in the calibration of the geotechnical resistance factors, where the dominant loads are dead load (DL, load factor 1.25) and live load (LL, load factor 1.75), with an average load factor of around 1.4 (see Table 3.3). These resistance factors should be employed in the design with all the other strength limit load combinations. As discussed in Table 3.3, the load factors of other load combinations (Strength II, III, IV, and V) are selected to reflect the certainty, importance, and weight of these loads and to generate overall reliability with loads close to those considered under the Strength I load combination. This may explain why the average load factor for some of these load combinations is close to 1.4, similar to that used with the Strength I load combination (see Table 3.3). Consequently, resistance factors calibrated using the Strength I load combination are believed to be reasonable for use with other strength limit load combinations, but this may be changed in the future.

Table 3.3. Descriptions of the Strength Limit Load Combinations

Strength Limit	**Description**	**Load factors**
I	Normal use of short span bridge without wind. The basic and most common group.	Load factor, γ= 1.25 for Dead load (DL) and 1.75 for Live loads (LL), with an average load factor, γ_{ave}, of 1.4 for DL/LL ratio of 2 to 3.
II	For use with specified design vehicles- No wind.	As with Load Combination I but with γ= 1.35 for LL because of the more certainty of this load than with Load Combination I.
III	When Bridge is exposed to wind> 55 mph.	γ=0 for LL (high winds prevent the presence of live loads on the bridge), and 1.4 for the wind load (the most dominant).
IV	For long spans (>250 ft) with DD/LL ratio > 7.	γ = 0 for LL and 1.5 for DL, leading to γ_{ave} = 1.35 for a DL/LL ratio of 9.
V	For normal use (as Strength I) but with wind speed up to 55 mph.	Smaller γ for LL (1.35) than in Strength I Load Combination because wind load is considered in this load combination

For permanent loads, AASHTO LRFD provides maximum and minimum load factors (see Table 3.2). <u>For live loads, maximum load factors are provided (see Table 3.1), but a minimum load factor of zero can be considered for these loads.</u> Maximum and minimum load factors are employed in the strength limit to maximize the force effect for the failure mode investigated, either by increasing the loads that contribute to this failure (e.g., consider maximum load factors with the vertical loads in the evaluation of bearing failure for shallow foundations), or decreasing the loads that will increase the resistances to this failure (e.g., consider minimum load factors with the vertical loads in the evaluation of sliding failure for shallow foundations).

To maximize the force effect for any strength limit's geotechnical resistance, first determine the applicable load combinations that should be evaluated in the design (based on Table 3.3). Then, for each load in the load combination, consider either maximum or minimum load factors. If you are unsure of which to use, try both the maximum and minimum load factors in the design. The consideration of all applicable load combinations with maximum and minimum load factors would lead to the identification of the critical load combination with the maximum force effects. The Reference Manual for NHI Course 130082 (NHI, 2005) provides good examples of how to maximize the force effect in the evaluation of various geotechnical and structural resistances for foundations.

The need to consider various combinations of loads with maximum and minimum load factors in the evaluation of strength limit resistances represents a major difference from ASD design, in which only one load combination (Dead loads+ Live Loads) is considered in the geotechnical design of foundations. This concept is introduced in the LRFD platform to account for all possible combinations of loads that may act on the structure during its design life.

3.3 NEW AND IMPROVED METHODS TO DETERMINE LOADS, RESISTANCES, AND DISPLACEMENTS

The governing LRFD and ASD design equations at each limit state are:

$$\text{LRFD: } \Sigma\gamma_i\, Q_i \leq \sum\phi_i\, R_{ni} \tag{3.3}$$

$$\text{ASD: } \Sigma Q_i \leq \sum R_{ni}/FS_i \tag{3.4}$$

Although foundation design load (Q_i) and nominal resistance (R_{ni}) are used in both the ASD and LRFD platforms, methods to compute them continue to be improved and updated in the LRFD platform. By contrast, the last technical update to the AASHTO Standard Specifications was in 1998, with no plans for future updates. Improvements to the methods to compute foundation design load (Qi) and geotechnical nominal resistance (R_{ni}) would be applicable to the ASD platform if updates to the Standard Specifications were to be continued.

3.3.1 Examples of Improved AASHTO Methods to Determine Loads

- The truck loading model (HL-93) was adopted due to the increase in live loads carried by trucks. The increased live load from HS-20 (ASD) to HL-93 (LRFD) is a pure loading issue that would be applicable to ASD if updates to the Standard Specifications were to be continued.
- In AASHTO LRFD, there are new methods to estimate earthquake and vessel collision loads (AASHTO 2009a and 2009b) that would be applicable to ASD if updates to the Standard Specifications were to be continued.
- According to AASHTO LRFD Articles 3.11.8 and 10.7.3.7 (AASHTO LRFD, 2010a), downdrag (DD) is an axial compressive load that should be considered in the design of

drilled shafts and driven piles at all limit states using a load factor of $\gamma_{p.}$ DD load is computed as the nominal side geotechnical resistances of the soil layers located in and above the lowest layer contributing to downdrag (see AASHTO Article 10.7.1.6.2). The total factored axial compressive load per pile/shaft is calculated as $\Sigma\gamma_i Q_i + \gamma_p$ DD; where $\Sigma\gamma_i Q_i$ is the factored axial load applied to the top of the pile from the superstructure and γ_p DD is the factored downdrag load. In the ASD platform at the strength limit, DD effect is treated by considering it as a negative resistance, while in the LRFD, DD is treated as an additional load and lost resistance (zero resistance as will discussed later). If the ASD specifications were to continue to be updated, DD will be treated as an additional load (total axial compressive load per pile/shaft would be calculated as $\sum Q_i$ + DD) and lost resistance.

3.3.2 Improved Methods to Determine Foundation Displacements and Resistances

AASHTO LRFD covers popular methods to calculate the foundation nominal geotechnical resistances, R_n, and displacements that have track records of long-term success. For example, the shaft nominal axial unit tip resistance in rocks can be computed as $R_n = 2.5A_bq_u$, where A_b is the shaft base area and q_u is the unconfined compressive strength of the rock mass. The three types of resistance and displacement determination methods discussed in AASHTO are:

Static analysis methods. These methods have two components: a) soil/rock strength design properties collected in the design phase from the subsurface exploration program (e.g., q_u), as discussed in AASHTO LRFD Article 10.4; and b) analytical expression used to calculate R_n (e.g., $R_n = 2.5A_bq_u$) or displacement. These methods are discussed in AASHTO LRFD Articles 10.6 to 10.9 for different foundation types.

Field dynamic analysis methods. These methods are used with driven piles to determine their axial static compression geotechnical resistance, R_n, in the field using a) information collected during driving the pile (e.g., hammer developed energy and number of blows to drive 1 inch), and b) analytical model. These methods are discussed in AASHTO LRFD Article 10.7, and include dynamic testing with signal matching (referred to here as dynamic load test), wave equation analysis methods, and the FHWA modified Gates and Engineering News (EN) dynamic formulas.

Static load testing. With this design method, foundation geotechnical resistance and displacement are measured directly in the field. This method is described in AASHTO LRFD Articles 10.6 to 10.9 for different foundation types.

3.3.2.1. AASHTO LRFD Article 10.4: Soil and Rock Properties

This Article was improved and expanded in AASHTO LRFD (2010a), when compared to AASHTO Standards, to emphasize the importance of geotechnical investigation and testing work in geotechnical design. It describes the determination and selection of soil/rock properties needed for the design and construction of foundations. Recommended AASHTO guidance for subsurface investigation programs includes the location, number, and depth of subsurface borings, and DOTs need to adhere to this guidance. The level of subsurface exploration in AASHTO LRFD Table 10.4.2-1 should be considered a **minimum** and should be increased based on past experience, the degree of site variability, and the importance of the structure. The laboratory and in situ tests for the determination of the design geotechnical properties for various geomaterials are discussed. Geophysical testing methods for soils and rocks are discussed. Article 10.4.6 discusses the selection of deformation and strength properties for soil and rock mass. SPT-N values (from the Standard Penetration Test) should always be corrected for hammer efficiency (N_{60}) and sometimes for overburden $N1_{60}$. Rock classification is based on rock mass rating (RMR), rather than solely on rock quality designation (RQD) as in older ASD methods. Erodability (scourability) of rock mass is also discussed in Article 10.4.6. DOTs need to know that AASHTO LRFD allows for consideration of local experience and specific geology in the selection of soil and rock properties.

3.3.2.2 AASHTO LRFD Article 10.5.5: Tolerable Movement and Movement Criteria for Foundations

Vertical and lateral displacements and rotation of the foundation lead to movements of the bridge superstructure at critical locations (e.g., at the girder seat).

In contrast to AASHTO standards (2002), specific tolerable movements are not provided in the AASHTO LRFD (2010a). Both AASHTO LRFD (Article 10.5.5) and the FHWA Soils and Foundations Workshop Manual (2006) suggest the development of project-specific movement criteria for foundations as a function of structural tolerance of the bridge, the tolerance of the structures around the bridge (the approach slab), and rideability, economy (cost of future maintenance and repair), safety (clearance), and aesthetics. The DOTs should establish final guidelines for movement criteria that are appropriate to their bridges, field conditions, and design requirements.

AASHTO LRFD (2010a) indicates that transient live loads may be omitted in the time-dependent settlement analysis of foundations bearing on cohesive soils. Note also that the goal of the settlement analysis should be to determine the settlement of the bridge that would impact its performance, not the settlement of the foundation. Therefore, only certain loads should be considered in the estimation of the bridge settlement, not all loads that lead to settlement of the foundation.

For the service limit, the force effect can be defined as the foundation displacement generated by the foundation design load (OR the foundation design load), and the nominal resistance as the tolerable displacement (OR the foundation design load that would generate the tolerable displacement). In both cases, tolerable movement values, as discussed in AASHTO LRFD Article 10.5.2, and estimation of the foundation displacement, as discussed in AASHTO LRFD Articles 10.6 to 10.9 for different foundation types, are needed.

3.3.2.3 AASHTO LRFD Article 10.6: Spread Footings

- **Article 10.6.1. General Considerations.** For footings on soils, a uniform bearing pressure and effective dimension (B′ and L′) should be considered to evaluate the bearing resistance and settlement.
- **Article 10.6.2. Service Limit State Design.** This article covers the methods used to estimate settlement of footings on soils and rocks as well as overall stability. Methods of estimating bearing resistance at the service limit state are also discussed.
- **Article 10.6.3. Strength Limit State Design.** Geotechnical resistances needing evaluation include the bearing resistance of soils and rocks, eccentricity, and sliding. The overturning stability in the ASD platform has been replaced with the eccentricity limit under LRFD. Eccentricity provisions are different from those in ASD because they are based on factored loads.
- **Article 10.6.4. Extreme Event Limit State Design.** Geotechnical resistances needing evaluation include bearing capacity, eccentricity, sliding resistance, and *overall stability.*

3.3.2.4 AASHTO LRFD Article 10.7: Driven Piles

- **Article 10.7.2. Service Limit State Design.** Addresses settlement (including settlement due to downdrag loads), horizontal displacement, and lateral squeeze.
- **Article 10.7.3. Strength Limit State Design.** Covers the following geotechnical resistances: axial compression, uplift, and lateral resistances of a single pile and a group of piles. It also briefly describes the structural resistances (axial, bending, and shear) of steel, concrete, and timber piles.
- **Article 10.7.4. Extreme Event Limit State Design.** The nominal resistances considered are similar to those for the strength limit.

There have been significant changes in the AASHTO LRFD specifications for driven piles (2010a) as compared to the ASD specifications. The p-y method is now specified for lateral loading analysis. The Nordlund-Thurman method is now included in the static analysis methods. Requirements for drivability analysis are discussed (see Article 10.7.8). New diagrams for computing settlement of pile combinations are provided. New articles have been added: Article 10.7.3.2, "Pile Length Estimate for Contract Documents," and Article 10.7.6, "Determination of Minimum Pile Penetration." *Other changes between ASD and LRFD for driven piles are discussed in chapters 5 and 6.*

3.3.2.5 AASHTO LRFD Article 10.8: Drilled Shafts

Single and group shaft resistances are discussed in this article. The geotechnical and structural resistances covered are similar to those for piles (except without drivability analysis). AASHTO LRFD recommends the use of the Strain Wedge model in the lateral loading analysis of large-diameter and relatively short, shafts that have a tendency to rotate rather than bend. Design methods for cohesive and granular intermediate geomaterials (IGM; materials between soils and rocks) are now included. The most recent O'Neill and Reese method (1999) replaces the Reese and O'Neill method (1988) for soils.

3.3.2.6 AASHTO LRFD Article 10.9: Micropiles

This article was added in the AASHTO 2007 Interim Revisions. Its technical content is mainly based on the FHWA micropile design guidance (2005).

3.3.2.7 AASHTO LRFD Appendix A of Section 10

This appendix discuses the seismic analysis and design for foundations. See Article 3.1.2 of this manual for other AAASHTO references on this topic.

3.3.2.8 Nominal Geotechnical Resistance Losses

Only the foundation nominal geotechnical resistance, R_n, that would be available to support the applied foundation loads during the bridge entire design life should be considered in the design ($\Sigma\gamma_i Q_i \leq \sum\phi_i R_{ni}$). Hence, R_n should not include resistances that may not be available during the design life of the structure to support the applied loads. The sources of geotechnical resistance losses (GL) that should not be included in determination of R_n are downdrag, scour, liquefaction, and future increases of groundwater level (GWL). These will decrease the available foundation geotechnical axial and lateral resistances at all limit state, thus leading to longer deep foundations. Note that a zone within the soil layer will contribute to only one type of loss:

downdrag, scour, or liquefaction. The GWL expected over the life of the structure should be considered in the geotechnical design of foundations. With the static analysis methods, R_n can be estimated directly without the need to estimate GL. With the pile field axial design methods, estimation of GL is necessary to determine the required axial geotechnical resistance the pile needs to be driven to in the field. In this case, future increases in the GWL beyond the level at the time of driving the pile should be considered in the estimation of GL. Other sources for GL are discussed next.

Downdrag (AASHTO Articles 10.7.3.7 and 3.11.8). Assume zero axial resistance (R_n=0) in and above the lowest layer contributing to downdrag. To compute R_n below the depth contributing to downdrag, consider only the foundation base and sides resistances of the soil layers located below the lowest layer contributing to downdrag. The GL is the nominal side geotechnical resistances of the soil layers located in and above the lowest layer contributing to downdrag (same as the DD loads). Hence, according to AASHTO LRFD, downdrag effect at ALL axial limit states should be applied twice: as an additional load (as discussed previously) and as an additional lost nominal geotechnical resistance (the loss is applicable to the geotechnical limit states). At the service limit, the DD effect will increase foundation settlement due to additional loads and lost geotechnical resistance. To address the DD effect increase the length of deep foundations for the axial compression geotechnical limit states and increase the size (diameter) of the deep foundation for the structural axial compression limit states.

Scour. The consequences of changes in foundation conditions (e.g., removal of scoured soil layers) resulting from the design flood (100- year event) for scour shall be considered at the strength and service limit states (AASHTO Articles 3.7.5, 2.6.4.2, 10.7.3.6). The consequences of changes in foundation conditions due to scour resulting from the check flood (500- year event) for bridge scour and from hurricanes shall be considered at the extreme event limit states (AASHTO Articles 2.6.4.2, 3.7.5., 10.5.5.3.2, and 10.7.4). With these two types of floods, identify the scour depths due to degradation scour and due to contraction scour, both occur over a large area, and the scour depth due to local scour that occurs in a limited area around the pile/abutment (Hannigan et al., 2005). Add these three depths to determine the lowest depth, D_{scour}, for scour conditions. Assume zero axial resistance, R_n for all soil layers that will be lost due to scour (along D_{scour}). For computation of R_n for the soil layers located below the scoured soil layer (below D_{scour}), assume zero vertical effective stresses ONLY along the depth of scour due to degradation and contraction, but not along the lower depth due to local scour. In combination with extreme event limit states *EQ*, *IC*, *CV*, or *CT*, the commentary AASHTO article 3.4.1 suggests to consider only the scour due degradation or one-half of the total scour. AASHTO Article 2.6.4.2 recommends to place the bottom of spread foundation below the check flood scour elevation, and to place top of deep foundation below the estimated contraction scour depth.

Liquefaction. In the 2009 AASHTO LRFD interims, the new Article 10.5.4.2 "Liquefaction Design Requirements" was added to the code specifications with valuable information in the corresponding commentary article. A thorough discussion of the assessment and consideration of liquefaction in the design of drilled shafts (also applicable to driven piles) is described in chapters 12 and 16 of the new FHWA Manual on Drilled Shafts (Brown et. al., 2010). If the assessment of liquefaction potential identifies subsurface zones where liquefaction is likely, the following principles will govern the design:

- Assume zero axial resistance (R_n=0) in and above the liquefied zone. To compute R_n below the liquefied zone, consider only the foundation base and sides resistances of the soil layers located below the liquefied zone. Deep foundations must, therefore, be sufficiently deep to develop adequate resistance in the geomaterials located below the zone of liquefaction.
- Consider a lower lateral resistance value for the liquefied soil layer. The p-y curve should be selected based on the soil strength properties corresponding to the liquefied state.
- Post-liquefaction settlement would cause downdrag, which would increase foundation axial compression loads (see AASHTO LRFD commentary to Article C3.11.8) and reduce the available axial compression geotechnical resistance. The downdrag forces due to liquefaction should not be combined with downdrag considered in the strength limit.
- Consider the horizontal forces on the foundation due to the lateral spreading (movements) of a sloped soil mass due to liquefaction. Deep foundations may be designed with the intent to restrain the lateral movement of a soil mass during earthquakes to prevent an overall stability (or soil slope) failure and prevent any damage to the deep foundation.

CHAPTER 4

CALIBRATION METHODS FOR GEOTECHNICAL RESISTANCE FACTORS

This chapter describes the two calibration methods for geotechnical resistance factors—fitting to ASD, and reliability analysis—to the extent needed by DOTs for the local calibration of their resistance factors, and then describes the methods employed to develop AASHTO's geotechnical resistance factors.

Both calibration by fitting and reliability analysis discussed herein are demonstrated for the axial compression resistance determination methods at the Strength I load combination but the general principles presented in this section can be used to calibrate any kind of resistance or displacement determination method with any load combination.

4.1 CALIBRATION BY FITTING TO ASD METHODS

A simplified version of the governing LRFD and ASD design equations at the strength limit for bridge foundations is

$$\text{LRFD: } Q_f \leq \phi\, R_n\,; \;\text{ASD: } Q_s \leq R_n/FS \tag{4.1}$$

Where Q_f and Q_s are, respectively, the summation of factored loads and unfactored loads, and R_n is the summations of all nominal (or ultimate in the ASD) resistances assuming similar ϕ for various sources of resistances.

This calibration requires the factor of safety (FS) for the ASD resistance determination to be calibrated and the average weighted load factor, γ_{ave}, defined as the mean ratio between the factored loads (used in LRFD design) and the unfactored loads (used in ASD design, using a load factor of 1 for each load), or $Q_f = \gamma_{ave}Q_s$. The same values of loads should be used to compute the factored and unfactored loads. Resistance factor from calibration by fitting is computed as $\phi = \gamma_{ave}/FS$, which leads to factored resistance = γ_{ave} × allowable ASD capacity. This implies that the factored resistances and factored loads in LRFD design are increased by the same ratio from the service loads and allowable capacity used in ASD design. With this calibration, the outcomes of the calibrated LRFD geotechnical design method will be similar to the results of the ASD geotechnical design method and the true margin of safety will remain unknown. Some ASD methods employed by DOTs provide allowable design loads (or allowable capacity) directly, rather than estimating it as the nominal resistance divided by the safety factor. In this case, a reasonable FS can be assumed for estimation of the nominal resistance from these methods, but the end result for the calculated factored resistance will be independent of any selected FS value.

For the axial compression resistance determination methods, at the strength limit, with a load factor of 1.25 for dead loads (DL) and of 1.75 for live loads (LL), γ_{ave} can be computed as γ_{ave} = [1.25(DL/LL) + 1.75]/[(DL/LL) + 1], and γ_{ave} is approximately 1.4 for a typical DL/LL ratio of 2 to 3. Note that the DL/LL ratio could vary from State to State depending on a State's typical bridge types and span lengths, and this may lead to a slightly different γ_{ave} value.

Example. For an allowable stress design (ASD) method, the safety factor is 2.8 and its prediction of the allowable resistance for a foundation at depth of 20 ft is 300 kips. With calibration by fitting, the calibrated resistance factor is 1.4/2.8= 0.5, and the equivalent factored resistance is 1.4 x 300= 420 kips.

4.2 RELIABILITY ANALYSIS OF DATA COLLECTED AT LOAD TEST SITES

Two types of calibrated resistance determination methods (called also design methods) will be discussed in this manual:

I. Static analysis methods (resistance determined in the design phase**).** For example, the shaft nominal axial tip resistance in rocks is computed as $R_n = 2.5A_bq_u$, where A_b is the shaft base area and q_u is the unconfined compressive strength of the rock mass. These methods have two main components: a) soil/rock strength properties collected from a subsurface exploration program (e.g., q_u) during design, and b) analytical models (or equations) used to calculate R_n.

II. Field dynamic analysis methods (resistance determined in the field). These methods are used with driven piles to determine their axial static compression geotechnical resistance, R_n, in the field either at the end of driving (EOD) conditions or, after sufficient time has passed, at the beginning of redrive conditions (BOR) conditions (See Table 4.1). Setup leads to an increase in pile resistance over time between EOD and BOR conditions, and this time is called restrike time. The resistance factors for these methods can be calibrated either at EOD conditions, when EOD resistances are used in the calibration, or at BOR conditions, when BOR resistances are used in the calibration or at both EOD and BOR conditions (see Table 4.1). These methods include dynamic testing with signal matching (referred to here as dynamic load test), wave equation analysis methods, and the FHWA modified Gates and Engineering News (EN) dynamic formulas. They have two components:

- Analytical model.
- Pile driving information needed in the analytical model, mainly the hammer developed energy or stroke (and hammer efficiency in some methods) and the penetration resistance, N_b, defined as the number of hammer blows needed to drive the pile 1 inch, expressed as blows per inch, or bpi.

Example: In the Engineering News (EN) dynamic formula (AASHTO, 2010a), the geotechnical nominal static resistance during driving assuming zero geotechnical losses is determined during driving as R_n (Kips)= $12E_d/(s + 0.1)$, where E_d (ft.-tons) is the developed hammer energy and s is the pile permanent set in inches per blow, equal to $1/N_b$.

Table 4.1 Demonstration for Determination of Resistance in the Field with Pile Dynamic Analysis Methods

Time	Depth (ft)	Hammer Blow Count (bpi)	Stroke (ft)	Geotechnical Resistance (kips)
Driving	10	1	5	15
Driving	15	1.5	6	75
Driving	27	3.5	6.5	160
Driving	32	4.5	6.5	190
End of Driving (EOD) Condition	33	5	6.5	220
Restrike or Waiting Time				
Begin of Redrive (BOR) Condition	33	10	6.5	320

Reliability analysis involves three steps, Step 1: compilation of information at load test sites, Step 2: statistical analysis, and Step 3: reliability Analysis. This section will initially describe these three steps and then discuss the use of reliability calibrated results to evaluate the economics and improve accuracy of the calibrated design methods.

4.2.1 Compilation of Information at the Load Test Sites

At the load test sites, collect for each test foundation (for same soil, see Table 4.2):

- **Measured foundation axial nominal geotechnical resistances, R_m,** from the load test and all the **conditions** used to obtain them, including size, type, and methods for construction and quality control of test foundation, and load testing, type, procedure and results.
- **Predicted foundation axial geotechnical resistances, R_n,** from the calibrated method and all the conditions considered to predict them, including the procedures to determine and select strength properties with the static analysis methods, and driving information for the dynamic analysis methods (e.g., type and efficiency of hammer, penetration resistance, BOR or EOD conditions). Describe in details the analytical expression used to predict the

resistance and any assumptions used in this expression (e.g., plugged or unplugged conditions with the static analysis methods, input parameters for the wave equation analysis).

The conditions employed to obtain the measured and predicted resistances on test foundations at the load test sites should be similar to the conditions considered in the design and construction of production foundations as will be discussed in Section 5.

Table 4. 2. Demonstration Example: Statistical Analysis of Bias Resistance Data

# of data	Location	SPT-N for the Base Material	Base Resistance (base area, A_b = 1 ft^2)		Bias Resistance = Measured Resistance /Predicted Resistance
			Predicted Resistance from the Design Method = N A_b	Measured Resistance from Load Test	
		bpf	Kips	Kips	
1	Colorado	5	5	4.5	0.90
2	New York	22.5	22.5	20	0.89
3	Florida	15	15	12	0.80
4	California	16.5	16.5	23.5	1.42
5	Egypt	10	10	15	1.50
.					
.					
n = adequate # of load tests			**Assuming Normal Distribution:**		
			Resistance Mean Bias (λ)		**1.10**
			Standard Deviation		**0.33**
			COV		**0.30**

4.2.2 Statistical Analysis to Account for Design and Construction Uncertainties

For demonstration purposes, Table 4.2 presents sample data (not actual data) for calibration of a design method that predicts the axial base resistance for a drilled shaft as R_n (kips) = NA_b, where N is the measured SPT-N value and A_b is the shaft base area. The bias resistance is defined as R_m/R_n, where R_m is the measured resistance from the static load test. The bias resistance data are fitted to a probability distribution curve (see Figure 4.1) that is expressed analytically with a mean bias, λ, and coefficient of variation, COV. Assuming normal distribution curves for the bias resistance data, the resistance mean bias is computed as $\lambda = \sum(R_m/R_n)/n$, where n = number of data, and COV can be computed as demonstrated in Table 4.2.

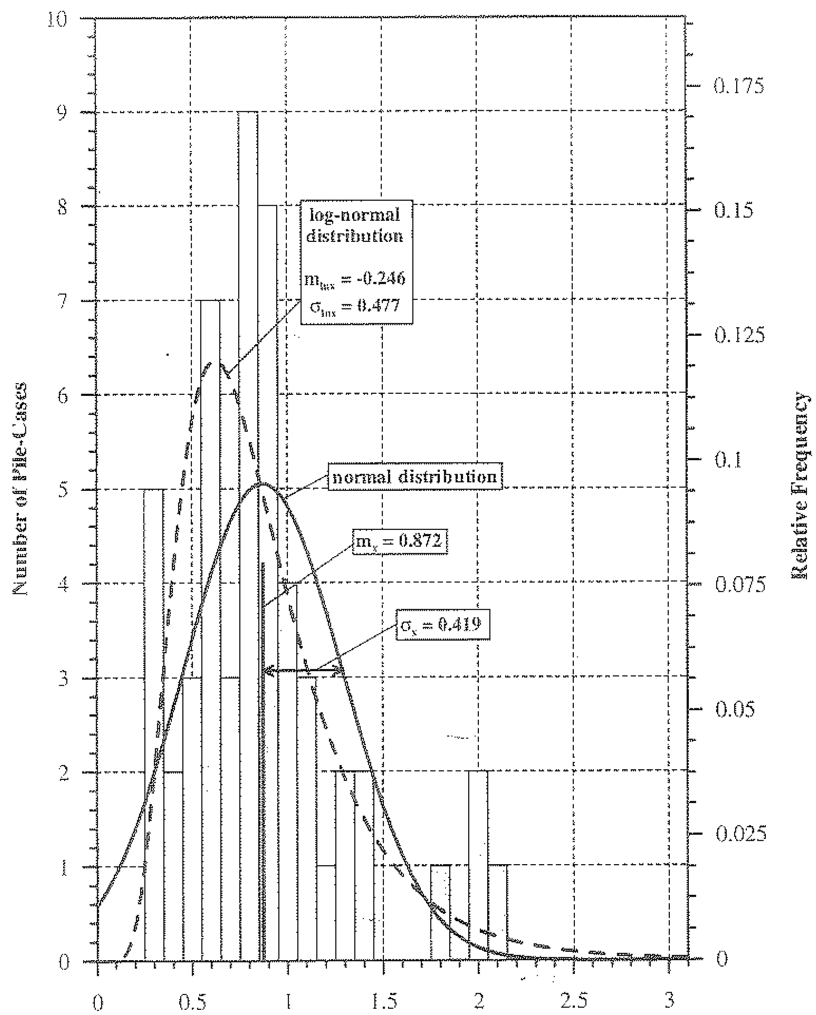

Figure 4.1. Example Histogram and Frequency Distributions of Bias Resistance Data (NCHRP Report 507)

The geotechnical resistance factor of a design method, ϕ, is mainly a function of its COV and λ (Figure 4.2).

- **COV** measures the variability of the method in predicting the resistance. The smaller the COV, the larger the calibratedϕ, and more accurate, precise, and economical is the calibrated method.
- **Resistance Mean Bias,** λ, measures the tendency of the design method to underestimate or overestimate the resistance, R_n, relative to the resistance measured by the static load test, R_m. For the same COV, the closer λ is to 1, the more accurate the design method is, and as λ increases, ϕ increases. The value of ϕ could even exceed 1 (see Figure 4.2) with large λ values (when the design method significantly underestimates the resistance measured by the load test).

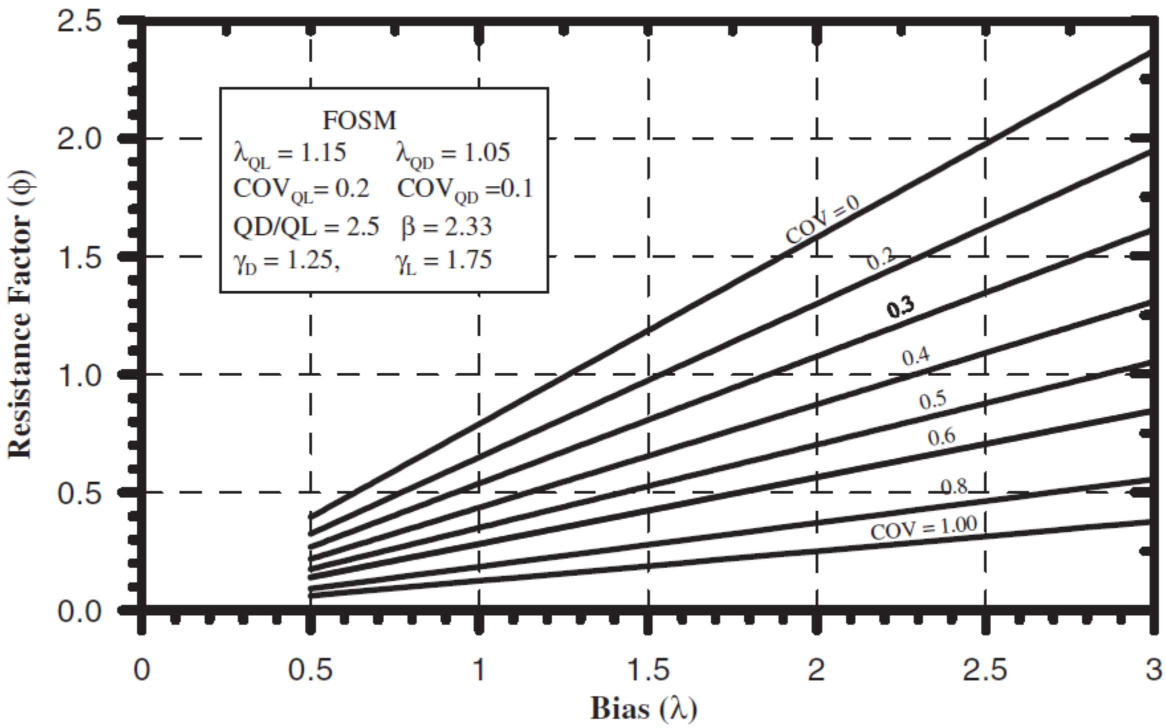

Figure 4.2. Resistance Factor as a Function of Resistance Mean Bias, λ, and COV (from NCHRP Report 507)

The statistical parameters λ and COV account for the following sources of uncertainty (errors and variations) in the predicted resistance, R_n (see Table 4.2):

- Errors in the analytical model (or $R_n = NA_b$ in Table 4.2) that predicts the nominal geotechnical resistance. This error would include the error resulting from the selection of input parameters in the analytical model. The more these input parameters (SPT-N values in our example) and the weight they represent in the analytical model (N vs. N^2) correlate with measured resistance from load tests, the smaller this error, and the more efficient is the calibrated design method. It would be expected that the design methods that employ soil properties obtained from in situ tests rather than laboratory tests, or from cone penetration test (CPT) rather than SPT, would be more efficient.

- Inherent variability of the measured soil properties due to:
  - Inherent (natural) variation in subsurface materials due to the natural characteristics of soil, where soil properties within the same stratum (e.g., dense sand) vary from point to

point. Due to this natural variation, the average value of soil/rock properties measured at various points in the vertical direction are used in the calibration to predict the foundation nominal geotechnical resistance, R_n. COV will include this variation because the measured and predicted resistances are obtained for the same soil/rock layer.

  - Inherent (natural) variation in geotechnical testing methods (e.g., the SPT in our example). Repeatability of test results under "almost identical" soil conditions have been used to assess the extent of this variation. For example, the use of the automatic hammer with the SPT will lead to more consistent N values and larger resistance factors than traditional hammers.

- Inherent variability of the driving systems in the field dynamic analysis design methods for driven piles (similar to the variation of testing methods in the static analysis methods).
- Errors and variations with methods for construction and QC/QA. The better and more consistent these methods are, the larger the measured resistance from the load tests would be, leading to larger resistance factors. Future research should consider the influence of construction method and QC/QA method on the efficiency of the design method, which would be increased with better methods for construction and QC/QA.
- Errors and variations of the load testing procedure, which will lead to errors and variations in the measured resistances from load tests. This emphasizes the importance of using quality load test data in the calibration of resistance factors.

4.2.3 Reliability Analysis to Determine Resistance Factors

The following information is needed in the reliability analysis (see Figure 4.2):

- **Similar given Information for all calibrated resistance determination methods,** including the target reliability index β (index of probability of failure), Load factors for DL (1.25) and LL (1.75), DL/LL ratio (often assumed to be 2), and λ and COV of dead and live loads (given).
- **Resistance λ and COV of the calibrated resistance determination method that will be calibrated** (discussed in the previous section). Different design methods for prediction of R_n have different combinations of λ and COV, depending on their accuracy and precision, which lead to different ϕ values for these methods (see Figure 4.2).

The uncertainties on the load side (represented by load factors) and resistance side (represented by resistance factors) are tied together through a target reliability index β. For the same reliability index, as the load factors increase, the calibrated resistance factor would increase. This rule is true for the calibration by fitting but for the same safety factor. Three reliability analysis methods have been reported in the literature ((Allen et al., 2005; Paikowsky et al., 2004) to determine ϕ that vary in terms of sophistication and accuracy. The simplest method is called the “First Order Second Moment (FOSM),” which seems to generate conservative resistance factors. The advanced methods are the “First Order Reliability Method (FORM)” and ”Monte Carlo” method.

4.2.4 Evaluation of Economics and Improvement of Accuracy of the Calibrated Design Method

Table 4.3 lists the results of reliability-based calibration for some of the static and field methods for driven piles as published in NCHRP Report 507 (Paikowsky et al., 2004).

Table 4.3. Resistance Factors for Driven Piles Design Methods from NCHRP Report 507

NCHRP Report 507						
Design Method		**# of Cases**	**λ**	**COV**	**ϕ**	**Efficiency ϕ/λ**
Static Analysis Methods						
Nordlund method: H-pile, sand		19	0.94	0.4	0.46	**0.49**
λ-method: concrete pile, clay		8	0.81	0.51	0.32	**0.39**
α-Tomlinson method		18	0.87	0.48	0.36	**0.41**
α-API method: concrete pile, clay		17	0.81	0.26	0.54	**0.67**
FHWA CPT method: concrete pile, mixed soil		30	0.84	0.31	0.51	**0.6**
Field Dynamic Analysis Methods						
Dynamic load test	EOD	125	1.63	0.49	0.64	**0.4**
	BOR	162	1.16	0.34	0.65	**0.56**
Wave Equation Analysis	EOD	99	1.66	0.72	0.39	0.24
	BOR*	99	0.94	0.42	0.43	0.46*
FHWA modified Gates	EOD	135	1.07	0.53	0.38	**0.36**

*Calibration results for BOR conditions are reported in Appendix B of NCHRP Report 507, where $\phi = 0.49$ for a reliability index (β) of 2 and $\phi = 0.40$ for $\beta = 2.5$, so it is estimated that $\phi = 0.43$ for $\beta = 2.33$

The determination of the resistance mean bias, λ, in addition to the resistance factor, ϕ, is one of the key advantages of statistical/reliability analysis over calibration by fitting to ASD. These two parameters, λ and ϕ, can be used to evaluate the economics and improve the accuracy of the calibrated resistance determination methods (called also design methods in this manual) as discussed next.

Economics of the calibrated resistance determination methods is function of its predicted factored resistances, which correlate with its efficiency **defined as ϕ/λ.** The larger the efficiency of the calibrated method, the larger the factored resistance, leading to either shorter pile lengths or a smaller number of piles. It is a common misinterpretation to associate more economical design methods as those with higher values of ϕ (or lower values of FS). The efficiency of a method cannot be related directly to the resistance factor, ϕ, because ϕ is also affected by the bias of the method (whether it over- or under-predicts capacity on average). The efficiency of the design method correlates with its COV (see Figure 16 in NCHRP Report 507). The more precise (lower COV) a design method is in predicting resistances, the higher its efficiency (ϕ/λ), and the more economical the method is. (See the efficiency results for different pile design methods in Table 4.3.) The static load test, which has a λ of 1 and efficiency equal to its resistance factor (around 0.7), is the most economical and efficient design method. The λ of the normal probability distribution curve for the bias resistance data should be considered in computing the efficiency of the design method even if λ and COV of the lognormal distribution curve for the bias resistance data are employed to determine ϕ.

The variations in λ among different calibrated design methods lead to the variation of the predicted resistance generated by these methods at the same location in the soil mass (see Figure 4.3). The static load test is the most accurate design method with λ= 1. The accuracy of the calibrated design method increases as its λ gets closer to 1. Accuracy of the calibrated design method can be improved by adjusting its λ to 1 so that its resistance predictions on the average would be similar to those measured by the static load test. This can be accomplished by adding a multiplication factor= λ to the analytical expression of the calibrated method.

Example: Reliability calibrated results for pile design methods 1, 2 and 3 are:

Design Method	ϕ	λ	ϕ/λ
1. R_n (Kips) = 4 A_bS_u	0.4	0.8	0.5
2. R_n (Kips) = 8 A_bS_u	0.2	0.4	0.5
3. R_n (Kips) = 3.2 A_bS_u	0.5	1.0	0.5

All of these design methods would generate similar pile lengths because they all have the same efficiency. Method 3 is the most accurate because its mean bias is the closet to one. The accuracy of all methods can be improved by multiplying their analytical expression by the mean bias, resulting in analytical expression as that for Method 3.

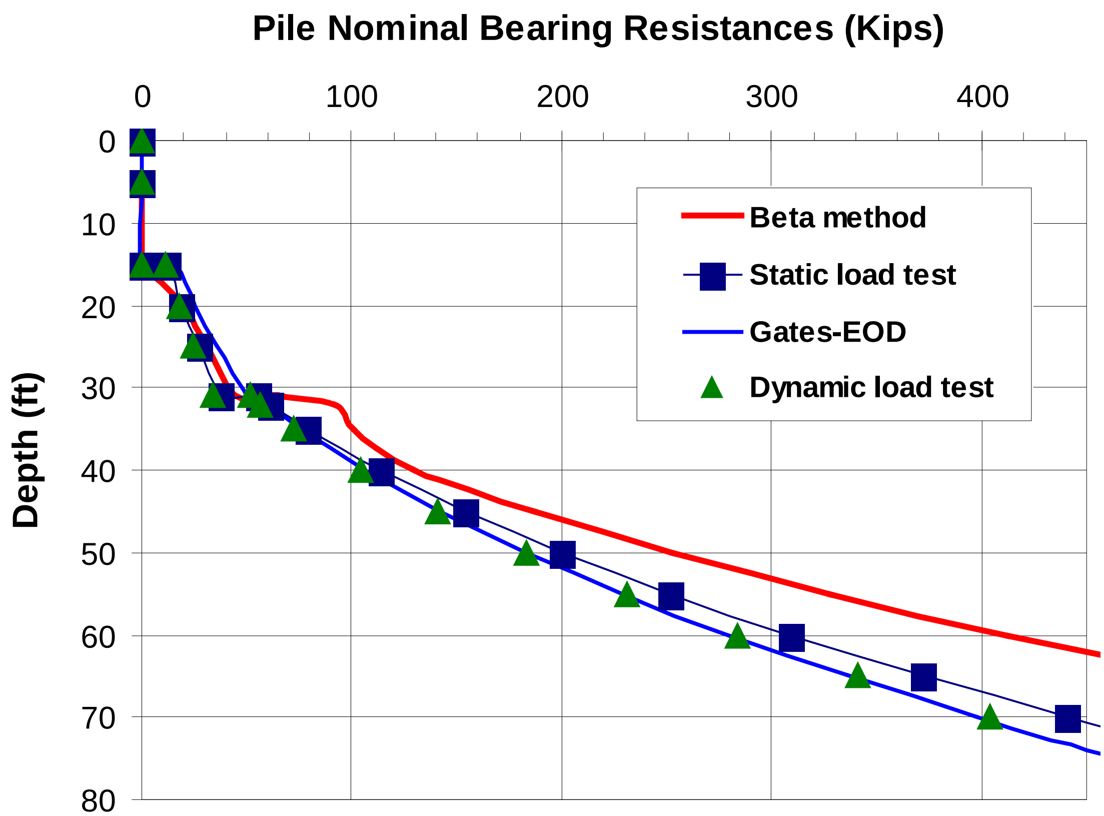

Figure 4.3. Pile Nominal Bearing Resistances, R_n, from Various Pile Design Methods

4.3 AASHTO'S LRFD CALIBRATION METHODS

In developing the AASHTO LRFD platform, reliability analysis was considered when a reliable, adequate number of load test data were available. Such data were available for the calibration of the axial resistance determination methods for a single drilled shaft and a driven pile. These data are compiled and analyzed in NCHRP Report 507 (Paikowsky et al., 2004).

Allen (2005) finalized most of the resistance factors for the AASHTO LRFD 2006 Interims for bridge foundations based on the results of NCHRP Reports 343 (Barker et al., 1991) and 507 (Paikowsky et al., 2004), and additional calibration work. These AASHTO's geotechnical resistance factors (ϕ) were developed by fitting to ASD and use of statistical/reliability analysis. When significantly different ϕ values were generated using different approaches, engineering judgment was considered to establish the final AASHTO LRFD ϕ values, considering the quality and quantity of load test data. **For the bearing and sliding resistances of spread footings**, the data used in NCHRP Report 343 (Baker et al., 1991) for reliability-based calibration of ϕ are limited and were developed from small-scale model tests. The AASHTO LRFD resistance factors for spread footings are very close to the resistance factors implied from calibration by fitting to ASD estimated using a γ_{ave} of around 1.4 and the safety factor reported in the AASHTO Standard Specifications (2002).

For micropiles, resistance factors for all resistances (including axial compression) were obtained through calibration by fitting to ASD methods (AASHTO 2010a).

4.3.1 AASHTO's Resistance Factors for Driven piles and Drilled Shafts

For the uplift resistance, the limited number of load test data for uplift resistance reported in NCHRP Report 507 justified the selection of lower ϕ values for uplift resistance than for axial compression resistance. **For the horizontal resistance,** presently, a ϕ value of 1 is employed for the horizontal geotechnical resistance of a single pile or pile group in AASHTO LRFD, which is not equivalent to that implied from calibration by fitting to ASD. Changes are expected in the future to address this issue. **For the block resistance,** the authors of this manual think that resistance factor was developed based on calibration by fitting using a γ_{ave} of around 1.4 and the safety factor reported in the AASHTO Standard Specifications (2002).

Methods to determine compression resistance of a driven pile and a drilled shaft. Resistance factors were initially selected mainly based on the reliability analysis developed in NCHRP Report 507 (Paikowsky et al., 2004). For drilled shafts, the analyzed cases in this report were grouped by soil type, design method, and construction technique. Reliability-calibrated results are presented for each combination. For static analysis methods of driven piles, the analyzed cases in this report were grouped by soil and pile types and design method. Reliability-calibrated results are presented for each group. For field dynamic analysis methods of driven piles, the analyzed cases were grouped only by design method (not by soil and pile types as with the static analysis methods). Reliability-calibrated results are presented for each design method. In 2009, AASHTO approved many changes to AASHTO LRFD Section 10, primarily addressing the redundancy issue with the design of driven piles, addressing the handling of site variability when selecting resistance factors for static load tests and dynamic testing, and adjusting resistance factors for the wave equation analysis method. These changes were implemented to reflect successful past ASD practices and the need to use engineering judgment when appropriate. **These methods are impacted with the transition from ASD to LRFD, and therefore the focus of Chapters 5 and 6 will be on these methods.**

CHAPTER 5

CALIBRATION CONDITIONS AND ASSESSMENT OF SITE VARIABILITY

This chapter describes the conditions that the DOTs need to adhere to when they adopt AASHTO's LRFD geotechnical design methods and provides recommendations to the DOTs to consider with they develop local LRFD geotechnical design methods. *As discussed in the previous chapter, reliability calibration of the AASHTO LRFD resistance factors is considered mainly for the axial compression geotechnical resistance determination methods of a driven pile and a drilled shaft at the strength limit. Therefore, these methods could be impacted by the transition from ASD to LRFD, and the focus of this chapter will be on the conditions to consider with these methods*. Other geotechnical design methods will not be impacted because their resistance factors were mainly developed through calibration by fitting to the ASD methods furnished in the AASHTO Standard Specifications. As discussed in the previous chapter, a certain acceptable level of project site variability in the horizontal direction called natural or inherent site variability is considered in the calibration of resistance factors. This chapter concludes with recommendations to evaluate and address the project site variability, which need to be considered with both AASHTO and local LRFD design methods.

It is assumed here that DOTs will adopt and use AASHTO LRFD loads in the design with both AASHTO LRFD and locally developed geotechnical LRFD design methods. As discussed in Chapter 2, the methods used to compute loads have been improved in AASHTO LRFD, and these changes (which resulted in increased loads in some cases) *would be applicable to ASD design if updates to the Standard Specifications were to be continued.*

5.1 AASHTO LRFD CONDITIONS

The DOTs need to justify any deviation from these conditions when they adopt AASHTO's LRFD geotechnical design methods.

Many of the conditions and parameters described in this section are presented in NCHRP Report 507 (Paikowsky et al., 2004) and Article 10.5 of AASHTO LRFD (2010a). AASHTO's resistance factors are calibrated for certain axial compression resistance determination methods for a driven pile and a drilled shaft at the strength limit only. It is possible that some well-developed axial compression strength limit design methods are not covered in AASHTO LRFD. Future development of reliability-based resistance factors is an ongoing process, and it is expected to cover new design methods for all limit states, other resistances, and new types of foundations (micropiles). Refinement of the existing reliability-based resistance factors is also expected in the future. As the AASHTO LRFD Specifications continue to be applied for bridge foundation design, the AASHTO's resistance factors are expected to be refined in the future to reflect the current standard of practice and results of new reliability-based research studies.

5.1.1 Design Conditions

These include the conditions employed in the reliability calibration to predict resistances, R_n, at load test sites.

5.1.1.1 Soil and Rock Properties

The DOTs need to adhere to the minimum guidelines for a subsurface exploration program as described in AASHTO LRFD Article 10.4.2 to establish reliable soil/rock properties for the design and construction of foundations.

In the reliability-based calibration, average measured soil and rock properties at each boring were used in the calibration of resistance factors. Use of conservative or lower bound design properties can negate some of the efficiency provided by LRFD principles and can result in overly conservative designs. AASHTO LRFD Article 10.4 acknowledges that some soil/rock deposits have natural variations in consistency and strength with depth, and recommends in this case either presenting the property value as a function of depth, or dividing the deposit into more than one layer. As will be discussed in Section 5.3, AASHTO's resistance factors account for a certain acceptable level of site variability across the site (horizontal site variability) called natural or inherent site variability.

AASHTO's reliability resistance factors are calibrated for certain groups of geomaterials: Sand, clay, and rock (for all foundation types), and cohesive and cohesionless intermediate geomaterials (IGM; for drilled shafts). Specific methods to determine and select the design soil/rock properties are considered in the calibration of resistance factors. These methods are described in AASHTO LRFD Articles 10.4 and 10.5 and in more detail in NCHRP Report 507 (Paikowsky et al., 2004). As an example for the use of static pile analysis methods for clays, AASHTO LRFD Article 10.5.5.2.3 states: "If the soil cohesion was not measured in the laboratory, the correlation between the SPT N value and s_u by Hara et al. (1974) was used for calibration. Use of other methods to estimate s_u may require the development of resistance factors based on those methods." See also the requirements in this article for obtaining the friction angle for use with the Nordlund/Thurman method.

5.1.1.2 Design Methods for Driven Piles

In the ASD axial design of a driven pile, the resistances predicted from a static analysis method are used with the safety factor of a field method to estimate the pile length in the design phase. The required field resistance is estimated based on the safety factor of the field method. The pile length must be finalized in the field at a depth where the measured resistance from the field method exceeds the required resistance. This is no longer the case with LRFD, where the geotechnical resistance factor, ϕ, is calibrated for both

- Static analysis methods, ϕ_{stat} (resistance predictions from these methods are used in the calibration of ϕ_{stat})
- Field dynamic analysis methods, ϕ_{dyn} (resistance predictions from these methods are used in the calibration of ϕ_{dyn})

The reliability calibration of resistance factors for the static analysis methods was totally independent of the reliability calibration of the field dynamic analysis methods. Therefore, and although AASHTO recommends using static analysis methods only in cases where the dynamic analysis methods are deemed unsuitable for field verification of the pile nominal bearing resistance, we believe that the static analysis methods can be used to finalize the pile length in the design phase as long as site variability is addressed.

The AASHTO's resistance factor for any static analysis method is applicable to different pile types (influence of pile type on the resistance factor is not considered).

Conditions with the field dynamic analysis methods. See Table 4.1 for the definition of EOD and BOR conditions. NCHRP Report 507 (Paikowsky et al., 2004) resistance factors for design methods for driven piles (static analysis and field dynamic analysis methods) are listed in Table 4.3.

According to AASHTO LRFD, if relaxation is possible in the foundation soils, then determination of the BOR resistance after a sufficient time is a must and should be considered in the design. If setup will be directly considered in the field design method, then setup must be verified in the field after a specified length of time (called "restrike time") by measurement of the pile BOR resistance. AASHTO LRFD provides recommendations for restrike times for different types of soils.

The AASHTO's resistance factor for any field dynamic analysis method is applicable to different pile, soil, and driving system types. This is because the influence of pile and soil types on the resistance factor is not investigated in the reliability calibration (NCHRP Report 507). In the

NCHRP Report 507, information on the restrike time and range of penetration resistances (blows per inch) considered in the calibration is not clear.

For the dynamic analysis methods, the same safety factor is used in ASD design for EOD and BOR conditions. This is not necessary correct with the reliability calibration where the calibrated resistance factor for EOD conditions can be different than for BOR conditions (see Table 4.3). AASHTO LRFD dynamic analysis methods (AASHTO 2010a) can be divided into two groups:

- **EOD methods:** where ϕ is calibrated using EOD resistances. With these methods, EOD resistances (assuming no relaxation) need to be measured and employed to determine pile length (no direct benefits from setup). In this case, setup should not be considered in the pile design. Methods in this category include the FHWA modified Gates and EN dynamic formulas. These dynamic formulas are calibrated in the NCHRP Report 507 only at EOD conditions (see Table 4.3). According to AASHTO LRFD, a lower ϕ should be used with these dynamic formulas at BOR conditions (to benefit from setup) than the ones provided in AASHTO LRFD for EOD conditions. However, no guidelines are provided in AASHTO to determine this lower ϕ. Dynamic testing, in lieu of dynamic formulas, is recommended to verify the resistance at BOR and benefit from setup.

- **EOD and BOR methods:** where ϕ for these methods were calibrated using both EOD and BOR resistances. To benefit from setup, use the BOR resistance factors in the design and measure the BOR resistance in the field. In the NCHRP Report 507, both the dynamic load test and the wave equation analysis method were calibrated at EOD and BOR conditions (see Table 4.3). For these two methods, the efficiency (economics) is significantly improved if setup is measured and verified at BOR conditions (see Table 4.3; note the larger efficiency at BOR than at EOD). These methods underestimate the resistance at EOD conditions because soil setup is not considered (see Table 4.3; note the large mean bias at EOD). For these two methods, AASHTO LRFD seems to suggest that the same AASHTO's resistance factor can be used for EOD and BOR conditions.

5.1.2 Construction and Load Testing Conditions

These include the conditions employed in the calibration to measure resistances, R_m, at load test sites.

<u>Construction Conditions:</u> AASHTO's ϕ are calibrated for certain pile and shaft diameters (<2 ft for piles; <6 ft for drilled shafts). Hence, AASHTO's resistance factors are not applicable to large size and high capacity deep foundations. AASHTO LRFD (2010a) suggests using the

AAHSTO's resistance factors with caution for design of piles of significantly larger diameter. AASHTO recommends static load testing for piles larger than 2 ft.

The AASHTO's resistance factors for the drilled shaft design methods are applicable to various methods for construction and quality control (influence of methods for construction and quality control on resistance factor is not discussed). Larger resistance factors for driven piles are recommended in AASHTO LRFD with increased level of quality control as will be discussed in the next chapter. To use AASHTO LRFD design methods, the DOTs need to employ good methods for construction and quality control based on: (a) AASHTO LRFD Bridge construction specifications (2010b); (b) FHWA manuals for design, construction, and inspection of various foundation types; and (c) the DOT's current construction specifications, which presents the DOT's experiences, practices and judgment with respect to construction of foundations.

Load Testing Conditions: AASHTO's reliability resistance factors are calibrated using the quick load tests (AASHTO, 2010a). Hence, it is assumed in the calibration that the resistance predicted during design will not change during the entire design life of the foundation. The designer needs to accommodate any possible reduction of this resistance during the design life of the structure (e.g., creep, softening and deterioration of resistance due to water or environmental factors). Also, AASHTO's resistance factors were calibrated for certain definitions of the geotechnical nominal resistance measured from load test. Davisson's failure criterion is selected for driven piles. For drilled shafts, the nominal resistance is defined as the load associated with a displacement of 5 percent of the shaft diameter, if plunging of the shaft cannot be achieved. The calibrated resistance factors would change if these definitions are changed.

5.1.3 Calibration Analysis Methods and Parameters

AASHTO's ϕ are calibrated for the strength I load factors/load combinations. This may be changed in the future, especially with the calibration of lateral geotechnical resistance determination methods where the dead and live loads are not the dominant loads in the design. Specific statistical/reliability analysis methods (FORM method) and assumptions (Lognormal distribution for bias resistance data) are considered in the calibration. Changes to these methods or to the assumptions used in these methods would change the calibrated resistance factors.

Specific target reliability index values (β) were selected in the reliability analysis with consideration of the foundation redundancy. Resistance factors would increase if the target β for calibration is reduced in the future. Redundancy for a group of deep foundation elements means that as one element is overstressed, the other elements can take on additional loads so that the foundation group will remain stable (see Figure 5.1). According to NCHRP Report 507 (Paikowsky et. al., 2004) redundant group foundations are defined as having at least five piles or shafts in the group, and a group with a small amount of redundancy is assumed for number of deep foundations between 2 and 4. In this report, the reliability-based calibration of resistance factors for the axial compression geotechnical resistance determination methods is based on β = 2.3 (which corresponds to P_f = 1/100) for a number of driven piles ≥ 5, andβ = 3.0 (P_f = 1/1,000) for number of shafts between 2 and 4. These β values were selected to achieve two purposes: (a) to generate resistance factors close to those obtained through calibration by fitting to ASD methods, and (b) to have a level of reliability or safety for foundations consistent with those employed in the structural design of bridge superstructures, where a reliability index of 3.5 has been used. The reliability of foundation groups will be greater than the reliability of individual foundation elements. Hence, a lower reliability index such as those listed above can be used for redundant foundations compared with those used for superstructures.

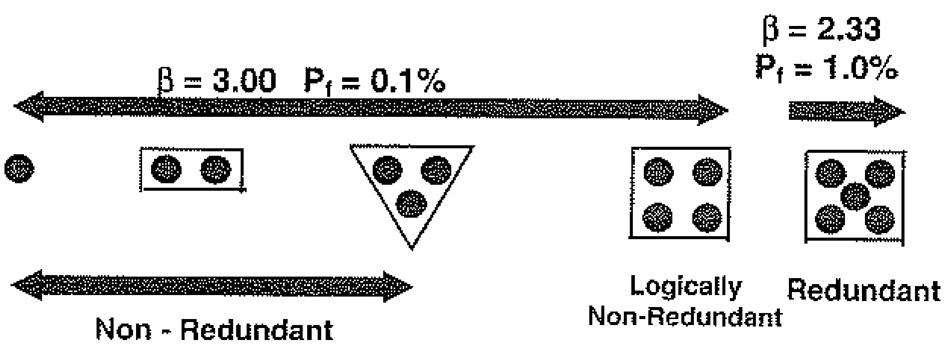

Figure 5.1. Redundancy of a Group of Deep Foundation Elements (from NCHRP Report 507)

The requirements to address foundation redundancy for driven piles in AASHTO LRFD (2010) is covered in article C10.5.5.2.3 (commentary) to allow for some engineering judgment in addressing foundation redundancy. For a small pile group, it is recommended in this article to reduce ϕ by 20 percent. Based on engineering judgment, the number of piles in a small pile group can range from one to four (our interpretation of Article 10.5.2.3). A pile group with five piles or more can be called redundant. For a single shaft, it is recommended in AASHTO Article 10.5.5.2.4 (not commentary article) to reduce ϕ by 20 percent to address the lack of redundancy.

5.2 SELECTION OF CONDITIONS FOR LOCAL CALIBRATION

The following AASHTO LRFD calibration conditions are appropriate to consider both with local calibration by fitting to ASD and based on reliability analysis:

- Strength I load factors/load combinations (can be changed if justified)
- Extent of subsurface exploration program per AASHTO LRFD Article 10.4.2

- Use of average measured soil and rock properties in the design.
- Certain acceptable level of variability across the site called natural or inherent site variability
- A reduced resistance factor with lower foundation redundancy
- Account for future changes of the foundation nominal geotechnical resistances
- Good methods for construction and quality control as discussed before.

With calibration by fitting to ASD, adopt design conditions similar to those used in the previous ASD geotechnical design method. For example, continue the use of the same ASD testing methods to determine soil/rock properties.

DOTs should document all conditions considered in the local reliability calibration of resistance factors based on load tests and request that the DOT foundation designers adhere to or are aware of them when using these local design methods.
Four categories of conditions to consider in the local reliability development of LRFD geotechnical design methods are discussed next.

5.2.1 Conditions from AASHTO's Reliability Calibration

To enable direct comparison of the locally developed resistance factors with those suggested by AASHTO, consider the following AASHTO's conditions discussed before (a) Load testing conditions, (b) reliability index. Article 10.5.5.2.1 of AASHTO LRFD reads: "If the resistance factors provided in this article are adjusted to account for regional practices using statistical data and calibration, they should be developed using the β values provided above, with consideration given to the redundancy in the foundation system." Document and justify any deviation from these AASHTO's conditions.

5.2.2 Statistical and Reliability Analyses

Consider one of the two advanced reliability methods to generate larger resistance factors (FORM and Monte Carlo). As Figure 4.1 suggests, researchers found that lognormal probability distribution curves better fit the bias resistance data (Allen, 2005; Long, 2009). In fitting the bias resistance data to a lognormal probability distribution curve, fit the lower range of bias resistance data (when predicted $R_n > R_m$), which is the most critical portion of the bias resistance data for the accurate calibration of resistance factors. Some researchers (Long, 2009) have employed the lognormal best fit of the tail bias resistance data in calibrating resistance factors, and this led to higher resistance factors than using the lognormal best fit of all bias resistance data, as was done in the NCHRP Report 507 (Paikowsky et al., 2004).

According to Article 3.5.4 of NCHRP Report 507 (Paikowsky et al., 2004), higher resistance factors will be obtained if only the available actual bias resistance data are used to calibrate the resistance factors. Use of a fitted probability distribution function in the reliability analysis may lead to the consideration of new extrapolated bias resistance data in the analysis. If an adequate number of bias resistance data can be obtained, it is suggested to limit the calibration of resistance factors to the measured range of bias resistance data and NOT extended beyond that range.

5.2.3 Local Design and Construction Conditions

The calibration should reflect the local design and construction practices. Cover local soils and rock formations that may not be covered in AASHTO calibration. Cover local design methods, including those not covered by AASHTO and local construction practices. Cover local methods to determine and select soil/rock properties, including those not endorsed by AASHTO. As discussed before, DOTs should consider improvements of their geotechnical design and construction practices at the same time as they're transitioning to LRFD. The DOT needs to consider new and efficient geotechnical design and construction practices that would improve the local practices. Consider methods recommended by AASHTO and FHWA technical manuals, and new promising methods reported in the literature or used by other DOTs. The reliability calibration can be used to compare the local and new design methods and select the more economical methods as will be discussed in the next chapter. The local reliability calibration will capture the value and justify the consideration of better methods for construction and quality control in the local practices (leading to large φ).

The efficiency of local reliability calibrated design methods can also be improved by narrowing the range and ensuring consistency of calibrated conditions as discussed next**.**

Drilled shafts: consider the influence of type of construction method and type of quality control method on the calibrated resistance factor. NCHRP Report 507 (Paikowsky et. al., 2004) examined the influence of construction methods for drilled shafts on the calibrated resistance factors. For sand, the efficiency of the design method is higher with the use of casing than with the use of slurry. The calibrated resistance factor varies from 0.28 with the slurry method to 0.73 with the use of casing. For clays, AASHTO (2010a) recommends considering local experience with the geologic formation and construction practices in the selection of resistance factors. To ensure quality of constructed deep foundations, NCHRP Report 507 suggests testing all production shafts for major defects after construction.

Static Analysis Methods for Driven Pile. Consider pile and soil type in the local calibration of resistance factors**.** Calibrated ϕ results were reported in NCHRP Report 507 (Paikowsky et. al.,

2004) for various soil and pile types, but AASHTO's resistance factors are reported for various soil types. Allen (2005) recommended that designers be aware of the NCHRP Report 507 results and the influence of pile type on the resistance factors, which can be significant. Consider new static analysis methods not described in AASHTO in the local calibration. NCHRP Report 507 calibration results suggest that some of the new static analysis methods are more efficient than some of the field methods (see Table 4.3). For example, the α-API method described in the FHWA manual on driven piles (Hannigan et. al., 2006) was calibrated in NCHRP Report 507 and found to be very efficient (see Table 4.3). In the calibration for open-ended piles, calibrate the resistance factors assuming both plugged conditions and unplugged conditions, and select the conditions with larger efficiency.

Dynamic Analysis Methods for Driven Piles. Calibrate ϕ at EOD and BOR conditions. The efficiency of the field dynamic analysis method would increase if ϕ is calibrated at BOR conditions as indicated in the calibration results for the wave equation analysis and dynamic load test (Table 4.3). Consider pile and soil types (and type of driving system if possible) in the calibration of ϕ. For example, it would be preferable to calibrate a dynamic analysis method specifically for closed-end pipe piles driven into cohesive soils with certain common driving systems than to calibrate this method for use with all pile types; all soil types, and all driving systems as was performed in the NCHRP Report 507 calibrations. The AASHTO's resistance factor for any dynamic analysis method is calibrated to different types of piles, soil, and driving systems. This could reduce the overall efficiency of the methods in order to be applicable to different conditions. Employ consistent restrike time and procedure as employed in the DOT's practices and a small range of penetration resistances for better calibrated results (this range at EOD conditions can be different than for the BOR conditions). It is acceptable to consider in the calibration restrike times and procedures, and ranges of penetration resistances as a function of the soil and pile types as long these conditions are observed in the installation of production driven piles.

5.2.4 Conditions at Load Test Sites

Load test information needed for calibration can be obtained from existing local load test results obtained in the state. These can be supplemented with load test data obtained from other states, especially neighboring states with similar field and construction conditions, and by reviewing records of load tests reported in the literature. NCHRP Report 507 (Paikowsky et al., 2004) has a robust database of load tests for driven piles and drilled shafts. Additionally, to get valuable load test information for calibration, consider development and execution of a program of new local load tests, especially in large projects where load testing can be very cost-effective. For selection of existing load test data or planning of new local load tests, **the DOTs need to ensure that the calibrated conditions at the load test sites on test foundations (disused below) cover and be**

consistent with the local conditions considered by the DOT in the design and construction of its production foundations. This will determine the number and details of load tests selected for calibration. The DOTs should document all the conditions employed to obtain the measured and predicted resistances on test foundations at load test sites so that the DOT engineers adhere to in the design and construction of production foundations. These conditions are presented next.

- The soil type and its range of strengths. For example, if the calibrated design method is for sand that ranges in its friction angle from 25 to 40 degrees, then several load tests should be conducted at sites that have sand with a friction angle ranging from 25 to 40 degrees. If the design method is for different soil types, then all soil types and their ranges of strengths should be tested.
- Foundation types and sizes. If the calibrated design method and its resistance factors are intended to be applicable to different pile types and sizes, then all these pile types and sizes should be load tested.
- Foundation construction and QC/QA methods. The methods employed to construct the test foundation at load test sites need to be similar to those employed in the construction of production foundations.
- Details of the analytical expression employed to predict resistances at load test sites and any assumptions used in this expression so designer can follow in their design. **For the static analysis methods,** the conditions include the procedure to determine soil/rock strength properties. If plugged conditions for piles are assumed in the design method, then plugged conditions should be assumed in predicting the resistance at the load test sites. **For the pile field dynamic analysis methods**, the conditions include BOR and/or EOD conditions, restrike procedure and time, type of driving systems, and ranges of penetration resistances. The restrike time and procedure employed to predict BOR resistance at the load test sites should be similar to the waiting time and procedure employed by the DOT to measure BOR resistance. The load tests should also cover the types of driving systems commonly used by the DOT, and should be limited to the range of penetration resistances allowed by the DOT in the installation of its production piles.

- **Load Testing Conditions**. Consider those described in AASHTO's conditions, and type of load tests (e.g., static, Osterberg), which may have an influence on the calibrated ϕ, and "load test time" defined as the elapsed time between the end of construction of test foundations and load testing. The resistance factor would increase with the increase of measured resistance. Hence, select in the calibration the longest practical load test time to capture increases (common with setup) or even reduction (relaxation) of resistances. The calibrated load test time should be consistent among the calibrated cases. It is acceptable to consider different load test times in the calibration of resistance factors for different combinations of soil and pile types.

5.3 ASSESSMENT OF SITE VARIABILITY

Site variability occurs due to the presence of subsurface soil or rock deposits that are geologically different across the project site in terms of layer thickness, loading history, and engineering properties. The focus here is on site variability due to variation in the measured soil or rock properties across the site. This site variability can be quantified by calculating the COV of the measured soil and rock properties across the site, as will be discussed later.

AASHTO LRFD commentary to Article C10.5.2.4 (AASHTO, 2010a) suggests that the site variability is not an issue if it does not exceed the inherent variability, $COV_{inherent}$, of the subsurface materials and testing methods. This is in agreement with the reliability analysis discussed in Section 4.2, where the calibrated ϕ accounts for this inherent variability, $COV_{inherent}$. In this manual, the site or a zone in the site are is defined as "**uniform**" if the computed COV for the site/zone, COV, is smaller or equal to $COV_{inherent}$ of the site/zone.

Assessment of site variability requires the evaluation of site variability and if needed addressing it.

5.3.1. Evaluation of Site Variability

Initially, perform subsurface exploration program in accordance with AASHTO LRFD Article 10.4.2 to establish reliable design and construction soil and rock properties at locations of various foundations. Based on the measured soil properties at various borings, ignore the variability of soil and rock layers that would not contribute to resistance (e.g., soft surface soil layers, or when the foundation resistance is derived from competent rock, the variability of the soil layer above the rock is not an issue). For the soil/rock layer (s) that contribute to foundation geotechnical resistance, determine its inherent variability, $COV_{\mathbf{inherent}}$, and evaluate its variability across the site, COV as discussed next

5.3.1.1 Determination of Inherent Site Variability

The $COV_{inherent}$ can be estimated by the project geotechnical based on the results of subsurface exploration program, the type of testing employed in the subsurface exploration program, judgment and past experience. Also, consider the COV reported by Duncan (2000) in Table 5.1. for common soil properties, where COV for the SPT results ranges from 15 percent to 45 percent, and it is much larger than COV for the Cone Penetration Test (CPT) results. Finally, consider recommendations of of the NCHRP Report 507 (Paikowsky et al., 2004) where the upper limit for the low site variability is defined as 25%. It seems appropriate to assume COV= 25% corresponds to the inherent site variability. The disadvantage of this approach is that one $COV_{inherent}$= 25% is assumed for all soil types and testing methods.

Table 5.1. Values of Coefficient of Variation for Geotechnical Properties (after Duncan, 2000)

Measured or interpreted parameter value	Coefficient of Variation, V (%)
Unit weight, γ	3 to 7 %
Buoyant unit weight, γ_b	0 to 10 %
Effective stress friction angle, ϕ'	2 to 13 %
Undrained shear strength, s_u	13 to 40 %
Undrained strength ratio (s_u/σ_v')	5 to 15 %
Compression index, C_c	10 to 37 %
Preconsolidation stress, σ_p'	10 to 35 %
Hydraulic conductivity of saturated clay, k	68 to 90 %
Hydraulic conductivity of partly-saturated clay, k	130 to 240 %
Coefficient of consolidation, c_v	33 to 68 %
Standard penetration blowcount, N	15 to 45 %
Electric cone penetration test, q_c	5 to 15 %
Mechanical cone penetration test, q_c	15 to 37 %
Vane shear test undrained strength, s_{uVST}	10 to 20 %

5.3.1.2 Determination of Site Variability

The evaluation of site variability requires analysis of the measured soil properties at various borings from the subsurface exploration program. For demonstration purposes (see Table 5.2), only one sand layer is assumed to contribute to foundation resistance, and its variability across the site will be evaluated. Table 5.2 shows the measured SPT-N values for this sand layer in terms of # blows per foot (bpf) at three borings located at the east abutment, center pier, and west abutment.

Table 5.2. Determination of Site Variability, Step 1: For Each Boring, Obtain the Average Property Values

		Boring 1	Boring 2	Boring 3
Soft clay layer: not considered in the design, so no need to address its variability		**East Abutment**	**Pier**	**West Abutment**
		SPT N-Values (bpf)		
Sand Layer		6	6	13
		6		14
		9	6	15
		9	9	14
		10	7	12
Average N value (bpf)		**8**	**7**	**14**

Step 1. In the vertical direction, obtain the average soil and rock properties for the soil layer in each sounding or boring (see Table 5.2). Note that a**verage soil property values are used in the calibration of resistance factors.** Use of conservative or lower bound design properties can negate some of the efficiency provided by LRFD principles and can result in overly conservative designs. AASHTO LRFD Article 10.4 acknowledges that some soil/rock deposits have natural variations in consistency and strength with depth, and recommends in this case either presenting the property value as a function of depth, or dividing the deposit into more than one layer.

Step 2. Determine the mean property value, standard deviation, and COV for the average property values measured at various borings (see Table 5.3). If the COV is lower than the COV for the inherent site variability, use the mean soil/rock property value to develop a single geotechnical design profile for the entire project site, since site variability is not issue. If it is not, compute the lower limit property value as the mean value minus the standard deviation (SPT-N = 6 blows per foot in our example shown in Table 5.3) and move to step 3.

Table 5. 3. Determination of Site Variability, Step 2: Analyze Average Property Values Obtained from Various Borings

Boring Results	
Boring Location	**Average Corrected N Value (bpf)**
Boring 1, East Abutment	8
Boring 2, Pier	7
Boring 3, West Abutment	14
Analysis of Average Property Values (N)	
Mean Property Value	10
Standard Deviation	3.3
Coefficient of Variation (%)	34
Lower Limit Property Value	6

Step 3. According to AASHTO LRFD (commentary to Article C10.4.6.1), initially conduct a sensitivity analysis with the mean property value (SPT-N = 10 bpf) and lower limit property value (N = 6 bpf) to assess their influences on the geotechnical design. If the sensitivity analysis shows minimal influence (e.g., the foundation depth or width is almost the same using the mean property value and the lower limit property value), conclude the analysis. Otherwise, site variability is an issue and should be addressed in the design.

5.3.2 Addressing Site Variability

The simplest way to address site variability is to design all foundation in the project site using the same conservative geotechnical design values (property values and resistance factors) as demonstrated next. In the static analysis methods, select conservative property value (lower limit property value, SPT-N = 6 bpf; see Table 5.3). In the static load test and dynamic analysis methods, perform tests at the locations where the soil/rock strength is relatively small (where SPT-N= 6 bpf). In this case, there is no need to change ϕ in the design. The other alternative is to select conservative ϕ value as a function of the level of site variability (low, medium, or high) and the number of tests. This later approach is recommended in the NCHRP Report 507 (Paikowsky et al., 2004) and allowed by AASHTO LRFD (2010a). However, these approaches are not economical! **Better approaches to address site variability are presented next.**

Initially, it is suggested to finalize the design (size) of production foundations in the field based on the soil and rock conditions encountered during construction. This is possible with some field design methods, like the dynamic formulas, where all production piles are driven to a driving criterion determined *before driving the pile.* In this case, the pile will penetrate more deeply at the zones with weaker soils than at the zones with stronger soils. This would lead to varying pile lengths across the entire project site for the support of the same foundation factored load, so site variability is addressed in these methods. **Coupled or in lieu of this approach (if is it not feasible), and based on AASHTO LRFD (2010), it is suggested to drill more and deeper borings (AASHTO LRFD article 10.4.2) and divide the project site into smaller "uniform" zones** where the subsurface conditions for each zone are relatively uniform. More and deeper borings may be needed for better assessment of the site variability in the vertical and directions and for locating boundaries of different uniform zones. In this manual, uniform zone has COV smaller than its $COV_{inherent}$. A uniform zone could even be limited to a single substructure unit. For example, the project site in Table 5.2 can be divided into two smaller zones: Zone A for the east abutment and the pier (mean SPT-N value = 7.5 bpf) and Zone B for the west abutment. Another test hole should be drilled at the west abutment in order to ensure that there is no significant variation in the soil properties in that zone and to determine the mean N-value. Then, we suggest to design foundations in each zone based on the measured soil properties in that zone.

Applications of the approaches recommended above in addressing site variability are discussed next.

Static Analysis Methods. For each uniform zone, use the mean property value for that zone in design with no changes to ϕ. Design foundation in each zone based on soil properties in that zone. This would lead to varying foundation lengths across the project site for the support of the same foundation factored load. Also with these methods, it is suggested to finalize design in the field based on the soil and rock conditions encountered during construction. As discussed by Brown et. al. (2010), the design of drilled shafts should be based on some criterion of a minimum length of embedment into a certain bearing stratum. This would require the shaft tip elevation to be finalized during construction, and this elevation could be different from what is provided in the design plans. If the number of test holes is adequate to divide the project site into uniform zones as suggested before, the difference in the tip elevation between the plan and field would be minimal. It is also recommended to require one boring per shaft for rock-socketed shafts to provide site-specific information on depth to rock and quality and strength of the rock along the side and beneath the tip of the shaft. Where extreme site variability is expected over a short distance or with depth, multiple and deeper borings may be justified at a single shaft location (e.g., large-diameter shafts in karstic areas).

Field Dynamic Analysis Methods for Driven Piles. Develop the boundaries of the uniform zones through the boring results in the design phase, and finalize these boundaries in the field with test pile results. Use the recommended ϕ (no change) to estimate the required resistance. At a minimum, consider a test pile for each uniform zone and drill to a depth below the required depth or resistance (to capture vertical variation in the influence zone). Based on the test pile results, develop driving criteria and order pile lengths for the production piles in each zone. The driving system employed with the test pile should be the same as that employed for the production piles.

Static Load Tests. After the project site is divided into uniform zones as previously discussed, select the locations and number of load tests. For locations of load tests, consider the zone with lowest and highest and average soil and rock strength, and where heaviest loads will be applied.
Investigate if the measured resistances from the load tests at different zones suggest that there is site variability (as suggested by the results of the measured soil properties). If a static load test will not be performed in each zone, remember that the static load test results reflect the resistance in the zone where it was performed. Calibrate the static analysis methods or field dynamic analysis methods to predict resistances at zones where static load tests were not performed. Then, use this calibration, engineering judgment, and past ASD experience to extrapolate the load test results to other zones where load tests were not conducted. In this extrapolation, you may need to consider a resistance factor smaller than the resistance factor for load tests. In some cases, extrapolation of load test results to other zones may not be possible (i.e., if the type of soil/rock deposit varies significantly across the project site).

CHAPTER 6

SELECTION OF LRFD GEOTECHNICAL DESIGN METHODS

As discussed before, DOTs have three options for the selection of LRFD geotechnical design methods: i) Adopt AASHTO's LRFD methods; ii) Develop local LRFD methods by fitting to ASD methods that have track records of long-term success, and iii) Develop local LRFD methods through reliability analysis of information collected at load test sites. Chapters 4 and 5 described how these three options can be implemented by the DOTs. This chapter evaluates and compares these three options in order to assist the DOTs in the selection of the most appropriate option.

Reliability calibration of the AASHTO LRFD resistance factors is considered mainly for the axial resistance determination methods of a driven pile and a drilled shaft at the strength limit. Therefore, these methods could be impacted by the transition from ASD to LRFD, and this chapter focused on them. Other geotechnical design methods will not be impacted because their resistance factors were mainly developed through calibration by fitting to the ASD methods given in the AASHTO Standard Specifications.

6.1 COMPARSION OF 2010 AASHTO LRFD AND AASHTO STANDARD

This section compares the 2010 AASHTO LRFD geotechnical resistance factors (2010a) with those implied from calibration by fitting to the ASD methods given in the 2002 AASHTO Standard Specifications (2002). This comparison will help DOTs to determine the impact of AASHTO LRFD geotechnical design on their practices.

6.1.1. Drilled Shafts

AASHTO LRFD resistance factors for drilled shafts supported by sand, IGM, and rocks are close to those implied from AASHTO Standards. For shafts in clays, AASHTO LRFD resistance factors are lower than those implied from AASHTO Standards. This difference is attributed to local geologic conditions and construction practices not considered in the reliability calibration (Allen, 2005).

6.1.2 Driven Piles

In chapter 5, two differences between AASHTO LRFD and AASHTO Standards are discussed. First, in the AASHTO Standards (2002), the pile length must be finalized in the field based on the geotechnical resistance measured from the "field resistance determination method" (also called in this manual “field design method”). With AASHTO LRFD, the static analysis methods can also used to finalize the pile length in the design phase as long as site variability is addressed. Second, for the dynamic analysis methods, the same safety factor is used in ASD design for EOD and BOR conditions (see Table 4.1 for definition of these two conditions). This is not necessary correct with the reliability calibration where the calibrated resistance factor for EOD conditions can be different than for BOR conditions (see Table 4.3).

Comparison of the Pile Field Design Methods. This comparison assumes uniform sites/zones (low site variability) as discussed in Section 5.3.

Static load tests. A geotechnical resistance factor of 0.75 is recommended in AASHTO LRFD, which is larger than 0.7 implied from the ASD with a safety factor of 2 (AASHTO Standards, 2002). A large ϕ of 0.8 is recommended in AASHTO LRFD if dynamic testing is conducted on a minimum of 2 percent of production piles, but no fewer than two piles. This means that the use of static load tests in AASHTO LRFD would lead to more savings than in ASD.

Dynamic testing with signal matching. If the dynamic testing is limited to 2 percent of the production piles, but no less than two piles, a resistance factor of 0.65 is recommended in AASHTO LRFD, which is very close to the resistance factor of 0.62 implied from the ASD with a safety factor of 2.25. Higher geotechnical resistance factor of 0.75 is allowed in AASHTO LRFD if dynamic testing is conducted on 100 percent of production piles. Hence, with AASHTO LRFD, there is potential for savings with this method.

It is clear from the above comparisons that AASHTO LRFD rewards increased level of quality control.

FHWA modified Gates dynamic formula. The AASHTO LRFD resistance factor for this method (0.4) corresponds to the safety factor recommended in the ASD (3.5).

Engineering News (EN) dynamic formula. Assume no geotechnical losses from scour or downdrag in the comparison discussed next. In the ASD, the form of this EN formula for the allowable capacity is $2/[E_d(s + 0.1)]$, and its equivalent factored resistance (multiplied by the average load factor of 1.4) is $2.8/[E_d(s + 0.1)]$. In the LRFD, the nominal resistance form of this EN formula is $12/[E_d(s + 0.1)]$, which is six times the allowable form (a safety factor of 6 is assumed to get this form from the allowable form). The resistance factor for the LRFD nominal resistance form is 0.1 because this form has i) a small resistance mean bias, λ, suggesting this form significantly overestimates the resistance when compared to the resistance measured from load tests), and ii) a large COV of 0.92, suggesting large variability and low efficiency of this form. The form for the factored resistance of the AASHTO LRFD EN formula is $1.2/[E_d(s +$

0.1)]. Based on the above, it can be concluded that the AASHTO LRFD form for the EN formula, when compared with the allowable ASD form used by many DOTs, will lead to a smaller factored resistance and more costly designs (longer and larger number of piles). AASHTO suggests that the Gates formula is preferred over the EN formula.

Wave Equation Analysis (WEAP). The current AASHTO LRFD ϕ value of 0.5 for this method is developed based on calibration by fitting to ASD. This resistance factor can be used if local experience or site-specific test results are employed in the selection of the wave equation soil parameters, and if field verification of the hammer performance is performed. This resistance factor is larger than the reliability-calibrated ϕ for this method (around 0.4; see Table 4.3) recommended in the NCHRP Report 507 (Paikowsky et al., 2004) and adopted by AASHTO until 2010. This difference is attributed to the use of default WEAP input parameters for hammer and soil in the reliability-based calibration, without consideration of local experience or site-specific tests, as is required with the use of a larger resistance factor (ϕ = 0.5).

6.2 COMPARISON OF AASHTO LRFD AND LOCAL ASD METHODS

It is common for DOTs to select the AASHTO LRFD design methods for service limit states (all foundation types), spread footings and micropiles (all limit states), and the geotechnical resistances for deep foundations except the axial compression resistance of a single driven pile or drilled shaft. Deviations from the AASHTO LRFD have been reported by DOTs for the axial compression resistance determination methods for a single driven pile and a drilled shaft. These deviations, most likely existed even when those agencies used the AASHTO Standard Specifications, and are often attributed to i) Local ASD methods are more economical than those provided by AASHTO LRFD, and ii) Local ASD practices are not covered by AASHTO LRFD in terms of design methods, special soil/rock formations, and local testing methods to determine soil/rock design properties.

DOTs need to compare AASHTO LRFD and local ASD methods before selecting LRFD design methods based on calibration by fitting. Two criteria can be considered in this comparison:

- **Based on reliability.** Advantages of reliability calibration are discussed later. Some AASHTO design methods were calibrated based on statistical/reliability analysis. At a minimum, it is recommended that DOTs select the AASHTO LRFD design methods if they will lead to slightly higher foundation costs.
- **Based on economics.** Select the geotechnical design method that would either add no costs (so that costs would be similar to those for existing ASD design), or would generate savings. This comparison is discussed next.

AASHTO LRFD (2010a) provides several alternative geotechnical design methods. For comparison purposes, select a method from AASHTO based on its feasibility for use within the DOT's current practices and economics (based on its efficiency, ϕ/λ, as discussed in Section 4.2). In the comparison, the AASHTO LRFD loads (AASHTO, 2010a) should be used with both the AASHTO LRFD and local ASD geotechnical design methods. As discussed in Chapter 2, the methods used to compute loads have been improved in AASHTO LRFD, and these improved methods would be applicable to ASD design platform if updates to the Standard Specifications were to be continued.

Two approaches for the economical comparison of AASHTO LRFD and local ASD methods are suggested. In the 1st approach, compare results of the ASD and LRFD platforms on actual projects. The differences in the foundation sizes between the two approaches can be due to differences in the structural design (not addressed), or/and differences in the geotechnical design (our focus). With this approach, it is important to account for any hidden conservatism in the ASD methodology, where the loads could be overestimated and the resistances underestimated. In LRFD, the load and mean resistances computed per prescribed methods should be considered in the design and any uncertainties involving loads and resistances are accounted for by the load and resistance factors. In the 2nd approach, compare results of the ASD and LRFD geotechnical design methods. For the service limit design methods, compare the predictions for foundation displacements generated by the ASD method and the AASHTO LRFD method. For the strength limit design methods, compare the predictions for the foundation factored geotechnical resistance generated by the ASD method and the AASHTO LRFD method. The factored geotechnical resistance of an ASD method can be determined by multiplying its allowable capacity by the average load factor, γ_{ave}, (around 1.4) as discussed in Chapter 4. The comparison suggested in the 2nd approach becomes more complicated if the input information (e.g., soil and rock design properties) needed for the local and AASHTO design methods are different.

Demonstration:

In 2006, the Geotechnical Office of the Illinois Department of Transportation (IDOT) decided to abandon its modified form of the EN dynamic formula and adopt the FHWA modified Gates dynamic formula as presented in AASHTO LRFD. IDOT started an investigation to compare its ASD modified EN formula with the AASHTO LRFD FHWA modified Gates formula to assess the impact of this transition on the department's practices. The FHWA assisted with this investigation where it is assumed for simplification that there are no geotechnical losses from scour or downdrag, and the units for loads and resistances are in kips.

IDOT used a modified form of the EN dynamic formula to predict the pile allowable capacity during driving as $1.33E_d/(s + 0.1)$. The form for the factored resistance that this dynamic formula would generate is obtained by multiplying its ASD form by 1.4, leading to $1.86E_d/(s + 0.1)$. For the FHWA modified Gates formula, the form for the nominal static geotechnical resistance, R_n, is $1.75(E_d)^{0.5}\log_{10}(N_b) - 100$, and the form for its factored resistance is $\phi\,[1.75(E_d)^{0.5}\log_{10}(N_b) - 100]$. IDOT investigated the use of $\phi = 0.4$ (as recommended by AASHTO) and higher ϕ values of 0.5 and 0.6. The objective was to select a resistance factor for use with the Gates formula that would not increase the pile quantities compared with the department's previous practices. IDOT has long-term experience and success with driving piles using the modified EN dynamic formula.

The results of the comparison between the ASD modified EN formula and the LRFD FHWA modified Gates formula for various hammer energies and penetration resistances of 10 bpi and 5 bpi are listed in Table 6.1. Based on these results, IDOT decided to use a resistance factor of 0.5 with the FHWA modified Gates, higher than the 0.4 recommended by AASHTO. According to the results in Table 6.1, this would lead in some cases to the use of larger hammers and longer piles (when the factored resistance from AASHTO LRFD is smaller than from the ASD modified EN formula), and in some cases to smaller hammers and shorter piles than have been required in the past with the EN dynamic formula.

Table 6.1. Economical Comparison of ASD and LRFD Design Methods Conducted by FHWA for Illinois Department of Transportation

Developed Hammer Energy (Ib-ft)	**Factored Resistance (Kips) for Penetration Resistance of 10 bpi**			
	ASD modified ENR formula	**FHWA Modified Gates per AASHTO LRFD**		
		ϕ= 0.4	ϕ= 0.5	ϕ= 0.6
14,000	131	126	157	188
20,000	187	158	197	237
25,000	233	181	227	272
30,000	280	202	253	304
35,000	327	222	277	333
40,000	373	240	300	360
50,000	467	273	341	410
60,000	560	303	379	454

6.2.1 Selection of Local ASD Design Method to Develop the LRFD Design Method

The DOT is responsible for this selection and should provide in its LRFD design manual justification for the local calibration of resistances and selection of resistance factors different from those recommended by AASHTO (2010a). In this case, the AASHTO LRFD loads should be used in the design, but, as discussed in Section 5 of this manual, some of the conditions considered in the ASD platform should be continued (e.g., ASD procedure for the estimation of soil and rock design properties).

It is acceptable to combine locally calibrated design methods with AASHTO LRFD design methods as long as all applicable limit states are evaluated. This approach may require two different procedures to evaluate soil and rock properties, AASHTO LRFD procedures and local testing procedures. It is recommended that DOTs adopt the AASHTO LRFD testing procedures wherever possible. Through research studies, develop correlations between the results of local testing methods and AASHTO's testing methods for the determination of soil and rock properties. Update the locally calibrated LRFD design methods based on this correlation.

6.3. ADVANTAGES OF LOCAL RELIABILITY CALIBRATION

The development of LRFD geotechnical methods based on local reliability calibration of data collected at load test sites is recommended because it has many advantages discussed in this section.

6.3.1 Advantages of Reliability Calibration over Calibration by Fitting

Some of the following advantages are demonstrated in Chapter 4.

- With calibration by fitting, uncertainties in loads and resistances are treated equally, with one safety factor selected mainly based on past experience and engineering judgment. The true level of risk or probability of failure is unknown and can be small or large. Under reliability-based LRFD design, load and resistance factors are evaluated based on scientific methods and are related to each other or tied through a prescribed reliability index selected to have a level of reliability or safety for foundations consistent with those employed in the structural design of bridge superstructures

- In the reliability calibration, design and construction uncertainties are considered and value of better geotechnical design or better construction of foundations can be captured in the calibrated resistance factors.
- The reliability calibration provides parameters to evaluate the economics and improve the accuracy of the calibrated design methods.
- With calibration by fitting to ASD methods, potential benefits are limited because practice won't improve upon what was done previously and possible conservative design methods would continue.

6.3.2 Advantages of Local over AASHTO Reliability Calibrations

The AASHTO LRFD reliability-based resistance factors were developed based on static load tests conducted at various locations in the United States and abroad where geological conditions and testing, design, and construction practices vary. This approach would lead to relatively large COV values and conservative ϕ values to be applicable and fit various conditions and practices ("one size fits all"). The local reliability calibration based on static load tests is recommended to account for a State's specific geology and its testing, design, and construction practices. This would narrow the range of calibration conditions and lead to more economical and reliable designs. The reliability calibration of several local and AASHTO LRFD resistance determination methods would generate resistance factors, ϕ, resistance mean bias, λ, and efficiency, ϕ/λ; for all these methods. These results would enable a DOT to compare all these methods and select or develop the most economical and accurate resistance determination method. Local reliability calibration is expected to lead to more economical and accurate LRFD geotechnical design methods than AASHTO LRFD methods and identify and address any conservatism of local ASD methods. The results of load tests can also be employed to evaluate various displacement determination methods and select or develop the most appropriate method.

As will be discussed in the next chapter, it is suggested that DOTs always consider load tests in the preliminary design for large bridge construction projects. The results of load tests generate more accurate design information and, in some cases, significant savings to the project. More important, as discussed above, load test data can be used in the local reliability calibration to develop more reliable, accurate and possibly economical LRFD geotechnical design methods.

6.4. SUMMARY AND CONCLUSIONS

For immediate implementation of LRFD, the DOTs need to consider two options for selection of LRFD geotechnical design methods: adopt AASHTO's LRFD methods or develop LRFD methods by fitting to local ASD methods that have track records of long-term success. These two options are fast and easy to implement. To select the most appropriate option, compare these options based on economics and reliability as discussed in this chapter. For the long-term implementation of LRFD, it is suggested that the DOTs develop local LRFD geotechnical design methods through reliability calibration of information collected at load test sites. This option will require an initial investment of more time and resources but the return will be significant in the future as it will lead to development of more reliable, accurate, and possibly economical design methods than the two short-term options discussed above.

When properly implemented, using LRFD methodology for geotechnical design will lead to savings or to equivalent foundation costs as compared with the ASD practices. The increases in foundation costs that have been reported by some agencies using LRFD design are primarily due to increases in design loads and improvements to the design methods and requirements that would also be applicable to the ASD platform if the AASHTO Standard Specifications continued to be updated.

The DOTs need to keep updating their LRFD design specifications based on future interims of AASHTO LRFD. Improvements to the AASHTO LRFD platform will continue in the future based on applied research studies, the results of additional load tests, and the experience of the highway community with implementing this methodology. The LRFD reliability calibration process is dynamic: it allows for continued refinement of the resistance factors with more data. In the future, it is expected that the geotechnical resistance factors will be increased and the focus will be shifted to the calibration of the service limit design methods.

CHAPTER 7

DEVELOPMENT OF LRFD DESIGN GUIDANCE FOR BRIDGE FOUNDATIONS

This chapter provides a roadmap to assist DOTs with the development of LRFD Design Guidance that consists of LRFD design specifications and delivery processes for bridge foundations.

7.1 DEVELOPMENT OF LRFD DESIGN SPECIFICATIONS

7.1.1 Materials Needed for Development

Development of the LRFD Design Manual by the DOT should follow the sequence presented in the AASHTO LRFD Design Specifications. First, develop sections to cover the LRFD design specification on loads and on hydraulic and structural designs based on AASHTO LRFD Sections 1 to 8 (AASHTO, 2010a). We strongly recommend that DOTs accept AASHTO's loads, load factors, and load combinations at various limit states (see AASHTO LRFD Section 3). Then, develop a section on LRFD design specification for bridge foundations based on AASHTO LRFD Section 10. This section should refer to the earlier sections on loads and on hydraulic and structural design, as is the case with AASHTO LRFD Section 10, which refers to AASHTO LRFD Sections 1 to 8.

To develop the LRFD design specifications for bridge foundations based on AASHTO LRFD Section 10 (2010a), the DOTs need to implement Steps 1 to 4 of the implementation plan presented in Chapter 2, and address these steps as discussed in Chapters 3 to 6. It is important to review the references presented in Chapters 3 to 6, which include the DOT current ASD design technical references for foundations, the most updated version of AASHTO LRFD Section 10 and related sections referenced within Section 10, FHWA and NCHRP manuals, and LRFD design manuals from neighboring and lead states. The outcomes for implementation of Steps 3 and 4 are the changes needed to transition from ASD to LRFD (as discussed in Chapters 3, 4, 5, and 6). Finalize the LRFD design methods to address all applicable structural and geotechnical limit states for all foundation types. Chapters 4, 5, and 6 provide suggestions on how to finalize the LRFD geotechnical design methods). It is important to identify and justify "Exceptions" from AASHTO LRFD Section 10 —deletions, additions, or modifications—and provide justifications for these exceptions: long-term successful past ASD experience/engineering judgment, research, and local issues not addressed in AASHTO LRFD. It is suggested that DOTs inform AASHTO of these exceptions. The AASHTO Subcommittee on Bridges and Structures has a process for resolving specification issues and identifying the need for special studies. Communication with

SCOB should be through the State bridge engineer, who would contact AASHTO. For more information, visit http://bridges.transportation.org/?siteid=34&pageid=339.

7.1.2 Roles and Responsibilities of Various Groups in the DOT

All members of a DOT's LRFD Implementation Committee (structural, geotechnical, hydraulic, and construction) should be responsible and work together for the development of the LRFD Design Specifications for foundations.

The geotechnical and hydraulic groups should also work together to develop the specifications related to scour. AASHTO LRFD Section 2 discusses the stability of foundations due to check flood (a flood not to exceed a 500-year event or an overtopping flood of lesser recurrence interval) and design flood (a flood of a 100-year event or an overtopping flood of lesser recurrence interval). Scour due to a check flood should be investigated under the Extreme Event limit state, and scour due to a design flood should be investigated under both the strength and service limit states. The FHWA Hydraulic Engineering Circular (HEC) 18 guidance is routinely used to evaluate scour depth on a national basis, but it is generally understood that in some cases the scour equations in this guidance yield overly conservative scour estimates because they were developed for granular soils. For additional guidance on scour in rock, refer to the FHWA Memorandum on scourability of rock formations (www.fhwa.dot.gov/engineering/hydraulics/policymemo/rscour.cfm), and to Article 10.4.6.6 of AASHTO LRFD. Deviations from the HEC 18 guidance on the estimation of scour depth, reduction of scour depth through the use of countermeasures (e.g., riprap), and the assumption of no scour at the abutments should be justified appropriately as previously discussed.

Additionally, geotechnical and hydraulic engineers should agree on a process to identify the highest groundwater level (GWL) expected over the life of the structure for consideration in the geotechnical design at various limit states. The difference between this GWL and the GWL at the time of design (or at the time of pile driving for the pile field design methods) leads to future axial and lateral geotechnical resistance losses (GL) that should be considered in the geotechnical design. A flood would elevate the GWL. It is suggested that the geotechnical engineer considers the: a) elevation for design flood provided by the hydraulic engineer in the determination of the highest ground water table (GWT) elevation needed in the analysis of service and strength limit states; and b) elevation for check flood provided by the hydraulic engineer in the determination of the highest GWT elevation needed in the analysis of the extreme event limit state with check flood.

The structural group will be responsible to develop specifications on foundation loads and structural design based on AAHSTO LRFD Section 10

The structural and geotechnical groups need to address common foundation design issues, such as:

- Earth loads, including vertical and lateral earth loads, downdrag loads, uplift loads due to soil swelling, and lateral spreading and downdrag loads due to liquefaction, and the soil and site information needed to estimate earthquake loads.
- Evaluation of the foundation factored loads that maximize the force effect for the geotechnical resistances at the strength limit.
- Design methods to evaluate both the structural and geotechnical limit states under lateral loading (e.g., the p-y method, strain-wedge method, and the Broms method).
- The service limit for all foundation types. Note that foundation displacements develop from both the structure and the geomaterials around the foundation. While the foundation factored loads and tolerable displacements are finalized by the structural engineer, the geotechnical engineer often computes the foundation displacements.
- For each foundation at each applicable limit, agree on the geotechnical and structural design requirements, and the structural and geotechnical resistances that need to be evaluated.

7.1.3 Contents of the LRFD Design Specifications

The geotechnical group should take the lead for development of the entire LRFD design specifications for bridge foundation and be responsible for the development of the geotechnical design specifications emphasized herein. As a starting point, the DOT may consider conversion of its ASD design specifications to LRFD (easier) or consider appropriate LRFD Design Specifications developed by other DOTs. In this development, the DOT must address all issues discussed in AASHTO LRFD Section 10 and applicable to the DOT and cover the local issues not covered in AASHTO. Suggestions to develop various components of these specifications are presented next.

Introduction (Based on AASHTO LRFD Section 10, Articles 10.5.1 to 10.5.4, Chapter 3 of this manual). In this section, address the limit states specific to foundations and their governing LRFD design equations. Present the governing ASD equations for foundations and discuss the similarities and differences between ASD and LRFD.

Note: AASHTO LRFD Articles 10.5.1 to 10.5.4 describe the structural and geotechnical resistances that should be addressed under each limit state, which we recommend to cover for each foundation under LRFD Design Methods discussed later.

Design Soil and Rock Properties (Based on AASHTO Article 10.4 and FHWA Manuals). In this section, cover the procedures to determine and select the soil/rock properties needed for the foundation geotechnical design methods at all applicable limit states (discussed later) and for construction of foundations. If these procedures are described in the DOT's LRFD Geotechnical Manual, make a brief reference to this manual in the LRFD Design Section on Foundations. Note that the AASHTO LRFD specifications allow for consideration of local experience and specific geology in the selection of soil and rock properties. Start by presenting the "Informational Needs" (as in AASHTO LRFD Article 10.4.1). The DOTs need to adhere to the minimum guidelines for a subsurface exploration program as described in AASHTO LRFD Article 10.4.2 to establish reliable soil/rock properties for the design and construction of foundations. Emphasize that the level of subsurface exploration in AASHTO LRFD Table 10.4.2-1 should be considered a ***minimum*** and should be increased based on past experience, the degree of site variability, and the importance of the structure. Next, define groups of geomaterials (in terms of measurable material properties if possible) for which geotechnical design methods are furnished in the LRFD design section. In addition to sand, clay, and rock, AASHTO provides static analysis methods for the design of drilled shafts supported by cohesive and cohesionless intermediate geomaterials. For each group of geomaterials, present the methods to determine the soil/rock properties from laboratory and in situ tests (as in AASHTO LRFD Articles 10.4.3 to 10.4.5). Cover also selection of design properties for each group of geomaterial (as in AASHTO LRFD Article 10.4.6):

- Summarize procedures for test interpretation (e.g., the use of computer programs to estimate the p-y curve for sand from the friction angle)
- Assessment of site variability (see Chapter 5)
- Selection of deformation and strength properties of soil and rock mass. Discuss the influence of special types of loads on these properties (wind load, seismic load, vessel impact load). Cover selection of properties for mixed local soils (see AASHTO LRFD) if they exist.
- Assessment of liquefaction and scour potential (under design and check floods).
- Soil properties needed in the estimation of earthquake loads.
- Properties of soil and rock needed for construction of the foundation.

Tolerable Movement Criteria (Based on AASHTO LRFD Article 5.5.5 and the 2006 FHWA Soils and Foundation Manual). The vertical and lateral displacements and rotation of the foundations lead to movements of the bridge superstructure at critical locations (e.g., the girder seat). For example, movement at the top of a pier would be amplified by differential settlement and rotation of the foundations. The permissible foundation movements should consider the type and configuration of the structure. AASHTO LRFD and FHWA manuals suggest the development of project-specific movement criteria for foundations as a function of tolerance of the bridge and the structures around the bridge (approach slab), rideability, economy (cost of future maintenance and repair), safety (clearance), and aesthetics. Therefore, each DOT should establish final guidelines for these movement criteria that are appropriate to its bridges, field conditions, and design requirements.

AASHTO LRFD indicates that transient live loads may be omitted in the time-dependent settlement analysis of foundations bearing on cohesive soils. Note also that the goal of the settlement analysis should be to determine the settlement of the bridge that influences its performance, not the settlement of the foundation. Therefore, certain loads should be considered in the estimation of the bridge settlement, not all the loads that lead to settlement of the foundation (see the 2006 FHWA Soils and Foundations Workshop Reference Manual).

Resistance Factors (Based on AASHTO LRFD Article 10.5.5 of AASHTO LRFD and Chapters 4 and 5). Present the resistance factors for all LRFD foundation design methods, and the methods considered in the developments of the geotechnical resistance factors. Provide justifications for selection of resistance factors different from those recommended by AASHTO. Present the conditions employed in the development of the resistance factors (either the AASHTO LRFD ϕ values or those developed locally). Emphasize that the designer need to adhere to these conditions in their design and justification is needed for any deviations from these conditions.

LRFD Design Methods at All Applicable Limit States (Based on AASHTO LRFD Articles 10.6 to 10.9, FHWA references discussed in this manual, Chapters 4 to 6 of this manual). For each foundation type (as in AASHTO LRFD Section 10), start by developing a General Section (e.g., AASHTO LRFD Article 10.7.1 for driven piles) that describes situations and conditions to be considered by the foundation designers in the selection of this foundation type. This will help the foundation designers select the most appropriate and economical foundation type. Also, in this section, cover design issues and requirements applicable to all limit states. Then, and for each foundation type, develop Sections for Service Limit Design, Strength Limit Design, and Extreme Event Limit Design (as in AASHTO LRFD Section 10). For each limit state design (as in AASHTO LRFD Section 10), discuss the design requirements, including the structural and geotechnical resistances that needs to be evaluated. Briefly describe the foundation loads and structural resistances, and refer the reader to other sections in the DOT's LRFD Design Manual for more specific details. As in AASHTO LRFD, elaborate more on some special earth loads (downdrag, uplift). Finally, present methods for estimating and evaluating the foundation nominal geotechnical resistances, R_n (or displacements for the service limit state). It is acceptable to reference AASHTO LRFD for these methods, but these references should be very specific since AASHTO LRFD often provides several design methods. Briefly describe the methods referred to in AASHTO LRFD and add specific directions for their use in local design practices. At the end, cover special design topics, such as those covered in Articles 10.5 to 10.7.9 of AASHTO LRFD for driven piles.

Construction Design Issues. The design specifications on bridge foundations shall briefly describe the foundation construction issues needed for the design of foundations and refer the reader to the foundation construction specifications on bridge foundations for more specific details. These construction specifications need to be consistent with the design specifications for bridge foundations and based on: i) The DOT's current construction specifications, which presents the DOT's experiences, practices and judgment with respect to construction of foundations; ii) The most updated version of AASHTO LRFD Bridge Construction Specifications (2010b); iii) FHWA manuals for design, construction, and inspection of various foundation types; and iv) the construction methods and inspection/testing methods for QC/QA procedures considered in the calibration of resistance factors.

In the LRFD design specifications for each foundation type, briefly describe the methods for construction and QC/QA assumed in the geotechnical design methods and the expected results from the QC/QA procedures (e.g., cleaned shaft bases for drilled shafts). This is needed to ensure that the assumed foundation geotechnical resistances in the design are met by the constructed foundations. For consideration of a construction or QC/QA method that is of lesser quality but still acceptable, the design section should describe the procedure for adjusting the foundation nominal geotechnical resistance value (not the resistance factor). Examples for drilled shafts: reduce the unit side resistance when permanent casing will be used, and reduce (neglect) base resistance when a cleaned base of the shaft hole could not be ensured. In many cases, the foundation design needs to be finalized during construction and this should be described in the design guidance. This would be the case when the foundation geotechnical design requires or is based on dynamic analysis methods for driven piles, static load tests, field verification of rock quality and strength, and minimum length of embedment of drilled shafts in the bearing soil/rock layer.

A project's design and construction specifications should address the possibility of construction and design problems encountered during construction, and the corrective measures needed during construction to address them. Foundation construction involves an inherent risk of encountering conditions that differ from those anticipated due to the complexity and variability of natural earth and rock formations and materials. Differing site conditions" (DSC) is a common source of contractor claims on highway construction projects. Federal law requires that a DSC clause be incorporated into all federal-aid highway projects. Geotechnical Engineering Notebook Issuance GT-15 was prepared to provide guidance to design and construction engineers to address DSC (see http://www.fhwa.dot.gov/engineering/geotech/policymemo/gt-15.pdf).

7.2 DEVELOPMENT OF LRFD DESIGN DELIVERY PROCESSES

Good references for development of LRFD design delivery process for various foundation types are:

- The NHI LRFD Manual on foundations (NHI, 2005) and FHWA LRFD Manuals (e.g., the 2010 Drilled Shaft Manual).
- Arizona DOT Design Memos (see http://www.ncsconsultants.com/).
- Washington State DOT LRFD Geotechnical Manual (see www.wsdot.wa.gov/fasc/EngineeringPublications/Manuals/GDM/GDM.htm).

7.2.1 Overview

The DOT needs to develop LRFD-based design delivery process for each foundation type consistent with its LRFD design specifications. For every type of foundation, translate the LRFD geotechnical and structural design specifications at various limit states into design steps and for each step define who will do what, when, and how." All members of a DOT's LRFD Implementation Committee (structural, geotechnical, hydraulics, and construction) should be responsible for the development of the foundation LRFD design delivery processes and work together to address all applicable structural and geotechnical limit states. Define the role of each group of the design and construction teams (structural, geotechnical, hydraulics, and construction). Identify the information needed from each design step and how it will be used in the following steps. Require the involvement of the geotechnical engineer in all phases of project development, from planning (i.e., site selection) to construction. Define the coordination, interaction, and meetings among the design and construction personnel throughout the project development, especially to resolve design and construction issues. And finally, employ advanced computer programs available for the analysis and design of foundations at all limit states. These programs can analyze several load combinations quickly and more accurately than conventional methods.

Specific recommendations for the roles and responsibilities of various groups in the DOT are presented next.

7.2.2 Role of the Construction Group

A foundation design should not be considered complete until foundation construction has been successfully completed. The role of the construction engineer is essential in finalizing design during construction (as discussed in the previous section) and ensuring that all field design assumptions for construction methods and quality of construction are met during construction. The construction engineer needs to report construction problems and work with the design team to develop corrective measures. Finally, the construction engineer needs to be involved in the Implementation of LRFD and document any problems encountered (as discussed previously).

7.2.3 Roles of the Geotechnical and Hydraulic Groups

Many DOTs determine the scour depth of deep foundations at both the design flood and the check flood in the early stages of the project and before the subsurface exploration program is performed. We suggest changing that and initially perform the subsurface exploration program. Then, the project geotechnical and hydraulic engineers need to work together to finalize the scour depth at all limit states, and the GWT at all limit states with consideration of the elevation of the design and check floods.

7.2.4 Roles of Structural and Geotechnical Groups

7.2.4.1 Preliminary Design Phase

Some DOTs perform the subsurface exploration program in the early stages of project development, before the exact location of the bridge is finalized, and some DOTs perform this investigation after the bridge's exact location is finalized but also after the types of foundations for the bridge are selected. There are problems with both alternatives. For major structures, the new FHWA Manual on Drilled Shafts (Brown et. al., 2010) 2010) recommends that DOTs divide the field exploration program into two phases: a preliminary phase (based on a few borings and geophysical tests) that provides sufficient information for planning and optimizing the subsequent, more comprehensive investigation phase. A staged investigation program provides sufficient information for preliminary design (including preliminary selection of foundation types) and defers much of the cost of the site investigation until the structure's exact location and layout is finalized. The preliminary phase can be performed during the Environmental Impact Statement phase of the project.

After the preliminary subsurface exploration program is completed, it is suggested that the entire design team (structural, geotechnical, construction, and hydraulic engineers) meet and agree on the types of foundations and design methods that need to be evaluated <u>in the preliminary and final design phases.</u> Always consider spread footings as a viable foundation system, and a static load test as an alternative design method. The subsequent design steps should determine if spread footings and a static load test are appropriate or not.

For all the alternative foundation types, the structural engineer should provide the following preliminary design information to the geotechnical engineer:

- Preliminary foundation factored loads (or a range of factored loads) at all applicable limit states. For deep foundations, this would include the range of factored axial and lateral loads that will be supported by a group of deep foundations and by a single deep foundation at each applicable limit state.
- Preliminary foundation sizes and any foundation structural information that the geotechnical engineer may need in the preliminary design. This would include the length of a shallow foundation, sizes of driven piles, and diameters of drilled shafts. The sizes of the deep foundations (e.g., shaft diameter, or HP 12 x 53 for a driven pile) explored in the design phase are often selected based on their structural capacities and expected loads.

For all the alternative foundation types, and based on the results of the subsurface investigation and analysis, the geotechnical engineer needs to prepare the following information:

- The design soil/rock properties that are needed in the geotechnical design of foundations at all applicable limit states (including information on scour and liquefaction).
- Information needed by the structural engineer for estimation of the earth loads and earthquake loads.
- Information on subsurface conditions necessary for the construction of foundations.
- The foundation geotechnical design methods and their resistance factors (with the static load test as one of the alternative design methods).

In some DOTs, the geotechnical engineer has no role in the foundation design. However, it is recommended that the geotechnical engineer also perform a preliminary design to assist the structural engineer with selection of trial widths for shallow foundations and trial depth and number of a group of deep foundations as suggested next.

For shallow foundations on soils at a given depth, develop a "Bearing Resistance Chart" relating foundation effective width to the factored axial geotechnical resistance available to support the applied axial factored loads at both the strength and service limit states. This chart may be developed at different elevations to allow selection of the most cost-effective footing depth/elevation.

For a single pile or shaft at the strength limit, develop a plot relating foundation penetration depth, L, to the factored axial or lateral load that can be supported at that depth, Q_f. This plot should cover the expected range of foundation factored loads acting on a single pile/shaft. For a single driven pile, establish the maximum axial factored load a single pile can support, Q_{fmax}, and the maximum penetration depth to which the pile can be safely driven, L_{max}. For a single deep foundation element under lateral loading, establish the maximum lateral factored load that the shaft or pile can support and its corresponding depth.

As an example, the predicted top axial factored load, Q_f, vs. pile length from various design methods up to Q_{fmax} and L_{max} are shown in Figure 7.1. This figure demonstrates the benefits of static load testing (e.g., shorter piles or a smaller number of piles). Other benefits of the static load tests were discussed previously and include providing data for the calibration of resistance and displacement determination methods. The geotechnical engineer needs to discuss these benefits with the structural and construction engineers and finalize the decision of whether to include a load test for the project. If it is feasible, conduct the static load test in the design phase.

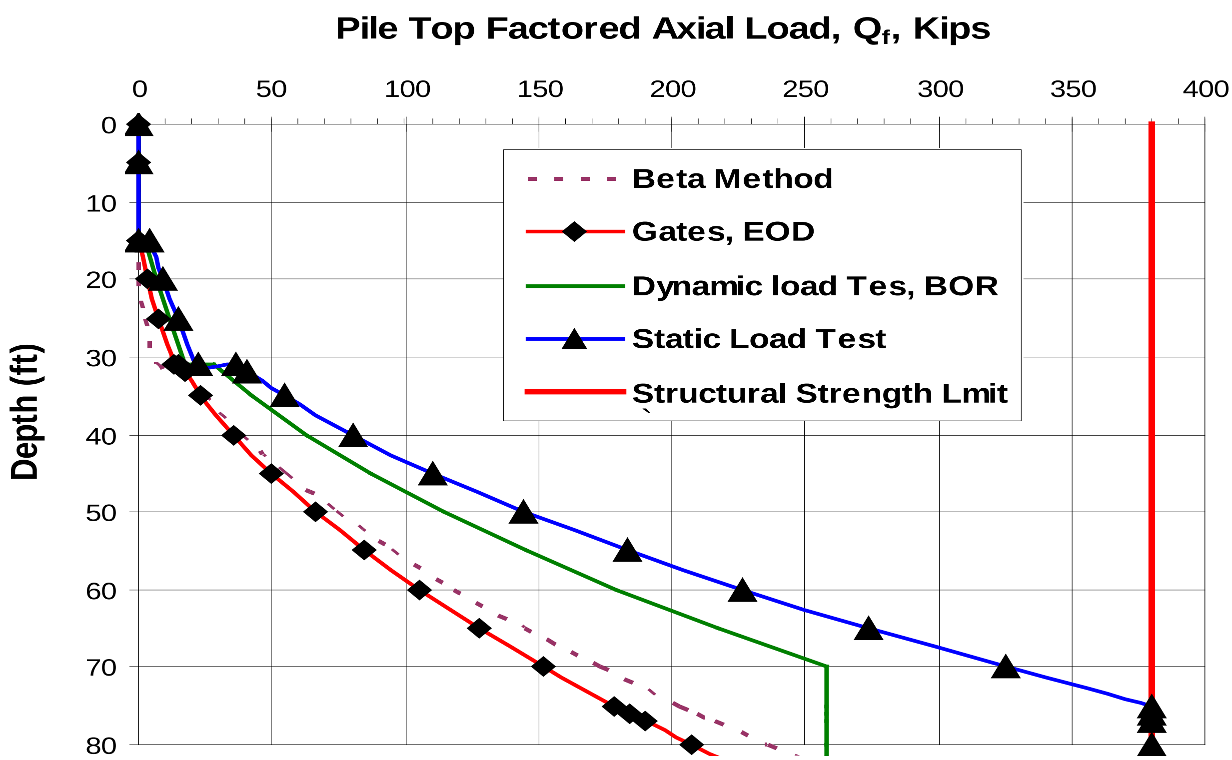

Figure 7.1. Example of the Information the Geotechnical Engineer Needs to Develop in the Preliminary Design

Note: The initial penetration length of a group of deep foundations needs to be estimated based on the limit state that most likely would control or govern the penetration length of a single pile/shaft, which is in most cases is the strength limit state, but in some cases could be the service limit state or one of the extreme event limit states.

7.2.4.2 Final Design Phase

The information provided in Phase I will help the structural engineer to select the *initial trial* width for shallow foundations, size and type of driven piles, diameter of shafts, and depth and number of a group of driven piles and drilled shafts. This and the cost information for the alternative foundation types and for the load test will help in the selection of the most cost-effective and appropriate foundation type and deciding if conducting a load test is appropriate. In the final LRFD design, the structural engineer needs to ensure that all applicable foundation structural and geotechnical limit states are met. The geotechnical, hydraulic, and construction engineers should have a formal role in the review of the final design.

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