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SOUTH DAKOTA
M
SCHOOL OF MINES & TECHNOLOGY

Review and Refinement of SDDOT's LRFD Shallow Foundation Design Method

Study SD2014-11

Final Report

Prepared by

South Dakota School of Mines and Technology

Rapid City, SD 57701

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16. Abstract
The objective of this project was to conduct a comprehensive review of Load Resistance Factor Design (LRFD) for bridge and wall shallow foundations. LRFD is a limit state design methodology based on reliability and probability. Specifically, this study investigated the concept, application, and implementation status of LRFD in shallow foundation designs for the South Dakota Department of Transportation (SDDOT) based on a review of the published research papers, American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, bridge design manuals and/or geotechnical manuals of instruction (MOI) from other states, research reports published by many US state and federal agencies including Federal Highway Administration (FHWA), National Cooperative Highway Research Program (NCHRP), and the National Highway Institute (NHI), in addition to specifications used internationally. The results of this study show that the LRFD approach has already been adopted by most US states due to mandatory use on federally funded projects. However, whereas deep foundation design parameters are relatively well established for LRFD methods, local calibration of shallow foundation design parameters has not yet been performed in most states. This report examines the current status of LRFD and discusses its limited level of implementation for shallow foundation design in South Dakota, provides a set of recommendations that SDDOT can consider as it pursues implementation of LRFD in its construction projects with the ultimate goal of economical designs incorporating quantitative estimations of failure.

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TABLE OF ACRONYMS

Acronym	Definition
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway Transportation Officials
ASD	Allowable Stress Design
BL	Blast Loading Force Effects
BR	Braking Force Effects
CALTRANS	California Department of Transportation
CE	Centrifugal Force Effects
CFEM	Canadian Foundation Engineering Manual
CPT	Cone Penetration Test
CR	Creep Force Effects
CT	Collision Force Effects
DA	Eurocode Design Approach
DC	Structural Component Dead Loads
EH	Horizontal Earth Pressure Loads
EQ	Earthquake Load Effects
EV	Vertical Pressure Caused by Earth Fill Dead Loads
ES	Earth Surcharge Loads
EL	Construction Process Force Effects
EU	European Union
FHWA	Federal Highway Administration
FR	Frictional Loads
GRS	Geosynthetic Reinforced Soil
IBS	Integral Bridge System
IC	Ice Loads
IGM	Intermediate Geomaterial
IM	Impact Allowance
LL	Live Loads
LRFD	Load and Resistance Factor Design
LS	Live Load Surcharge

Acronym	Definition
LSD	Limit State Design
MnDOT	Minnesota Department of Transportation
MoDOT	Missouri Department of Transportation
MOI	Manual of Instructions
MSE	Mechanically Stabilized Earth
NAVFAC	Naval Facilities Command, United States Department of Navy
NBCC	National Bridge Code of Canada
NCHRP	National Cooperative Highway Research Program
NHI	National Highway Institute
PL	Pedestrian Loads
PMT	Pressuremeter Testing
PS	Post Tensioning Secondary Forces
RBD	Reliability Based Design
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SD	South Dakota
SDDOT	South Dakota Department of Transportation
SDSMT	South Dakota School of Mines and Technology
SE	Settlement Force Effects
SLS	Serviceability Limit State
SPT	Standard Penetration Test
TG	Temperature Gradients
TRB	Transportation Research Board
TU	Uniform Temperature Effects
UC	Unconfined Compression
UDOT	Utah Department of Transportation
ULS	Ultimate Limit State
US	United States
WA	Water Loads and Force Effects
WASHDOT	Washington Department of Transportation

Acronym	Definition
WL	Wind Live Load
WS	Wind Load on Structure
WSD	Working Stress Design

1.0 EXECUTIVE SUMMARY

The objective of this project was to conduct a comprehensive review of Load Resistance Factor Design (LRFD) for bridge and wall shallow foundations. LRFD is a limit state design methodology based on reliability and probability. Specifically, this study investigated the concept, application, and implementation status of LRFD in shallow foundation designs for the South Dakota Department of Transportation (SDDOT) based on a review of the published research papers, American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, bridge design manuals and/or geotechnical manuals of instruction (MOI) from other states, research reports published by many US state and federal agencies including Federal Highway Administration (FHWA), National Cooperative Highway Research Program (NCHRP), and the National Highway Institute (NHI), in addition to specifications used internationally. The results of this study show that the LRFD approach has already been adopted by most US states due to mandatory use on federally funded projects. However, whereas deep foundation design parameters are relatively well established for LRFD methods, local calibration of shallow foundation design parameters has not yet been performed in most states. This report examines the current status of LRFD and discusses its limited level of implementation for shallow foundation design in South Dakota, provides a set of recommendations that SDDOT can consider as it pursues implementation of LRFD in its construction projects with the ultimate goal of economical designs incorporating quantitative estimations of failure.

1.1 Introduction

Allowable stress design (ASD), also known as working stress design (WSD), has been a widely accepted engineering design paradigm for decades around the globe. This design concept is based on the idea that ultimate strength of a material, structural member, or system is reduced by applying a reduction factor known as the factor of safety, which has been consensually accepted based on 1) shared experience for certain representative characteristics of a particular material and system, 2) defined by an expert panel for a predetermined level of risk, or 3) may be arbitrary. However, the major drawback of this approach is its lack of quantitative assessment. This lack of quantitative assessment in ASD has led to the development of LRFD that nominally incorporates reliability and probability of failure with resulting consequences into designs. LRFD has become normal in geotechnical and structural engineering designs all over the world due to its probability-based assessment of safety margins. FHWA has made the use of LRFD mandatory on all federal funded construction projects since October 1 2007 (Densmore 2000), and AASHTO has provided nominal LRFD load and resistance factors in the national bridge design code for the US.

However, the adoption of LRFD in geotechnical-based design projects has not been as universal as in structural engineering, because of 1) the high level of geographical variability and natural uncertainties in soils and rock, and 2) the lack of singular universal calculation methods for even the most standard design calculations that applies to all possible geologic conditions. Soil is a heterogeneous and anisotropic material that changes with both time and space. Calculation methods are developed under certain constraints of applicable geologic conditions. In general, a more reliable estimation of the resistance factors for geotechnical LRFD calculations is required and this can only be achieved by reviewing previous design parameters that are implemented in the wide range of calculation methods. To take full advantage of the benefits of the LRFD approach in geotechnical design, it is strongly recommended that the accuracy and reliability of the design methods and input parameters required in LRFD be reviewed. Although SDDOT has been complying with the regulations for LRFD use on FHWA partial to full funded projects, the use of LRFD for shallow foundations has been minimal, as in many other states, due in part to the limited information available on how best to ensure a smooth transition from the ASD concept that geotechnical engineers traditionally use to this new design concept. Shallow foundation use by SDDOT and other states for bridge structures is also limited by a variety of other factors including bridge scour. This report reviews the current state-of-the-art and suggests possible approaches for implementing LRFD for shallow foundation projects in South Dakota.

1.2 Objectives

The objectives of this research are:

1) Review current SDDOT shallow foundation design procedures, including field investigative techniques and laboratory analyses.
2) Recommend refinements to SDDOT's current shallow foundation procedures and processes based on the results of the review and a thorough analysis of resistance factors.

1.3 Research Approach

To accomplish the two main objectives, the research team reviewed various types of design documents and publications related to the design of shallow foundations. First, a thorough review of the design procedures currently used for shallow foundations in South Dakota (SD) was conducted, and the foundation design information for previous shallow foundation projects in the state examined. The soil test methods utilized, and the results obtained, as well as the designs used for each project, were considered. In addition, the research team conducted a comprehensive review of the relevant research literature and evaluated the approaches used by other states that are dealing with similar concerns regarding applying LRFD principles to shallow foundations.

After the tasks listed above for the first objective were completed, the research team analyzed the data obtained and identified that there is a lack of suitable data available for the refinement of the resistance factors, a vital aspect of LRFD, not only in South Dakota but also in the nation. This has led most states to simply adopt the resistance factors provided in the AASHTO LRFD Bridge Design Specifications without conducting any local calibrations. The research team considers it essential that further data on local materials and their properties must be collected for refinement and local calibration of the LRFD approach if it is to fully achieve its potential economic benefits for the State of South Dakota.

1.4 Conclusions

The research team reviewed the fundamental principles of LRFD, and their applications and implementations in shallow foundation designs. In addition, current implementation status of LRFD at various levels of jurisdictions (from states to countries) were compared. Intensive literature reviews on the state level implementation status across the US were conducted and the comparisons between major state implementation efforts were made. These results showed that most states use the nominal AASHTO LRFD (2017) resistance factors. Shallow foundations are actively being used for bridge construction in only a few states and generally only when the bridge is founded on shallow competent rock. Since most of the current foundation design practice for bridge structures is on rock, using AASHTO resistance factors instead of conducting local calibration is generally considered to be low risk and cost effective. Very few instances of shallow foundations for bridge structures on soils were identified, with even less construction monitoring data which is needed to develop a locally calibrated set of LRFD resistance factors.

Furthermore, from the investigation on 27 previous foundation designs performed on 22 highway projects in SD, only two bridge projects utilized shallow foundations, and both of these were constructed on rock. In both cases, the properties of the rock were given in a descriptive manner and no test data was available for the rock properties. Likewise, no construction performance data was collected or documented for these sites; data that is required to develop LRFD resistance factors. In addition, 77% of the reviewed projects were

Mechanically Stabilized Earth (MSE) walls and the results of bearing capacity design calculations were only available for 5 of these MSE wall projects. No bridge foundations in the collected database were integrated abutment systems such as Geosynthetic Reinforced Soil – Integrated Bridge Abutment Systems (GRS-IBS), although SDDOT has constructed one in the Custer, SD area. Most importantly, there was no load test data provided that could be used to evaluate the performance of the designs. Therefore, the research team was not able to refine the resistance factors in LRFD but instead reviewed what could be done and made recommendations for setting up a future refinement plan.

1.5 Recommendations

Due to the limited information available from previous projects that have incorporated shallow foundation design, refinement of LRFD resistance factors was not possible. However, it is recommended that SDDOT prepare a plan to collect design parameters for state projects to support future opportunities to conduct such refinements. The research team recommends that:

1.5.1 SDDOT use AASHTO LRFD (2017) Bridge Design Specifications Resistance Factors Adjusted for Site-Specific Uncertainty

SDDOT use AASHTO LRFD (2017) Bridge Design Specifications resistance factors of shallow foundation designs until appropriate local calibration and development of SDDOT specific resistance factors has been conducted. These should be adjusted for site-specific uncertainty.

1.5.2 SDDOT Quantify Intermediate Geomaterials on Projects

SDDOT implement quantification of rock, soil, and Intermediate Geomaterials (IGM) statewide for use in shallow foundation design in a future research project.

1.5.3 SDDOT Implement Rock Mass Rating (RMR) Procedures in Shales and Rock

SDDOT implement RMR shale characterization procedures to augment the current usage of Unconfined Compression (UC) tests in shales.

1.5.4 SDDOT Compile a Centralized Foundation Design Database

SDDOT standardize the geotechnical exploration process and foundation design input parameter development for soil and rock for projects within the state and collect into a centralized database of data and parameters, which will be an essential resource for future calibration/refinement.

1.5.5 SDDOT Compile a Centralized Foundation Performance Database

SDDOT instrument and monitor bridge foundation and MSE wall settlements and develop a centralized database of shallow foundation performance that includes foundations and walls monitored previously by SDDOT and in neighboring states.

1.5.6 Full Scale Shallow Foundation Loads Tests

SDDOT perform a set of well instrumented full-scale shallow foundation load tests, to failure, to add critical collapse condition data points to the foundation performance dataset.

1.5.7 SDDOT Develop SD Specific LRFD Resistance Factors

SDDOT use the centralized database of geology, geologic data, laboratory data, field monitoring and performance, and load tests to develop a set of South Dakota specific LRFD resistance factors for shallow foundations on rock, soil, or IGM.

1.5.8 SDDOT evaluate the use of more shallow foundations systems for bridge structures for bridges without scour hazards, wherein foundations are placed on reinforced structural fill.

SDDOT evaluate use of innovative shallow foundation systems such as those with geosynthetic reinforcement of structural fill below footings.

1.5.9 SDDOT utilize layered bearing capacity equations for cases of over-excavation and replacement above weak subgrades; and

SDDOT utilize layered bearing capacity equations for cases of over-excavation and replacement above weak subgrades. Current SDDOT shallow foundation design procedures for estimating factored bearing capacities are only single-layer calculations. These methods allow the high friction angles of structural fill to be used for the upper layer, and the weak subgrade material properties are used as in current design methods. These methods are often able to raise nominal and factored bearing resistances by up to 50%, with increases of up to 200% possible!

1.5.10 SDDOT place more emphasis on settlement and deflection in shallow foundation design.

SDDOT currently places almost all design emphasis on the development of factored bearing resistances for shallow foundations. However, shallow foundation performance in the field and in case histories is generally governed by Settlement or deflection of the foundation. SDDOT could place more emphasis on settlement estimates in their design work for shallow foundations.

2.0 PROBLEM DESCRIPTION

The first set of standard specifications for highway bridges was published in 1931 by the then American Association of State Highway Officials (AASHO), the predecessor to AASHTO, and has been updated at intervals as recommended by panels from FHWA and the Transportation Research Board (TRB) along with state agency suggestion (Lwin 1998). Two contrasting conditions have motivated the development of more reliable and cost-effective design methods and/or design criteria: 1) failures, and 2) cost-ineffective but very safe structures. The quest to develop design methods and criteria that transcends these two contrasting conditions has led to the proposal of concepts that directly or indirectly incorporate reliability and probability. Limit State Design (LSD) is one such method.

LSD was first introduced for the Union of Soviet Socialist Republic's building regulations in 1955 and was rapidly applied in many different countries including Canada and most of Western Europe under the name of LRFD. In 1986, state bridge engineers from California, Colorado, Florida, Michigan, and Washington began to express their concern that the AASHTO Standard Specifications for Highway Bridges was lagging behind the times as they did not incorporate the LRFD philosophy that was already being widely applied both internationally in bridge design, but also in other areas of structural engineering in the United States (US). This led to a National Cooperative Highway Research Program (NCHRP) study and the first edition of the AASHTO LRFD Bridge Design Specifications was released in 1994. The LRFD framework has been developed further, and the 8th edition of the specifications was published in 2014 with interim provisions added annually thereafter. The Federal Highway Administration (FHWA) made the use of LRFD for the design of new bridges on all federally funded projects mandatory from October 1, 2007 onwards (Densmore 2000), requiring that the approaches to the bridge that include retaining walls, bridge superstructure, and foundations should all be designed using LRFD. Currently, LRFD is implemented in all transportation related engineering designs for bridge and wall infrastructure projects in South Dakota.

Unfortunately, the adoption of LRFD in geotechnical-based design projects has not been as "easy" as in structural engineering, largely because of 1) the high level of geographical variability and natural uncertainties in soils and 2) the lack of a consensus methods for geotechnical designs in all geologic conditions which also contain inherent variability that may be difficult to constrain. Soil is a heterogeneous and anisotropic material that changes with time, location, and space. In general, more reliable resistance factors for geotechnical LRFD are required and this must be achieved by reviewing previous design parameters, case histories of failures, and instrumented full-scale tests or construction monitoring. However, in

South Dakota there is a paucity of case histories, full scale testing, and instrumented foundations limiting development of local LRFD geotechnical load factors to a review of geotechnical investigative techniques, soil mechanics parameters, and design inputs. Review of the variability and uncertainty in the design calculation methods themselves is beyond the scope of typical DOT implementation.

To take full advantage of the benefits of the LRFD approach in geotechnical and foundation design, it is strongly recommended that the accuracy and reliability of both the ASD and LRFD design methods, as well as those of the parameters required for design calculations, be reviewed. As an active member of AASHTO, SDDOT shares common national design standards for its state highway system and is required by the FHWA to comply with their regulations. Although SDDOT has conducted a successful research project on the geotechnical deep foundations LRFD method, the use of LRFD for shallow foundations has been minimal, as in many other states, in part due to the limited information available on how best to ensure a smooth transition from the ASD concept that geotechnical engineers traditionally use to this new design concept. Shallow foundation use is also limited by a number of other factors including settlement, scour, and stability concerns. This report reviews the current state-of-the-art and suggests possible approaches for implementing LRFD for shallow foundation projects in South Dakota.

3.0 RESEARCH OBJECTIVES

The objectives of the proposed research are to:

3.1 Review SDDOT shallow foundation design procedures

Review current SDDOT shallow foundation design procedures, including field investigative techniques and laboratory analyses

The research team reviewed the current design procedures for shallow foundations in South Dakota. As the evaluation and reliability of the resistance factors are the most critical aspects of the method, the team reviewed the relevant parameters that affect the resistance factors including, but not limited to, laboratory and in-situ test procedures as well as the results deemed essential for identifying the reliability of the soil properties utilized.

In addition, the research team reviewed the relevant research literature and evaluated the approaches used by neighboring states (such as MN, ND, WY, NE, MT, & IA), all of whom deal with similar situations as South Dakota, as well as other leading states, to deal with the application of LRFD principles to shallow foundations. As LRFD has been officially implemented since 1994, it is extremely helpful for SDDOT to review the materials published by those states and learn from their extensive experience in using the method for shallow foundations.

3.2 Recommend refinements to SDDOT's current shallow foundations procedures

Recommend refinements to SDDOT's current shallow foundation procedures and processes based on the results of the review and a thorough analysis of resistance factors.

After the tasks listed above for the first objective had been completed, the research team reviewed and are providing recommendations herein for refining SDDOT's current shallow foundation design procedures and processes using LRFD, including the analysis of the resistance factors, to ensure compliance and consistency with the FHWA LRFD implementation plans. However, it is important to point out that as the calibration of resistance factors usually requires a large amount of high-quality test data, inadequate available data may limit the team's ability to perform an appropriate calibration. Instead, refinements will be made with any available resources that are suitable to perform the refinements for the resistance factors. This review has been performed without examination of case histories or full-scale testing/instrumentation.

Each project objective has been accomplished through a series of project tasks, described in more detail in the next chapter of this proposal.

4.0 TASK DESCRIPTIONS

As per the SD 2014-11 Research Project Statement provided by the South Dakota Department of Transportation (SDDOT), 8 tasks were identified to complete this project by the research team. The project research team consists of personnel from the South Dakota School of Mines and Technology (SDSMT).

4.1 Project Scope Review

Meet with the project's technical panel to review the project scope and work plan.

Project staff met with the project's technical panel and reviewed the project scope and work plan shortly after award to SDSMT. This meeting took place in Pierre, SD.

4.2 Interview Stakeholders

Conduct interviews with key SDDOT stakeholders to review SDDOT shallow foundation design methodology and procedures, including field testing procedures, lab testing procedures, sampling procedures, and use of resistance factors.

Project staff interviewed SDDOT bridge design engineers and SDDOT foundation design engineers within the first several months of the project. A second round of interviews was held in the summer and fall of 2018 when project staff had changed.

4.3 Literature Review

Review the literature and evaluate approaches used by neighboring states (MN, ND, WY, NE, MT, and IA) related to application of LRFD principles to shallow foundations and identify opportunities where their application may benefit SDDOT methodology.

An extensive literature review was performed for a year after task 4.2 was initially completed. This literature review is presented in detail in Chapter 5 of this report.

4.4 Review Resistance Factors

Review resistance factors applied to shallow foundations in South Dakota and make recommendations for refinements to code values where applicable.

A review of project files made available by SDDOT engineers in Pierre was performed. A review of the resistance factors was performed based on the available project files and information from interviews performed in Task 4.2. A set of recommendations based on the review are presented in this report in Section 6.

4.5 Interim Report

Draft an interim report based on findings from Tasks 2, 3, and 4.

A draft report was prepared and submitted to SDDOT in August of 2017. SDDOT and the project technical panel reviewed the Interim Report, with comments provided to the Project Team in 2018.

4.6 Meet with the Technical Panel

Meet with the Technical Panel to review and approve interim results and recommendations, and to confirm project direction.

Once comments were received from SDDOT and the technical panel in 2018, phone call meetings were held in the fall of 2018 with SDDOT and key stakeholders/design engineers to clarify the comments and provide the project team with additional insights needed to finalize the report.

4.7 Prepare a Final Report

Prepare a final report and executive summary of project results, findings, conclusions, and recommendations including opportunities for optimization of current practices and potential impacts to existing field, lab, and design procedures.

A revised final report was submitted to SDDOT in late 2018. SDDOT provided a small number of additional comments and edits. This final report was reviewed by SDDOT a second time by other staff in January of 2019 and again in December of 2019. This final report is issued in July of 2020.

4.8 Executive Presentation

Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

An executive presentation to the SDDOT Research Review Board occurred September 1, 2020.

5.0 FINDINGS AND CONCLUSIONS

5.1 Background

Designing a foundation requires an iterative process between structural and geotechnical engineers. Development of design loads on a foundation is normally conducted by structural engineers before the geotechnical design is performed, and these load magnitude and locations are then given to geotechnical engineers to check the stability of the foundation with respect to the strength limit and service limit states. On occasion the geotechnical engineer must perform their calculations before the structural engineer can design the structure. In almost all cases, there is some back and forth as the changes in structural and geotechnical design affect one another. With the LRFD approach, the factored load is determined by selecting appropriate limit states and load factors. The load factor varies for each type of load, and the different magnitudes of the factored load can be obtained by choosing different combinations of loads and load factors. Finally, the load, resistance, and load modifier factors need to be adjusted to account for variability in the load, material properties, construction, model error, acceptable risk to the public and failure consequences (Fenton and Griffiths 2008). To properly apply LRFD, both loads, and resistances are adjusted based on their different probabilities of occurrence and acceptable level of risk to the public. As stated earlier, the major change when converting from ASD to LRFD is the need to incorporate load factors to increase the applied load to a factored load and a resistance factor to decrease the nominal resistance to a factored resistance. It is essential that users are aware that defining appropriate loads and resistances and selecting the correct load and resistance factors for given design conditions are crucial if LRFD is to successfully achieve reliability-based design.

5.1.1 Ultimate/Nominal Strength of Shallow Foundation

The ultimate strength of the geologic material beneath a shallow foundation can be estimated or determined using one of six method types. This ultimate strength is the maximum strength that the geologic material can provide to resist structural loads before a failure (aka limit-state) occurs. As no deliberate precautions to increase safety are added to these method types the strength is "nominal" and requires further modification for safe and reliable design. The six method types are as follows.

5.1.1.1 Theoretical Estimation

The bearing capacity of soil depends on the strength and compressibility properties of the soil, embedment below the ground surface, groundwater conditions, direction of the load onto the foundation, and the dimensions of the structure, as the support comes from the contact between the foundation and the soil. For a set of given soil conditions, engineers estimate the ultimate capacity (also known as the nominal or unfactored capacity) of the soil and apply a factor to allow for uncertainties in the foundation capacity and reduce risk to the public.

(1) Bearing Capacity Equations

Terzaghi (1943) suggested a theory to model the ultimate bearing capacity of shallow foundations, proposing a bearing capacity equation that can be applied for strip, square, and circular foundations with appropriate bearing capacity factors:

$$\boldsymbol{q_{ult} = \alpha c N_c + q N_q + \frac{1}{2}\beta\gamma B N_\gamma} \qquad \textbf{(5.1)}$$

where:

N_s, N_q, N_γ: Terzaghi's bearing capacity factors

c = the apparent cohesion of the soil

q = the effective overburden stress at the bearing level of the foundation

γ = the soil's unit weight

B = The width of the foundation

α=1.0, 1.3, and 1.3 and β =1.0, 0.8 and 0.6 for strip, square and circular foundations, respectively.

Due to the assumptions and simplifications made for the equation, this equation is not appropriate for rectangular foundations, deeper embedded depths, inclined loading, and a number of other cases, so to address these limitations, Meyerhof (1963) suggested the generalized bearing capacity equation shown below, which factors in the effects of shape, embedded depth, and inclined load:

$$\boldsymbol{q_{ult} = c N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}} \qquad \textbf{(5.2)}$$

where:

N_s, N_q, N_γ: Meyerhof's bearing capacity factors

$F_{cs}, F_{qs}, F_{\gamma s}$: shape factors

$F_{cd}, F_{qd}, F_{\gamma d}$: depth factors

$F_{ci}, F_{qi}, F_{\gamma i}$: load inclination factors

with other terms as defined in Eq. 5.1

A similar form of the equation accounting for the effects of footing shape, ground surface slope, base inclination, and inclined loading is suggested as follows in *Standard Specifications for Highway Bridge* (AASHTO 2002).

$$\boldsymbol{q_{ult} = cN_c s_c b_c i_c + qN_q s_q b_q i_q + \frac{1}{2}\gamma BN_\gamma s_\gamma b_\gamma i_\gamma} \quad \textbf{(5.3)}$$

where:

N_s, N_q, N_γ: Meyerhof's bearing capacity factors

s_c, s_q, s_γ: footing shape factors

b_c, b_q, b_γ: base inclination factors

i_c, i_q, i_γ: load inclination factors

with other terms as previously defined in Eq. 5.1

Methods for determining these factors, as well as additional special cases that take into account variables such as the effect of water table or eccentric loading can also be considered in the foundation design and are available in the literature (e.g. AASHTO 2002; Das 2010).

AASHTO LRFD Bridge Design Specifications (2017) uses the term "nominal bearing resistance" to describe the estimated maximum resistance that soil can exert before general shear failure occurs. Nominal bearing resistance is fundamentally the same as ultimate bearing capacity in the allowable stress design (ASD) methodology. An appropriate resistance factor is then applied to the nominal bearing resistance to yield a factored bearing resistance as illustrated in Eq. 5.4.

$$\boldsymbol{\sum \eta_i \gamma_i Q_i \leq \Phi R_n = R_r} \quad \textbf{(5.4)}$$

where:

R_n= nominal bearing resistance

Φ_b = Resistance factor

R_r= factored resistance (capacity)

γ_i = load factor for a given load and load combination that accounts for uncertainty in nominal design loads

Q_i = nominal load of a given type (e.g. dead, live, etc.) that acts on the foundation

η_i =Load modifier factor that relates to ductility, redundancy, and operational importance

As with the procedures to determine ultimate bearing capacity in the ASD methodology, shear strength parameters that adequately represent the shear strength of the soil under given loading conditions should be obtained by conducting appropriate tests and used for the analysis.

A basic formulation for the theoretical estimation for the nominal bearing capacity, in units of ksf, of a soil layer is presented as Equations 5.5-5.6, the formulations described in (Munfakh et al. 2001). This is a similar format to that given earlier in Eq. 5.2 for the ASD but with different factors to take into account the foundation shape, inclination of the load, and location of the water table.

$$\boldsymbol{q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + \frac{1}{2}\gamma B N_{\gamma m} C_{wq}} \quad \textbf{(5.5)}$$

where:

$N_{cm} = N_c s_c i_c$

$N_{qm} = N_q s_q d_q i_q$

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$

C_{wq} = Groundwater depth correction factor

D_f = Depth of the bottom of the footing below ground surface

with other terms as previously defined in Eq. 5.1

Details of the above parameters and their values are available in Section 10.6.3.1.2 in the AASHTO LRFD (2017).

Section 10.6.3.1.2 of AASHTO LRFD (2017) comments that many geotechnical engineers have not used the load inclination factors in their analyses partly because of their lack of knowledge of the vertical and horizontal loads at the point when the geotechnical explorations and preparation of bearing resistance recommendations are being prepared. Also, the resistance factors for geotechnical resistances at the strength limit state in Section 10.5.5.2.2 of AASHTO LRFD (2017) were derived specifically for vertical loads, even though the soil supporting a footing on an inclined base will experience both vertical and horizontal load combinations. AASHTO LRFD (2017) states that it is not yet known how applicable those resistance factors are to footing designs that resist inclined load combinations. However, Lesny and Paikowsky (2011) and Paikowsky et al. (2010) showed that based on their experimental findings, a number of different soil properties (soil friction angles), controlled vs. natural soil conditions, and loading effects (vertical-centric, vertical-eccentric, inclined-centric, and inclined-eccentric) all affect the resistance factors for geotechnical resistances calibrated for strength limit states.

Additional special cases are introduced in Section 10.6.3.1.2 of AASHTO LRFD (2017) where details of the modifications that can be applied to Eq. 5.2 are provided, including those designed to cope with various special case scenarios such as punching shear (Section 10.6.3.1.2b), footings on slopes or near the top of a slope (Section 10.6.3.1.2c), two-layer soil systems (Section 10.6.3.1.2d), and two-layer soil systems that are subjected to undrained and drained loading conditions (Sections 10.6.3.1.2e and 10.6.3.1.2f) (AASHTO 2017).

Spread footing designs for footings on rock are usually controlled by either their overall stability or load eccentricity considerations. Rock competency is typically judged by taking into consideration the nature of the intact rock and the orientation and discontinuities of the overall rock mass, and should be verified using standard rock mass rating (RMR) procedures (AASHTO 2017). In AASHTO LRFD (2017) Section 10.6.2.4.4 competent rock is defined as material meeting the RMR Geomechanics Classification system (Bieniawski 1976, 1989) designation of “fair” or better, with an RMR score of 41 or higher. Rock competency should be evaluated by a geologist or engineering geologist as best practice rather than by an engineer. Although engineers can be trained to make this judgement. Comprehensive rock foundation design procedures have been developed using the RMR system; for more information consult Sabatini et al. (2002). The methods presented in Kulhawy and Goodman (1987), Goodman (1989), and Sowers and Sowers (1979) or any other standard text that covers the topic of rock mechanics and procedures can be consulted when estimating the bearing resistance for different rock failure modes. Load tests should also be performed to determine the rock’s nominal bearing resistance for foundations where appropriate (AASHTO 2017).

AASHTO defines marginal geologic materials not meeting the requirements for soil or rock as Intermediate Geomaterials (IGMs) regardless of minerology. In the Geomechanics Classification system IGMs are materials that rate less than “fair” rock with RMR score of 40 or less. In the State of South Dakota, much of the “soil” and “rock” encountered on typical projects actually classifies as IGM. Spread footing designs on IGMs are not explicitly detailed in AASHTO LRFD, nor does AASHTO provide presumptive bearing resistances for IGMs. AASHTO does provide design procedures and LRFD resistance factors for deep foundations in IGMs. See Appendix A for more details on RMR.

5.1.1.2 Semi-empirical Estimation

In-situ tests such as standard penetration test (SPT) and the cone penetration test (CPT), or the observed resistance of similar soils can be used to estimate the nominal bearing resistance of foundation soils. However, local experience should also be taken into account when selecting, conducting, and interpreting a test or a test result. Standard of practice in the United States is to correlate in-situ test data to engineering design parameters and use the methods in AASHTO LRFD (2017) Section 5.1.1.1 rather than utilize a semi-empirical method in shallow foundation design. However, as some engineers continue to use these semi-empirical methods, they are included here. Use of semi-empirical methods should be accompanied by appropriate resistance factors accounting for the increase in model error associated with these methods. Eq. 5.6 presents a method for estimating the nominal bearing resistance, in units of ksf, for sands based on SPT results, while Eq. 5.7 presents a method for estimating the nominal bearing resistance, in ksf, for cohesionless soils based on their CPT results.

$$q_n = \left(\frac{\bar{\mathrm{N}}1_{60}B}{5}\right)\left(\frac{C_{wq}D_f}{B+C_{w\gamma}}\right) \tag{5.6}$$

$$q_n = \left(\frac{\bar{\mathrm{q}}_c B}{40}\right)\left(\frac{C_{wq}D_f}{B+C_{w\gamma}}\right) \tag{5.7}$$

where:

B = width of footing (ft)

$\bar{\mathrm{N}}1_{60}$ = average SPT blow count corrected for rod length, sampler size, sampler liners, boring diameter, overburden and normalized to a hammer efficiency of 60%. The blow count is averaged over a depth range from the bottom of the footing to 1.5B below the bottom of the footing. Average should be the harmonic mean of blow counts in individual layers.

$\bar{\mathrm{q}}_c$ = average corrected cone tip resistance within a depth range B below the bottom of the footing (ksf) corrected for pore pressure transducer location (also known by some engineers as q_t). Average should be the harmonic mean of corrected tip resistance in the individual layers.

C_{wq}, $C_{w\gamma}$ = correction factors that account for groundwater table location as specified in Table 10.6.3.1.2a-2 in AASHTO LRFD (2017)

D_f = embedment depth (ft)

The nominal bearing resistance of rock should be determined using empirical correlation with the RMR system; local experience should also be taken into consideration (AASHTO 2017). Carter and Kulhawy (1988) developed a semi-empirical procedure with which to estimate the nominal bearing resistance of jointed or broken rock.

5.1.1.3 Plate Load Testing

The most accurate and reliable design approach to estimating nominal bearing resistance for spread footings at the strength limit state is through the use of small scale plate load tests or full scale load tests, which address design concerns and uncertainties with spread footings and confirm adequate performance (Abu-Hejleh et al. 2014). It is recommended that for large highway bridge projects, state DOTs should compile any existing load tests during the preliminary design phase and also review the documented results of load tests on other spread footings (Abu-Hejleh et al. 2014). Additional information is available in the reports by Paikowsky et al. (2010) and Samtani et al. (2010). A series of plate load tests can give strength and settlement characteristic of a soil under design loads but must be scaled correctly to the size of the full foundation as a small plate load test does not influence deeper soils that will be influenced by the full foundation. Full scale load tests do not need to be scaled as do plate load tests. The additional benefit of load testing is that the footing settlement (serviceability) is accurately determined, which is far more likely to control foundation design on soft soils than strength capacity.

5.1.1.4 Special Considerations

In general, theoretical, and semi-empirical equations are developed with assumptions and simplifications for the final equations, and thus cannot take into account effects of other common subsurface issues. This section introduces some of those that are included in the AASHTO LRFD Bridge Specifications (AASHTO 2017).

Based on the drainage characteristics of the ground/soil conditions, appropriate stress analysis methods for long term and/or short-term conditions should be performed using carefully selected test methods. For clayey soils, short-term total stress undrained conditions may govern over long-term effective stress drained conditions. Therefore, for clayey soils, best practice is to evaluate both the short-term undrained total stress and the long-term drained effective stress nominal bearing resistance. The lesser of the nominal bearing resistance of the footing calculated for the two conditions controls the design. The nominal bearing resistance of spread footings on granular soils should be evaluated using effective stress analysis methods and drained soil shear strength parameters regardless the analysis period (i.e. short-term or long-term).

The presence and location of a groundwater table can also affect the bearing resistance of soils because of the effect that water has on the shear strength and unit weight properties of soil. Effective unit weights of submerged soils are lower than the effective unit weights of the same soils above the water table. Submerged granular soils will generally experience a reduction in effective shear strength when compared to dry conditions; submerged cohesive soils will also generally experience a reduction in drained shear strength. This means that the submergence of soils, granular or cohesive, can lead to reduced bearing resistance in a foundation. To remain conservative, groundwater tables should be assumed to be the same as the highest expected groundwater table during the service life of a given structure for bearing resistance analyses (AASHTO 2017).

For cases where shallow foundations are on or near slopes, a global stability check should be performed. The bearing capacity equations in AASHTO LRFD are well developed for evaluating the capacity of a footing on or near a slope but are not intended to represent the global stability of the slope. The same is true for foundations near the top or bottom of a retaining wall. Both wall and foundations should be designed as an integrated system. AASHTO LRFD Chapter 11 contains more information on design of wall systems with the presence of a foundation.

There are also special cases of subsurface soils that should be given special attention by the design geotechnical engineer. If the soils within the groundwater active zone are susceptible to wetting induced collapse, wetting induced expansion, frost heave, or liquefaction, appropriate measures should be taken by the engineer to evaluate the hazard these soils may pose to the foundation in both short and long terms, effects on the serviceability, and mitigation measures to remediate the susceptible soil if a hazard is indeed posed.

5.1.1.5 Presumptive Method

Even though the classical equations discussed earlier are the generally accepted methods for estimating the bearing capacity, alternative methods are often used in practice as these equations require detailed information on both the soil properties and the foundation dimensions. A-priori estimation of the bearing capacity sacrifices accuracy and reliability for convenience and speed. One of the methods commonly used when making quick estimations is to apply presumptive values for different soil types. Tables 5.1 and 5.2 show the presumptive values provided in *British Standard 8004* (British Standards Institution 1986) and the *NAVFAC*

Design Manual 7.02 (United States Department of the Navy Facility Command [NAVFAC] 1986). The *AASHTO LRFD Bridge Design Specifications* (2012) also present a presumptive bearing resistance for spread footing foundations at the service limit state that was modified based on the NAVFAC (1986).

However, presumed bearing resistances may vary significantly even for the same soil or rock conditions. In addition, these values do not take into account the effects of many critical factors in foundations, such as a higher ground water table, the embedded depth, the shape of the foundation, and the design loading conditions. Experienced engineers with an in-depth knowledge of the soil and rock conditions at the project site must still exert the utmost care; these presumptive values should be used primarily for preliminary design purposes. While it is unlikely that a state DOT would approve a design for a bridge foundation on soil performed using presumptive bearing capacities, it is conceivable that minor structures and retaining wall footings may be designed with this method. Extreme caution should be exercised when presumptive values are used, and more stringent resistance factors should be used.

Table 5.1 Presumed allowable bearing values under static loading (British Standards Institution 1986)

Category	Types of rocks and soils	Presumed allowable bearing value (tsf)	Remarks
Rocks	Strong igneous and gneissic rocks in sound condition	100	Assumes that the foundations are taken down to unweathered rock.
	Strong limestones and strong sandstones	40	
	Schists and slates	30	
	Strong shales, strong mudstones, and strong siltstones	20	
Non-cohesive soils	Dense gravel, or dense sand and gravel	>6	Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation.
	Medium dense gravel, or medium dense sand and gravel	<2 to 6	
	Loose gravel, or loose sand and gravel	<2	
	Compact sand	>3	
	Medium dense sand	1 to 3	
	Loose sand	<1 *depends on degree of looseness*	
Cohesive soils	Very stiff bolder clays and hard clays	3 to 6	Group 3 is susceptible to long-term consolidation settlement.
	Stiff clays	1.5 to 3	
	Firm clays	0.75 to 1.5	
	Soft clays and silts	< 0.75	
	Very soft clays and silts	Not applicable	
Peat and organic soils		Not applicable	
Made ground or fill		Not applicable	

Table 5.2 Presumptive Values of Allowable Bearing Pressures NAVFAC 1986)

Types of bearing material	Consistency in place	Allowable bearing pressure (tsf)	
		Range	Recommended value for use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Hard, sound rock	60-100	80
Foliated metamorphic rock slate, schist (sound condition allows minor cracks)	Medium hard sound rock	30-40	35
Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities	Medium hard sound rock	15-25	20
Weathered or broken bed rock of any kind except highly argillaceous rock (shale). RQD less than 25	Soft rock	8-12	10
Compaction shale or other highly argillaceous rock in sound condition	Soft rock	8-12	10
Well graded mixture of fine and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	8-12	10
Gravel, gravel-sand mixtures, boulder gravel mixtures (GW, GP, SW, SP)	Very compact Medium to compact Loose	6-10 4-7 2-6	7 5 3
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	4-6 2-4 1-3	4 3 1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	3-5 2-4 1-2	3 2.5 1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	3-5 1-3 0.5-1	4 2 0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard Medium to stiff Soft	2-4 1-3 0.5-1	3 1.5 0.5

5.1.1.6 The Observational Approach and Past Performance Methods

It has been well known in geotechnical and foundation engineering that conventional design methods, equations, and protocols are insufficient for a number of cases that a field or design engineer may encounter in the course of their career. In these cases, the assumptions that the conventional design methods, equations, and protocols are predicated do not apply. AASHTO LRFD (2017) and all previous iterations of highway bridge codes include the conventional cases, design methods, equations and protocols in the approved methods and resistance factors prescribed in the code. A selection of these special cases is listed here to illustrate examples of conditions or cases in which conventional method assumptions may or may not apply and engineers face considerable obstacles in their judgement and design work. Conditions that are common to the State of South Dakota and neighboring states are noted in bold.

- Sites with high variability on soil layering (i.e. Lithology).
- Sites with high variability in soil properties in the lithology (i.e. heterogeneous and anisotropic conditions).
- Sites with uncertain and unpredictable artesian groundwater conditions.
- Sites with considerable previous construction such that existing foundations or structural elements may be buried at the site.
- Sites where the geologic materials have unique chemistry or origins such as crushable and brittle volcanic soils, diatomaceous soils, **highly expansive shales**, and thick organic rich clay deposits.
- **Sites where the geologic materials are very difficult to sample, test, or extract.**
- **Sites where the geologic materials cannot be easily assigned to conventional design methods for rock, sand, or clay**.

Rather than introduce additional risk or error into the design for sites like these, it was recommended by many of the early pioneers of the geotechnical and foundation design profession to use an Observational Approach that utilized instrumentation and past performance to design the foundations and earthwork for a bridge project. Advocates of this approach include Terzaghi, Peck, Thornburn, Hansen and Casagrande (Terzaghi 1943).

The Observational Approach is not directly incorporated into AASHTO LRFD (2017), but AASHTO makes allowances for this method as long as appropriate LRFD resistance factors are incorporated into the design. The Observational Approach is integrated into AASHTO LRFD (2017) deep foundation design more rigorously than in shallow foundation design.

The Observational Approach utilizing Past Performance design method has two phases, the design phase, and the construction phase. Both phases are essential to the Observational Approach. In cases where phase 1 is performed without phase 2, the engineer cannot indicate that they have followed an Observational Approach.

In the design phase, the site is investigated with conventional or non-conventional explorations and laboratory testing to sufficient detail that the site variability, lithology, and ground conditions can be assessed either qualitatively or quantitatively. Plate load tests or full-scale load tests are performed as is reasonable or project budgets allow. Past performance of similar structures for similar ground conditions and variability are evaluated and design parameters or values (such as nominal bearing capacity) are selected. Calculations are performed using the known ground conditions using conventional methods to check that past performance is reasonable for the site. The design proceeds with appropriate precautions for the actual ground conditions and utilizing AASHTO LRFD (2017) resistance factors appropriate for the variability and materials.

The second phase, the construction phase, is where the observations are included. The design includes a variety of geotechnical instruments to be installed during construction so that the performance of the structure can be monitored during and after construction to evaluate and assess performance. If the monitoring engineer identifies changed conditions or unacceptable movement, construction is halted, and the design is re-assessed and possibly re-designed, with the contractor having to adapt to the new design quickly. Geotechnical instruments include vertical and horizontal inclinometers, pore pressure transducers, settlement plates, pressure cells, strain gages, and survey points. Monitoring continues for several years after end of construction to ensure good performance of the system long-term. The observed performance, measured data, and construction drawings are then coupled with field and laboratory geotechnical data into a case history that is included in a database of past performance that is used by the design engineer on future projects where similar geologic conditions are encountered.

When performing designs with this method, Terzaghi and others recommend the use of as many plate loads tests, full scale load tests, and geotechnical instruments as can be included in the project budget. Monitoring should be performed several times a week during construction and continue at regular intervals for several years after construction. Several states adopt this approach for difficult ground conditions. Utah has used the Observational Approach successfully on many projects dating back to the 1950s and continues to use the Observational Approach on difficult ground conditions to this day.

5.1.2 Overview of Load and Resistance Factor Design (LRFD)

Foundations are designed to prevent two types of limit states: ultimate limit states (ULS) and serviceability limit states (SLS). Ultimate limit states are associated with dangerous conditions and can involve serious negative outcomes up to and including structural collapse. ULS are conditions of large and occasionally uncontrolled deflections that may or may not result in complete collapse of the structure. Although SLS are less severe, they are associated with impaired functionality and can involve adverse outcomes such as excessive settlement in the foundation (Foye et al. 2006a). Many DOTs, FHWA, and AASHTO have defined the SLS to be 1.0 to 2.0 inches for many projects in recent years. The SLS is much more difficult to define globally than the ULS, as the functionality limitations from excessive settlement vary by structure. More rigid structures tend to have more stringent SLS, while more ductile structures tend to have less stringent SLS. For example, a pre-cast concrete bridge with short span can tolerate less differential movement before cracking than a long steel bridge. Neither example bridge can tolerate the large uncontrolled deflections that accompany an ULS.

Figure 5.1 shows an example of the typical behavior of a shallow foundation when loaded to failure (i.e. the ULS). This behavior is typical of both small-scale plate load tests and instrumented full-scale load tests commonly found in the geotechnical literature. Note that the locations of the ULS and SLS are typical in this figure and may not represent the behavior of any particular foundation. In some cases, SLS may exceed the ULS. Figure 1 demonstrates that the ULS often occurs at much larger applied stress and resultant settlement than the SLS. The SLS in this example has been defined as 1.0-inches, and the SLS is shown lower in applied stress and deflection than the ULS. The ULS can be difficult to define on an actual load-deflection chart and is typically left to the judgement of the engineer to define. The bearing capacity equations were developed to represent the stress at which the maximum strength of the soil has been mobilized (i.e. the ULS) but have no regard to the deflections at which the ULS occurs. Figure 5.1 also shows the stress and resulting deflections of the factored (allowable) bearing capacity after application of LRFD resistance factors or ASD factor of safety. The application of resistance factors results in a lower stress than the ULS and construction to the factored bearing capacity results in a settlement much lower than the ULS. In some cases, the SLS may exceed the settlement from the factored bearing capacity, and in other cases the factored bearing capacity induced settlements will exceed the SLS. Thus, AASHTO requires that both the ULS and SLS be checked for design.

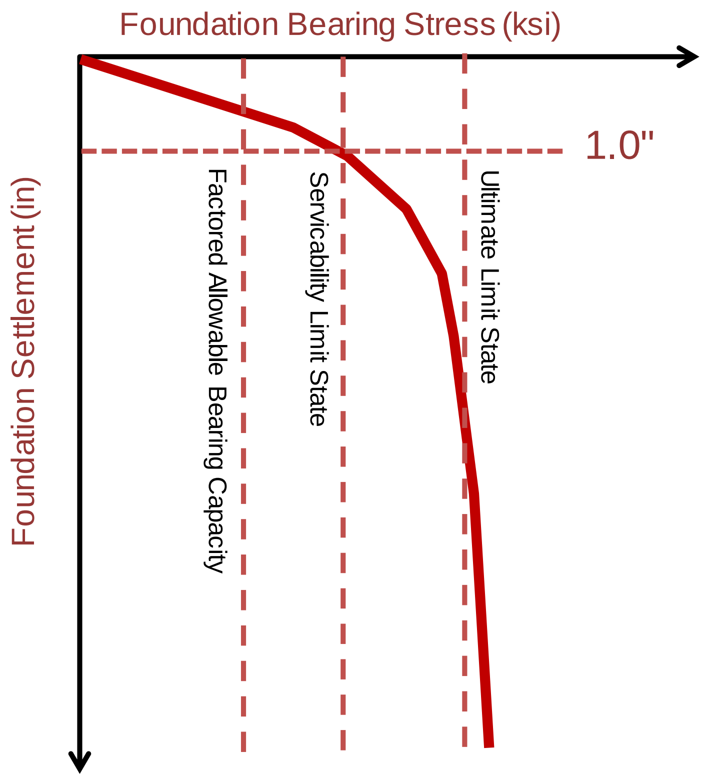

Figure 5.1 Typical load-deflection behavior of a shallow foundation showing example Ultimate Limit and Serviceability Limit States.

Allowable stress design (ASD), also known as working stress design (WSD), has been used in civil engineering since the early 1800s. Using allowable stress design, a structure (or structure element) is subjected to loads that are estimated and those loads are then compared to the structure's nominal resistance or ultimate capacity and a safety factor applied (Paikowsky et al. 2010). This is illustrated in Eq. 5.8:

$$\boldsymbol{Q \leq Q_{all} = \frac{Q_{ult}}{FS} = \frac{R_N}{FS}} \qquad \textbf{(5.8)}$$

where:

Q: Design load

Q_{all}: Allowable design load

R_N: Nominal structure (or structure element) resistance

Q_{ult}: Ultimate foundation resistance (capacity)

FS: Factor of safety

Over time, engineering experience has resulted in adequate factors of safety. However, in most cases their sources, reliability, and performance remain largely unknown because factors of safety may not take into account the bias of the analysis methods and variables used to calculate the factors of safety. Thus, the assumed impact on economics of design that factors of safety actually have are highly questionable (Paikowsky et al. 2010). As a result of the demand for more economical design practices and greater structural safety in recent decades, many engineering design processes have been reviewed and reevaluated. One solution to this dilemma is a design methodology known as load and resistance factor design (LRFD), which is a form of reliability-based design. LRFD is defined by AASHTO as a reliability-based design methodology in which the factored force effects of a structure cannot be greater than the factored resistance effects of the components of the structure (AASHTO 2012).

Although geotechnical engineers have been designing foundations with ultimate limit states and serviceability limit states for nearly a century, they have only recently begun migrating toward reliability-based design (RBD) (Fenton et al. 2008). Unlike allowable stress design, reliability-based design allows for a direct assessment of the risk associated with a design by attempting to keep the probability of reaching limit states lower than some appointed limiting value. However, the use of RBD is often not straightforward and can be very cumbersome for designers working on smaller projects. One alternative is LRFD, which although similar to RBD is much simpler to apply while still enjoying most of its benefits (Foye et al. 2006a). LRFD also has advantages over allowable stress design because it provides a more consistent level of reliability and makes it possible to separate load uncertainties from resistance uncertainties. For foundation design, LRFD proportions load effects and resistances by ensuring that the sum of factored loads is not larger than the factored resistances. This is done by multiplying the design load effects by the load factors, which typically has the effect of making them greater, and multiplying design resistances by resistance factors to make them smaller as shown in Eq. 5.9 (Abu-Hejleh et al. 2011; Foye et al. 2006a; Foye et al. 2006b; Lesny and Paikowsky 2011; Paikowsky et al. 2010):

$$\Phi R_n \geq \sum_{i=1}^{n} \eta_i \lambda_i Q_i \tag{5.9}$$

where:

Φ = resistance factor that accounts for uncertainty in nominal geotechnical resistance

R_n = nominal geotechnical resistance available to resist given load effects

λ_i = load factor for a given load and load combination that accounts for uncertainty in nominal design loads

Q_i = nominal load of a given type (e.g. dead, live, etc.) that acts on the foundation

η_i =Load modifier factor that relates to ductility, redundancy, and operational importance

An example of a probability density function (PDF) for load effect Q and resistance R is shown in Fig. 5.2.

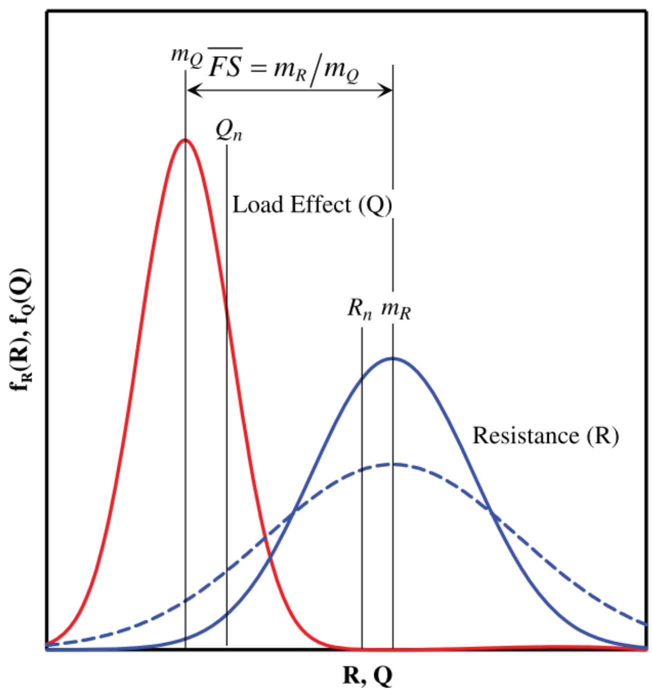

Figure 5.2 Probability Density Function Example for Load Effect (Q) and Resistance (R) (From Paikowsky et al. 2010)

where:

Q = calculated load effect that acts on an element, such as a foundation

R = estimated bearing capacity of loaded element

Q_n = nominal load effect

R_n = nominal resistance

m_Q = mean load effect

m_R = mean resistance

$\overline{FS}$ = central factor of safety $\left(m_R/m_Q\right)$

Calibrated design methods and loading conditions, respectively, are used to predict and evaluate nominal values (Q_n and R_n) and may or may not be the same as the mean values ($m_Q$and m_R), which are mean possible predictions that take into account the uncertainties that are associated with the calibrated design methods and loading conditions (Paikowsky et al. 2010).

Loads on a structural foundation are usually better known than foundation resistances in geotechnical engineering problems, so Q in Figure 5.2 will typically have a smaller coefficient of variation (smaller variability) than R and thus a narrower probability density function (Paikowsky et al. 2010). This is largely due to the inherently variable nature of soil, since soils can differ considerably from point to point (Fenton et al. 2005). If the resistance has an increase in uncertainty and thus a broader probability density function, as indicated by the dashed curve in Figure 5.2, the mean resistance remains the same but the coefficient of variation increases. This indicates that the resistance distribution with a higher coefficient of variation (broader distribution) and the resistance distribution with a lower coefficient of variation (narrower distribution) in Figure 5.2 will yield the same factor of safety in the ASD methodology, assuming the loading is the same for both cases. However, in LRFD, the probability density function for resistance with the higher coefficient of variation (broader distribution) will require the application of a smaller resistance factor in order to meet the same probability of failure assigned to both methods (Paikowsky et al. 2010).

A value for probability of failure is generally assigned based on considerations such as case histories and existing design practices. When load factors and resistance factors are applied to a design, the probability that the sum of the factored loads exceeds the factored resistance (actual probability of failure) should not exceed the appointed probability of failure (Paikowsky et al. 2010).

Equation 5.10 indicates the performance (g), or margin of safety, as the limit state function. Figures 5.3 and 5.4 show a derived probability density function for g as a function of load and resistance.

$$g = R - Q \tag{5.10}$$

Equation 5.11 shows the probability of failure (P_f) corresponds to the probability of the limit state (P) when g becomes less than 0. This is because when the load exceeds the resistance of a design element, it will be deemed unsafe and possibly fail (Paikowsky et al. 2010). This is illustrated in the probability density functions shown in Figures 5.3 and 5.4.

$$P_f = P\ (g < 0) \tag{5.11}$$

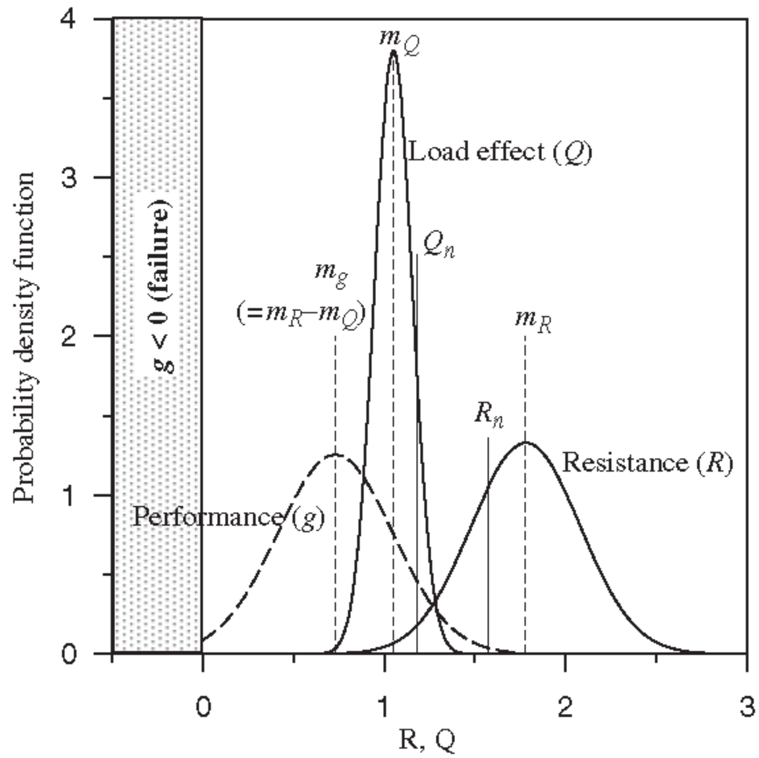

Figure 5.3 Probability density function for load, resistance, and performance (From Paikowsky et al. 2010)

For LRFD, instead of using a probability of failure to express the safety of a design element, a reliability index (β) is used (Lesny and Paikowsky 2011). The reliability index is directly related to the probability of failure for a design element (Foye et al. 2006b). It is defined as the number of standard deviations of the probability density function for a limit state g (σ_g) separating the mean of g from the nominal failure zone beginning at $g = 0$. Mathematically, this is shown in Equation 5.12. Thus, the reliability index is the margin of safety to be implemented in design work, as illustrated in Figure 5.3 (Foye et al. 2006b; Lesny and Paikowsky 2011).

$$\beta = {}^{m_g}/_{\sigma_g} = {}^{m_R - m_Q}\Big/_{\sqrt{\sigma_Q^2 + \sigma_R^2}} \tag{5.12}$$

where:

m_g = mean of performance or safety margin

σ_g = standard deviation of performance or safety margin

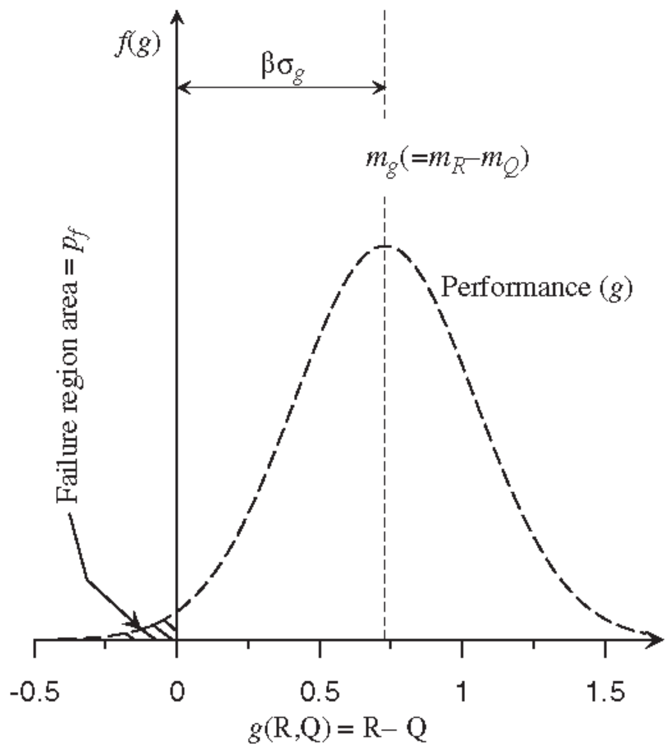

Figure 5.4 Probability density function for performance function $g(R, Q)$, and its relation to the reliability index (β) (From Paikowsky et al. 2010)

Table 5.3 shows the relationship between the probability of failure (P_f) and the reliability index (β) for the case where the reliability index follows a normal distribution. This relationship is described by Equation 5.13 (Lesny and Paikowsky 2011; Paikowsky et al. 2010).

$$P_f = \Phi(-\beta) \tag{5.13}$$

where:

Φ = error function which relates the reliability index to the probability of failure

The equation shows that as the probability of failure decreases, the reliability index increases. Hence, an element designed according to the LRFD methodology with a high reliability index will have a low probability of failure.

Table 5.3 Relationship between Probability of Failure (P_f) and Reliability Index (β) (From Paikowsky et al. 2010)

Reliability Index (β)	**Probability of Failure (P_f)**
1.0	0.159
1.2	0.115
1.4	0.0808
1.6	0.0548
1.8	0.0359
2.0	0.0228
2.2	0.0139
2.4	8.20E-3
2.6	4.66E-3
2.8	2.56E-3
3.0	1.35E-3
3.2	6.87E-4
3.4	3.87E-4
3.6	1.59E-4
3.8	7.23E-5
4.0	3.16E-5

The main objective of LRFD is to separate load uncertainties from resistance uncertainties and then apply probabilistic methods to create a safe design with a consistent level of reliability. In order to achieve this consistent reliability, (i.e. the specified reliability index), load (λ) and resistance (Φ) factors must be calibrated in such a way that the distributions of R and Q will both meet the requirements of this specified reliability index (Paikowsky et al. 2010). Figure 5.5 illustrates the determination and application of LRFD load and resistance factors for the PDF region where load is greater than resistance (Paikowsky et al. 2010).

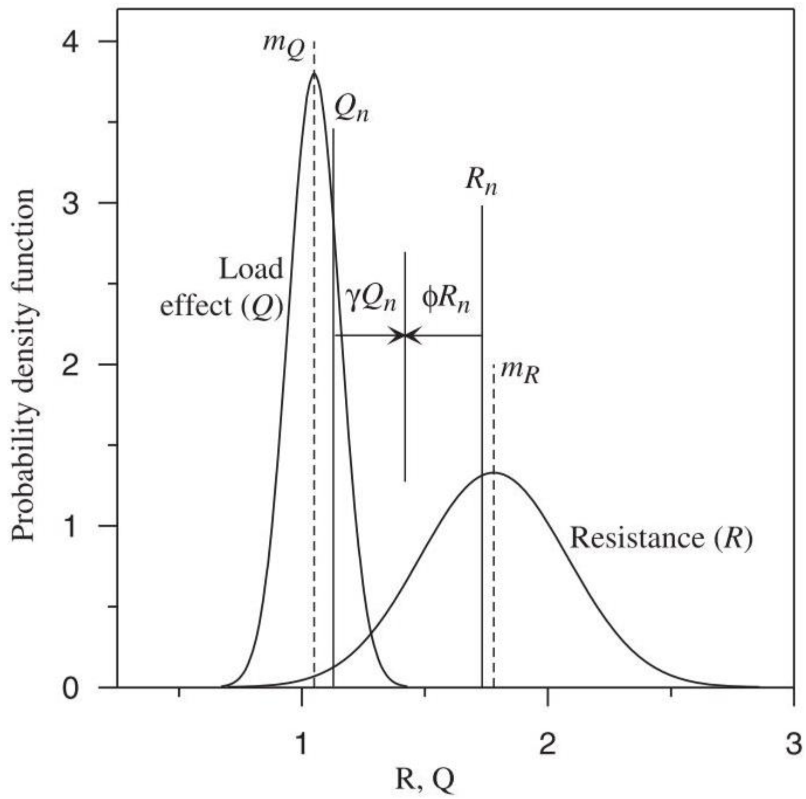

Figure 5.5 Application of LRFD load and resistance factors to meet target reliability (From Paikowsky et al. 2010)

Several LRFD calibration solutions are discussed in detail in Paikowsky et al. (2010), including AASHTO's preferred calibration procedure, Monte Carlo Simulation examples, which can be used to calculate the probability of failure without the need for close-formed solutions. A more detailed discussion of these calibration methods is beyond the scope of this report.

In conclusion, LRFD is a form of reliability-based design that, unlike allowable stress design (ASD), facilitates the direct assessment of risk associated with a design by attempting to keep the probability of reaching a limit state below a designated limiting value. This is done by separating load uncertainties and resistance uncertainties from one another and applying statistical methods to deliver a consistent level of reliability by enabling users to calibrate load and resistance factors to meet a target reliability index. The factor of safety, design loads, and allowable capacities in ASD are replaced in the LRFD with the reliability index (β) and load and resistance factors (λ and Φ), factored loads, and factored resistance, respectively. Note, however, that the resistance factors listed in AASHTO LRFD (2017) were developed based on calibration by fitting to ASD and reliability analyses and can be locally calibrated if long-term successful experience has been accumulated or special local issues not addressed in AASHTO are a concern.

5.1.2.1 Load Factors

The AASHTO service load design methodology does not recognize that some types of loads are more variable than others. However, AASHTO LRFD (2017) does consider the effects of both permanent and transient loads for bridge foundation designs. The ten load groups listed below provide a variety of possible loading conditions as well as imposing special loading requirements.

The Design Specifications take the following permanent load effects into account for load combination limit states and their corresponding load factors:

- Structural components and nonstructural attachments dead loads (DC)
- Down drag force effects (DD)
- Wearing surfaces and utility dead loads (DW)
- Horizontal earth pressure loads (EH)
- Vertical pressure caused by earth fill dead loads (EV)
- Earth surcharge loads (ES)
- Construction process force effects (EL)
- Secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states (PS)
- Creep force effects (CR)
- Shrinkage force effects (SH)

AASHTO LRFD (2017) bridge design also takes the following transient load effects into account for load combination limit states and their corresponding load factors:

- Vehicle live loads (LL)

- Vehicular dynamic load allowance (IM)
- Vehicle centrifugal force effects (CE)
- Vehicle braking force effects (BR)
- Pedestrian live loads (PL)
- Live load surcharge (LS)
- Water load and steam pressure force effects (WA)
- Wind load on structure (WS)
- Wind on live load (WL)
- Friction loads (FR)
- Uniform temperature force effects (TU)
- Temperature gradient force effects (TG)
- Settlement force effects (SE)
- Earthquake loads (EQ)
- Blast loading force effects (BL)
- Ice loads (IC)
- Vehicle collision force effects (CT)
- Vessel collision force effects (CV)

AASHTO LRFD (2017) states that the components and connections of a bridge must satisfy the following equation for applicable factored load combinations:

$$\boldsymbol{R_r = \Phi R_n \geq \sum \eta_i \gamma_i Q_i} \qquad \textbf{(5.14)}$$

where:

R_r= factored resistance
Φ = resistance factor: a statistically based multiplier applied to nominal resistances
η_i = load modifier
γ_i = load factor
Q_i = force effect

For loads where a maximum value of γ_i is appropriate:

$$\boldsymbol{\eta_i = \eta_D \eta_R \eta_I \geq 0.95} \qquad \textbf{(5.15)}$$

Where the load modifier (η_i) is a combination of the ductility, redundancy, and operational importance factors (η_D, η_R, and η_I , respectively).

AASHTO LRFD (2017) defines the following limit states where factored load combinations can be applied:

- Strength I – Basic load combination that relates to normal vehicular use of a bridge without wind.
- Strength II – Load combination that relates to the use of a bridge by owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
- Strength III – Load combination that relates to a bridge that experiences wind velocity exceeding 55 mph.
- Strength IV – Load combination that relates very high dead load to live load force effect ratios in bridge superstructures.
- Strength V – Load combination that relates normal vehicle use of a bridge with wind of 55 mph velocity.
- Extreme Event I – Load combination that includes earthquakes. The load factor for earthquake loads shall be determined on a project-specific basis.
- Extreme Event II – Load combination that relates to ice loads, vehicle and vessel collisions, check floods, and certain hydraulic events that have reduced live loads other than that which is part of the vehicle collision force effects. Check flood cases shall not be combined with blast loading force effects, vehicle collision force effects, vessel collision force effects, or ice loads.
- Service I – Load combination that relates to the normal operational use of a bridge that experiences wind velocities of 55 mph and all loads are taken at nominal (unfactored) values.
- Service II – Load combination that is meant to control yielding of steel structures and slip of slip-critical connections due to vehicle live loads.
- Service III – Load combination for longitudinal analysis that relates to tension in prestressed concrete superstructures with the goal of controlling cracks and to principal tension in the webs of segmental concrete girders.
- Service IV – Load combination that relates only to tension in in prestressed concrete columns with the goal of controlling cracks.
- Fatigue I – Fatigue and fracture load combination that relates to an infinite load-induced fatigue life.
- Fatigue II – Fatigue and fracture load combination that relates to a finite load-induced fatigue life.

For foundation design, the following load cases are generally considered: Strength I, Strength II, Strength III, Strength IV, Extreme 1, and Service I. Maximum and minimum load factors can be applied for any given foundation. In general, the maximum load factors will be used to find the maximum soil pressures, while the minimum load factors will be used to check stability of a foundation (Kimmerling 2002).

Tables 5.4 and 5.5 list the load factors for the various load types that can comprise typical design load combinations. Depending on the given loading conditions for the foundation, loads need to be selected properly for permanent and transient conditions and the associated possible limit states. Some load reactions could be either positive or negative, and thus, both extremes should be considered. Appropriate load factors (γ_P) should be selected to produce the total extreme factored force effect. For example, in load combinations where one force effect decreases another effect, the minimum load factor needs to be applied to the load reducing the force effect (AASHTO 2012). More details of how to apply the load factors, load combinations, and/or any modifications to the load factors and load combinations can be found in in Section 3 of AASHTO LRFD (2014).

In conclusion, AASHTO LRFD Bridge Design Specifications (2014) discusses several load effects (permanent and transient) that are taken into consideration during the LRFD bridge design process. It also discusses the limit states that load effects and the various applicable load combinations can be applied to. As the load factors are typically set by structural codes, the resistance factors can be determined if factor of safety data from previous ASD methods is available; alternatively, nominal bearing resistance can be determined using appropriate methods.

Table 5.4 Load Factors and Load Combinations (Table 3.4.1-1 in AASHTO 2014)

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	BL	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	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, and CE only	-	1.50	-	0	-		-	-	-	-	-	-	-	-
Fatigue II – LL, IM, and CE only	-	0.75	-	0	-		-	-	-	-	-	-	-	-

Table 5.5 Load Factors for Permanent Loads, $\boldsymbol{\gamma_P}$ (Table 3.4.1-2 in AASHTO 2014)

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor (Γ_P)	
		Maximum	Minimum
DC: Component and Attachments DC: Strength IV Only		1.25 1.50	0.90 0.90
DD: Down drag	Piles, α Tomlinson Method Piles, λ Method Drilled Shafts, O'Neill, and Reese (1999) Method	1.4 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure Active At-Rest AEP for Anchored Walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure Overall Stability Retaining Walls and Abutments Rigid Buried Structure Rigid Frames Flexible Buried Structures Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts Thermoplastic Culverts All Others		 1.00 1.35 1.30 1.35 - 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 - 0.9 0.9 0.9
ES: Earth Surcharge		1.50	0.75

5.1.2.2 Resistance Factors

The AASHTO service load design methodology does not recognize that some types of loads are more variable than others. However, AASHTO LRFD (2017) does consider the effects of both permanent and transient loads for bridge foundation designs. The ten load groups listed below provide a variety of possible loading conditions as well as imposing special loading requirements.

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- Earth surcharge loads (ES)
- Construction process force effects (EL)
- Secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states (PS)
- Creep force effects (CR)
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AASHTO LRFD (2017) bridge design also takes the following transient load effects into account for load combination limit states and their corresponding load factors:

- Vehicle live loads (LL)
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- Vehicle centrifugal force effects (CE)
- Vehicle braking force effects (BR)
- Pedestrian live loads (PL)
- Live load surcharge (LS)
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- Wind load on structure (WS)
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- Friction loads (FR)
- Uniform temperature force effects (TU)
- Temperature gradient force effects (TG)
- Settlement force effects (SE)
- Earthquake loads (EQ)

- Blast loading force effects (BL)
- Ice loads (IC)
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- Vessel collision force effects (CV)

AASHTO LRFD (2017) states that the components and connections of a bridge must satisfy the following equation for applicable factored load combinations:

$$R_r = \Phi R_n \geq \sum \eta_i \gamma_i Q_i \quad (5.16)$$

where:

R_r= factored resistance

Φ = resistance factor: a statistically based multiplier applied to nominal resistances

η_i = load modifier

γ_i = load factor

Q_i = force effect

For loads where a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \quad (5.17)$$

where the load modifier (η_i) is a combination of the ductility, redundancy, and operational importance factors (η_D, η_R, and η_I , respectively).

AASHTO LRFD (2017) defines the following limit states where factored load combinations can be applied:

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- Strength IV – Load combination that relates very high dead load to live load force effect ratios in bridge superstructures
- Strength V – Load combination that relates normal vehicle use of a bridge with wind of 55 mph velocity

- Extreme Event I – Load combination that includes earthquakes. The load factor for earthquake loads shall be determined on a project-specific basis.
- Extreme Event II – Load combination that relates to ice loads, vehicle and vessel collisions, check floods, and certain hydraulic events that have reduced live loads other than that which is part of the vehicle collision force effects. Check flood cases shall not be combined with blast loading force effects, vehicle collision force effects, vessel collision force effects, or ice loads.
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For foundation design, the following load cases are generally considered: Strength I, Strength II, Strength III, Strength IV, Extreme 1, and Service I. Maximum and minimum load factors can be applied for any given foundation. In general, the maximum load factors will be used to find the maximum soil pressures, while the minimum load factors will be used to check stability of a foundation (Kimmerling 2002).

Tables 5.6 and 5.7 list the load factors for the various load types that can comprise typical design load combinations. Depending on the given loading conditions for the foundation, loads need to be selected properly for permanent and transient conditions and the associated possible limit states. Some load reactions could be either positive or negative, and thus, both extremes should be considered. Appropriate load factors (γ_P) should be selected to produce the total extreme factored force effect. For example, in load combinations where one force effect decreases another effect, the minimum load factor needs to be applied to the load reducing the force effect (AASHTO 2012). More details of how to apply the load factors, load combinations, and/or any modifications to the load factors and load combinations can be found in in Section 3 of AASHTO LRFD (2014).

In conclusion, AASHTO LRFD Bridge Design Specifications (2014) discusses several load effects (permanent and transient) that are taken into consideration during the LRFD bridge design process. It also discusses the limit states and the various applicable load combinations that can be applied to the limit states. As the load factors are typically set by structural codes, the resistance factors can be determined if factor of safety data from previous ASD methods is available; alternatively, nominal bearing resistance can be determined using appropriate methods.

Table 5.6 Load Factors and Load Combinations (Table 3.4.1-1 in AASHTO 2014)

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	**BL**	**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	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, and CE only	-	1.50	-	0	-		-	-	-	-	-	-	-	-
Fatigue II – LL, IM, and CE only	-	0.75	-	0	-		-	-	-	-	-	-	-	-

Table 5.7 Load Factors for Permanent Loads, γ_P (Table 3.4.1-2 in AASHTO 2014)

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		**Load Factor (γ_P)**	
		Maximum	**Minimum**
DC: Component and Attachments DC: Strength IV Only		1.25 1.50	0.90 0.90
DD: Down drag	Piles, α Tomlinson Method Piles, λ Method Drilled Shafts, O'Neill, and Reese (1999) Method	1.4 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure Active At-Rest AEP for Anchored Walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure Overall Stability Retaining Walls and Abutments Rigid Buried Structure Rigid Frames Flexible Buried Structures Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts Thermoplastic Culverts All Others		 1.00 1.35 1.30 1.35 - 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 - 0.9 0.9 0.9
ES: Earth Surcharge		1.50	0.75

5.1.3 Serviceability Limits

Serviceability limit states, such as settlement or angular distortion, often govern the design of shallow foundations instead of ultimate limit states, such as bearing capacity (Fenton et al. 2005; Paikowsky et al. 2010). Major problems that can occur with excessive settlement or angular distortion at foundations include (Abu-Hejleh et al. 2014):

- Structural damage to superstructure components, which can lead to structural failure
- Superstructure function is detrimentally altered
- Drainage problems

- Potential damage to other structures connected to or associated with the main superstructure (drainage structures, utilities, etc.)

Properties of engineering materials such as steel and concrete are typically consistent because of their quality control in the manufacturing process having a small amount of variation. However, soil properties can vary significantly from point-to-point and even at a single site. These soil properties, along with soil profiles, help in determining the estimated total settlement that a foundation can experience. In general, total settlement of foundations consist of immediate settlement, consolidation settlement, and secondary settlement (creep). With the wide variance in soil properties at different sites, it is logical to utilize reliability-based design for settlement design. However, by that same logic, it may be cumbersome to apply statistical methods for every site, because each site will have different geotechnical parameters. Thus, it may be of use to utilize databases which index many different soils and their respective properties and utilize statistical methods to determine the appropriate resistance factors for those soils (Fenton et al. 2005).

Paikowsky et al. (2009) performed a study in which a large database of 329 case histories of shallow foundation load tests was used to develop and recommend resistance factors for service limit states in/on only granular soils using SPT data. Five settlement analysis methods were used to estimate the required loads to produce given settlements for the chosen soils, which ranged from 0.25 inches to 3.00 inches, which are within the established range for service limits of bridge foundations (Paikowsky and Lu 2006). A reliability analysis was done and resistance factors for each settlement analysis method were developed. During the reliability analysis, the mean bias was determined and expressed as the ratio of measured to calculated loads for a given settlement and the coefficient of variation (COV) of the mean bias was determined. These were then presented as functions of settlement. Specific information on the statistical trends are discussed in detail in the study. After the reliability analysis, resistance factors were calibrated for a target probability of failure of 10%. The results for the resistance factors for all analyzed settlement analysis methods for different settlement ranges are given in Table 5.8 and illustrates that different settlement analysis methods yield different resistance factors for SLS. Table 5.8 was developed using empirical relationships to SPT data (known to be highly variable), and not using theoretical approaches.

Table 5.8 Recommended Resistance Factors for Serviceability Limit State of Shallow Foundations in/on Granular Soils (Paikowsky et al. 2009)

Method	Range of Settlement S_e (inch)	Resistance Factor ϕ
Elastic Half-space Method as shown in AASHTO LRFD Section 10.6.2.4.2	$0.00 < S_e \leq 1.00$	0.85
	$1.00 < S_e \leq 1.50$	0.80
	$1.50 < S_e \leq 3.00$	0.60
Hough	$0.75 < S_e \leq 3.00$	$2.5e^{(-1.2Se)}$
Schmertmann (1978)	$0.00 < S_e \leq 3.00$	0.50
Schmertmann (1970)	$0.00 < S_e \leq 3.00$	0.30
D'Appolonia	$0.25 < S_e \leq 3.00$	$0.25\ S_e^{(-0.85)}$ where $\phi \leq 0.7$

The 2009 FHWA national survey (Abu-Hejleh et al. 2014) indicated that excessive settlement is the primary concern of state DOTs in using spread footings. Abu-Hejleh et al. (2014) offers a rational procedure and assumptions for settlement analysis of bridges supported on shallow foundations on soils. The study recognizes three different types of settlement for bridges:

- Bridge foundation settlement, S_F, which is generated from loads transferred to the foundation soil (e.g. During construction of bridge substructure or bridge superstructure).
- Bridge settlement at foundation locations, S_B, ($S_B \leq S_F$), which is the bridge foundation settlements that occurs during and after placement of a bridge superstructure. Bridge settlement at various foundation locations can lead to uniform settlement, differential settlement, angular distortion, and/or differential settlement between bridge and associated structures. These settlement types are illustrated further in Figure 5.6.
- Bridge settlement at foundations that impacts bridge performance, S_{BP}, ($S_{BP} \leq S_B$).

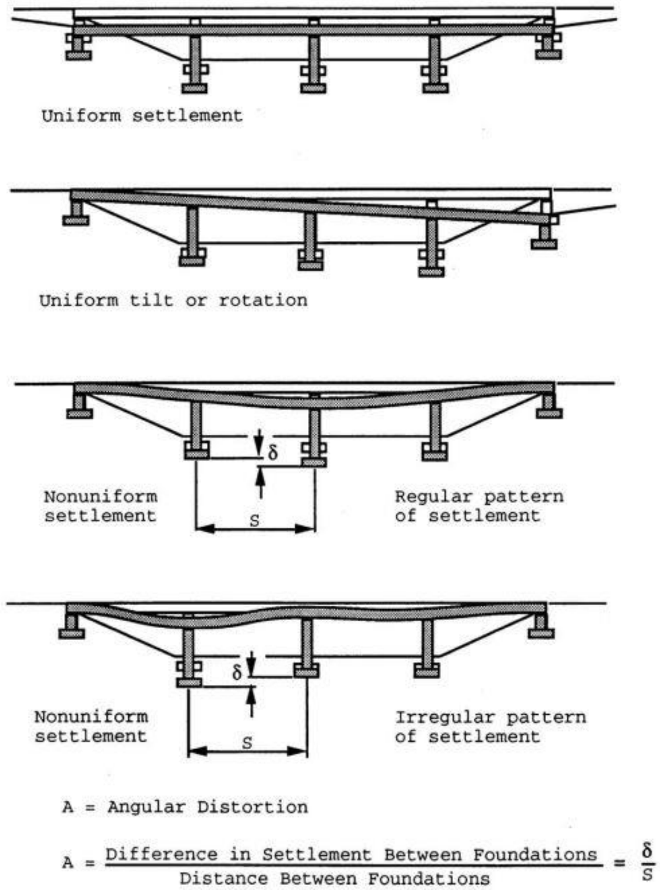

Figure 5.6 Types of Bridge Settlements (Abu-Hejleh et al. 2014)

It is imperative that structural and geotechnical engineers in a project work closely together to address any problems that a bridge can develop due to foundation settlement (Abu-Hejleh et al. 2014). Bridge differential settlements usually occur because of site variability, nonuniform stresses on foundations, and construction sequencing, and this can lead to additional distress to a bridge superstructure because of the added internal shear and moments of the bridge components. This can be alleviated by sizing foundations for uniform bridge settlements at all foundation locations (design phase) or by changing construction sequencing and utilizing procedures that may reduce bridge differential settlements and increase bridge tolerance to differential settlements (construction phase) (Abu-Hejleh et al. 2014).

The AASHTO LRFD Bridge Design Specifications assumes that load and resistance factors for settlement analysis at the service limit state are equal to unity due to reliability-based factors having not yet been developed (Abu-Hejleh et al. 2014). Abu-Hejleh et al. (2014) recommended following conservative assumptions for settlement analysis.

- Have the computed bridge differential settlement between two footings be equal to the larger of the computed bridge settlement at both footings. This is the same as assuming no settlement for the footing with lesser settlement. This assumption may be overly conservative for most design work.
- Have the bridge tolerable settlement equal the bridge tolerable differential settlement. Therefore, the bridge tolerable settlement at any foundation location may be estimated from the bridge tolerable differential settlement.

With these assumptions, the service limit state (SLS) for settlement of a bridge can be defined by Equation 5.18 as:

$$S_{BP} \leq S_{BT} \tag{5.18}$$

where:

S_{BP} = computed bridge settlement that impacts bridge performance

S_{BT} = bridge settlement that will not cause performance or function problems to the bridge and its associated structures during their design lives

5.1.3.1 Estimation of the Bridge Spread Footing Settlement

In general, total settlement is defined as the summation of immediate (elastic) settlement, consolidation settlement, and secondary compression/consolidation (creep). Immediate settlement is the instantaneous volume change or shear deformation of a soil mass when the soil is loaded and is the most predominant type of settlement for cohesionless soil masses, heavily over consolidated cohesive soils, or rock. Consolidation settlement is the volume change of a soil mass that occurs when a load is applied to a soil mass and the load expels air and water from the voids and is the predominant type of settlement for cohesive soil deposits. Applied loads are primarily carried by the pore water pressure in nearly saturated or saturated soils, but when consolidation settlement occurs, the applied loads are transferred from being primarily carried by the pore water pressure to the soil skeleton. Secondary compression/consolidation (creep) occurs often in organic or highly plastic soil deposits, but they are usually excluded from consideration for spread footings bearing on cohesionless soils.

AASHTO LRFD (2017) estimates settlements of spread footings on cohesionless soil deposits using elastic theory or empirical procedures. The settlements of spread footings on cohesionless soil deposits are estimated as a function of effective footing width and considers the effects of footing geometry and soil and rock layering with depth. The methods for estimating settlements on cohesionless soils provided in the AASHTO LRFD (2017) include the elastic half-space procedure and the empirical Hough method, which are considered to give generally conservative settlement estimates (Abu-Hejleh et al. 2014) . The accuracy of these methods and more general information regarding estimating settlements on sand can be found in Gifford et al. (1987) and Kimmerling (2002).

The elastic half-space method assumes a footing is flexible and supported on a homogeneous soil of infinite depth. The elastic-half space method for estimating immediate settlement, in feet, for spread footings is Equation 5.19 and is defined as:

$$S_e = \left[q_o(1-\upsilon^2)A'^{1/2}\right]144E_s\beta_z \tag{5.19}$$

where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil

β_z = shape factor as specified in Table 5.9 (Kulhawy et al. 1983)

υ= Poisson's Ratio

Table 5.9 Shape Factors for use in Elastic Half-Space Method (Kulhawy et al. 1983)

L/B	$\boldsymbol{\beta_z}$ **Flexible (average)**	$\boldsymbol{\beta_z}$ **Rigid**
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

The accuracy of estimated settlements using elastic theory depends on the selection of a soil modulus and the assumptions of infinite elastic half space. The soil modulus varies with depth as a function of overburden stress, but soil modulus analyses are based on a single value of a soil modulus. AASHTO LRFD (2017) suggests that if the soil modulus varies significantly with depth for a soil deposit, a weighted average for the soil modulus should be used, whereas the soil modulus should be determined at a depth of 1/3 to 2/3 of the footing width below the footing if the soil modulus does not vary significantly with depth.

The empirical Hough method was developed for normally consolidated cohesionless soils and is known to have advantages over other methods of estimating cohesionless soil settlement such as explicit considerations for soil layering and the zone of stress influencing below a finite size footing (AASHTO 2017).

The Hough method is defined using Equation 5.20:

$$S_e = \sum_{i=1}^{i=n} \Delta H_i \tag{5.20}$$

where

$$\Delta H_i = H_c (1/C') \log\left(\frac{\sigma_o' + \Delta\sigma_y}{\sigma_o'}\right) \tag{5.21}$$

where:

n = number of soil layers within zone of stress influence of the footings

ΔH_i = elastic settlement of layer i (ft)

H_c = initial height of layer i (ft)

C' = bearing capacity index from Figure 5.7

σ_o' = initial vertical effective stress at the midpoint of layer i (ksf)

$\Delta\sigma_y$ = increase in vertical stress at the midpoint of layer i (ksf)

For the Hough method, the subsurface soil profile should be divided into layers based on stratigraphy to a depth of approximately three times the footing width, and the maximum layer thickness should approximately be 10 feet (AASHTO 2017). Standard penetration test (SPT) blow counts shall be corrected for overburden pressure, so N1 shall be taken as $N1_{60}$ for use with Figure 5.7 to determine the bearing capacity index.

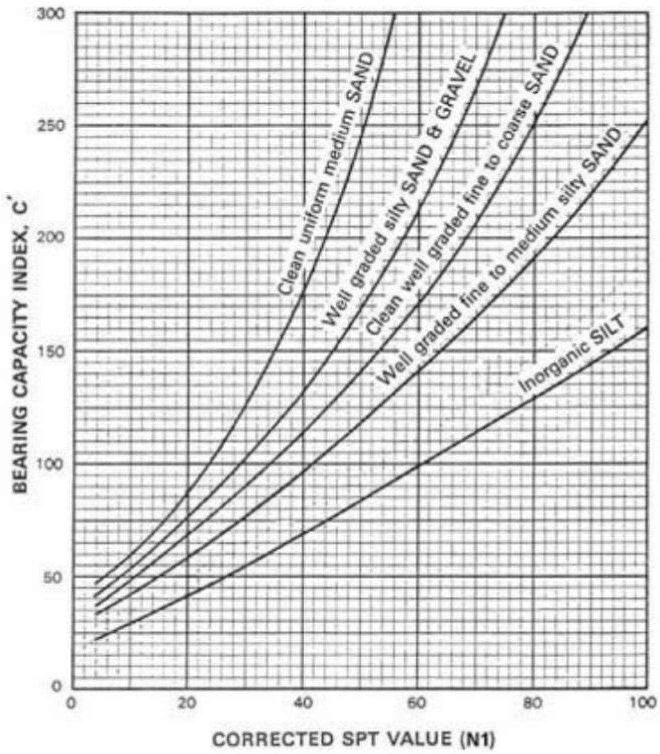

Figure 5.7 Bearing Capacity Index vs. Corrected SPT (AASHTO 2017)

Sargand et al. (2003) used measured settlement values of spread footings at various bridges in Ohio to evaluate various settlement estimation methods using SPT and CPT data. They found that the modified Schmertmann method (Schmertmann et al. 1978) is more reliable than other methods for normally consolidated sands and that using CPT data in settlement estimation methods provides more accurate results than SPT data (Abu-Hejleh et al. 2014). Gifford et al. (1987) found that the empirical Hough method is the least accurate and most conservative method of the various settlement estimation methods that they compared. Thus, engineers should use judgment, the latest in state of the art or state of the practice methods, and previous successful experiences in design to help determine which settlement estimation methods and which bridge tolerable settlements/bridge tolerable angular distortion will work best for their project's needs and resources.

For foundations on cohesive soils AASHTO (2017) requires that elastic, consolidation, and secondary settlements all be considered in design along with the timing and duration of the respective components of settlement. AASHTO (2017) makes no distinctions for heavily over consolidated clay soils such as glacial till or dry stiff clay compared to "classic" wet soft clay. For elastic settlement of cohesive soils, AASHTO (2017) recommends elastic theory or the tangent modulus method assuming undrained conditions and zero volume change (i.e. elastic settlement of cohesive soils is shear deformation only). For consolidation settlement, AASHTO (2017) recommends using non-linear consolidation settlement magnitude calculations with the pre-consolidation stress as the key parameter for analysis using either void ratio or strain. As spread footings and mat foundations are three-dimensional loading, AASHTO (2017) requires 3D reduction factors for all consolidation settlement calculations unless 1D conditions prevail from combined loading of embankment fill, foundation, and site fills. Time rate of consolidation is evaluated in AASHTO (2017) using Terzaghi's theory of the time rate of consolidation. Secondary compression magnitude is evaluated over the time period from 1) achieving 90% consolidation settlement to 2) the service life of the structure or other contractual period established by local agencies. AASHTO (2017) requires that settlement calculations include discrete consideration of cohesive soils on foundation settlement whenever encountered beneath a shallow foundation even if the preponderance of geologic materials are sand and/or rock.

For foundations on rock, AASHTO (2017) divides the settlement into two broad categories. The first category is for rock that rates as Fair to Very Good on the Geomechanics Classification system. In this category, settlements are assumed to be less than 0.5 inches. If the rock material rates less than Fair on the Geomechanics Classification system, then a settlement analysis is performed according to Section 10.6.2.4.4 using elastic continuum mechanics for weak, broken, or jointed rock. The specifications require the rock type, degree of weathering, joint characteristics, and condition of discontinuities to be considered in the analysis. Formations with unusual or poor rock mass conditions should have in-situ tests performed such as plate load tests or Pressuremeter testing. If time-dependent material such as clay infill or weathered claystone is present, the time-dependent characteristics and performance of the material should be estimated and applied to the design.

AASHTO LRFD (2017) does not explicitly address foundations on soils or rock that are susceptible to wetting induced collapse or wetting induced expansion. These considerations are often addressed on a local level by local highway agencies or State DOT design requirements. However, national standard of practice is for geotechnical investigations on highway projects where foundations are to be constructed to screen for soils susceptible to these wetting induced soil movements and to include design considerations explicitly. Expansive clays, claystone, shale, and loess are common moisture sensitive geologic materials that should be identified in the soils investigation and the geotechnical engineer should provide recommendations for remediating the materials if present at the project location.

5.1.3.2 Estimating Bridge Tolerable Settlement at Foundation Locations

For the final design, bridge tolerable settlement and angular distortion should be specified to mitigate or avoid the problems associated with excessive bridge settlement and/or excessive bridge angular distortion. These include (Abu-Hejleh et al. 2014):

- Structural damage to bridge structure components
- Bridge clearance
- Poor and unsafe ride quality on bridge
- Drainage problems
- Damage to structures associated with the bridge

In addition to specifying a bridge tolerable settlement and bridge tolerable angular distortion, bridge importance, design life, aesthetics, and past successful experiences should be taken into consideration during design.

For the preliminary design phase, designers should consider documented performance and published criteria. Table 5.10 adapted from Abu-Hejleh et al. (2014), compiled several references and the reported settlement measurements reported for bridges that performed well during service. Moulton et al. (1987) analyzed the footing movement data of 280 bridges. They determined the angular distortions for 56 simple span bridges and 119 continuous span bridges. Of the analyzed bridges, they found that longitudinal angular distortions of 0.005 radians for 36 simple span bridges and 0.004 radians for 79 continuous span bridges respectively are acceptable (Abu-Hejleh et al. 2014). Dimillio (1982) determined that bridges can experience 1-3 inches of differential settlement across spans without significant distress based on the measured movements of 28 bridges constructed with spread footings on compacted fill. Gifford et al. (1987) reported good performance of 21 bridges on cohesionless soils with experienced total and cross-span differential settlements up to 1 inch. Samtani et al. (2010) showed that 69 out of 78 measured bridges experienced settlements of 1 inch or less, that 8 out of 78 bridges experienced settlements between 1 inch and 2 inches, and that 1 bridge out of 78 experienced a settlement of 2.26 inches. All bridges were reported performing well during service. The results are summarized in Table 5.10.

Table 5.10 Documented Settlement Measurement of Bridges that Performed Well During Service

Reference	Reported Measurements	Conditions
Moulton et al. (1987)	Tolerable longitudinal angular distortions: • Simple span bridges: 0.005 • Continuous span bridges: 0.004	Based on analyzed footing movement of 280 bridges.
Dimillio (1982)	• Settlement: 1-3 inches	Based on analyzed measured movement of 28 bridges constructed with spread footings on compacted fill.
Gifford et al. (1987)	• Settlement: ≤ 1 inch	Based on 21 bridges with spread footings on granular soil.
Samtani et al. (2010)	• Settlement: ≤ 1 inch in 69 bridges • Settlement: 1-2 inches in 8 bridges • Settlement: 2.26 inches in 1 bridge	Based on 78 bridges.
Abu-Hejleh et al. (2014)	• Settlement: ≤1 inch	Minnesota, Colorado Bridges

Abu-Hejleh et al. (2014) summarized that:

- most states utilized a tolerable total and cross-span differential settlement of 1 inch for bridges successfully constructed on spread footings.
- Maine and Massachusetts utilized a tolerable total settlement of 2 inches for bridges successfully constructed on spread footings; and
- Utah utilized a tolerable total settlement of 1.5 inches for bridges successfully constructed on spread footings with maximum cross-span differential of 1.0 inches.

Using the criteria by Moulton et al. (1987), Kimmerling (2002) showed that a continuous span bridge having a span length of 150 ft can tolerate about 5 in of differential settlement with only minor loss of performance, and reported that bridges are generally designed for much smaller differential settlement and the following cross-span differential settlements between adjacent piers are not uncommon:

- Continuous span bridges: ≤ 1 inch
- Simple span bridges: ≤ 1.5-2.0 inches.

Figure 5.8 shows differential settlements as most commonly conceived by design engineers; differences in settlement between abutments and bents longitudinal to the bridge (i.e. along the length of the bridge). However, for large continuous spread footings or mat foundations for bridge abutments or bents, differential settlement may also occur transverse to the bridge. Figure 5.8 presents one such scenario where locally variable soil conditions beneath a single large foundation can cause more settlement under one portion of the foundation than another. Another common scenario is steeply dipping bedrock beneath the foundation with rapidly changing soil overburden. AASHTO LRFD (2017) does not directly provide any recommendations for geologic variability under a single foundation. Common practice in the United States is to address local geologic variability through 1) increased frequency and depth of geotechnical explorations, 2) use of ground improvement techniques to remediate the variability, or 3) use of deep foundations. Differential settlement limits for scenarios such as Figure 5.8 are not commonly covered in State DOT guidance documents. Therefore, engineers should use shallow foundations with caution when locally variable geologic conditions are suspected and/or documented through the site investigation.

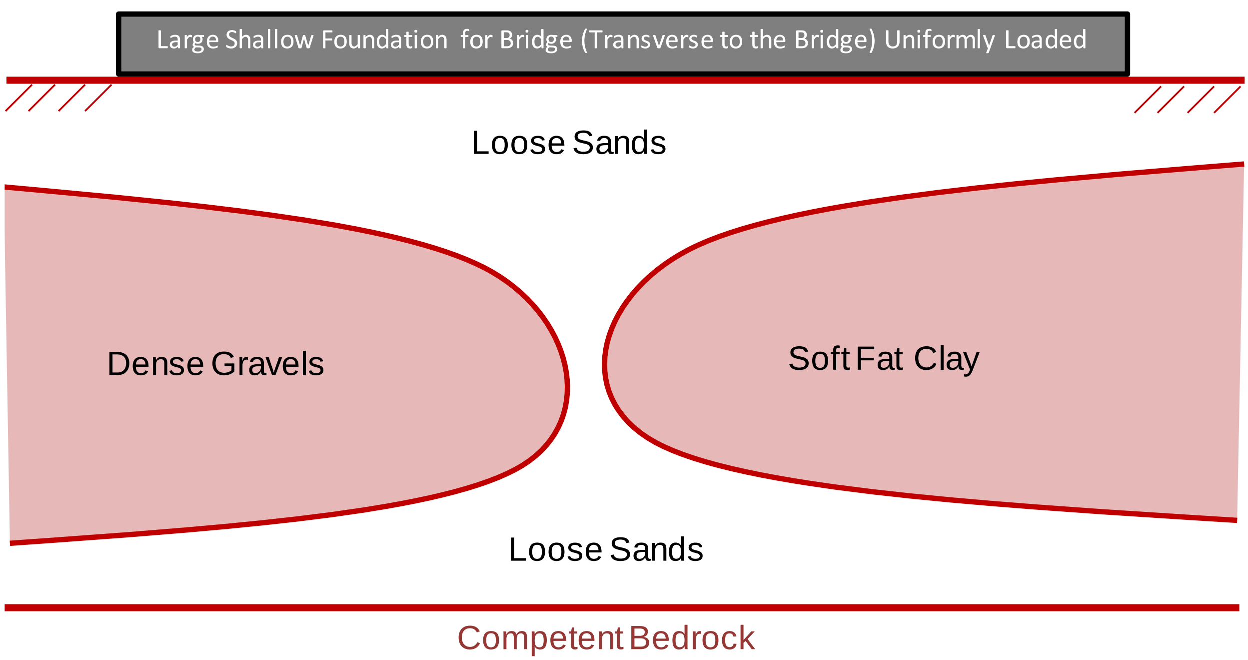

Figure 5.8 Example of scenario where foundation experiences differential settlements transverse to the bridge due to variable geology below ground surface.

5.2 Current LRFD Implementation for Shallow Foundations

The implementation of LRFD for shallow foundations has been limited compared to that for deep foundations for many reasons. One of which is the shortage of the available field data needed to execute competent statistical analyses and obtain reliable correlations. As a result, there is a lack of data on resistance factors for shallow foundations. Even with the mandatory use of LRFD for the FWHA's federally funded construction projects, the level of implementation has differed markedly from state to state. As one of the leading agencies responsible for developing specifications and standards, AASHTO has continued to update and improve the guidelines and recommendations in the *LRFD Bridge Design Specifications*. This chapter reviews the current status of implementation at the state level and discusses the reasons for its slow implementation for shallow foundations.

5.2.1 AASHTO

From the first *Standard Specifications for Highway Bridges and Incidental Structures* published by the American Association of State Highway Officials (AASHO) in 1931, the predecessor to today's AASHTO, the design methodologies and approaches were based on allowable stress design (ASD) also known as working stress design (WSD). AASHTO introduced its first reliability-based and probability-based *LRFD Bridge Design Specifications* in 1994 and has mandated the use of LRFD in bridge design since 2007 (AASHTO 2007). In LRFD, load and resistance factors are utilized to calculate the target level of safety and AASHTO provides resistance factors for geotechnical foundation design in its bridge design specifications. Table 5.9 shows the resistance factors suggested in the 6th Edition, published in 2012.

5.2.2 State DOTs

Several studies utilized surveys to analyze the use of shallow foundations and the implementation of LRFD in shallow foundation designs. These are described below and summarized in Table 5.11.

Chang (2006) conducted a study on the LRFD strategic implementation plan, sending a survey to all 50 state DOTs; 28 responded. The results indicated that seven out of the 28 states (ND, OK, SC, TX, LA, SD, and MS) did not use spread footings for their bridge foundations at all, and only 4 states (PA, WY, CT, and VT) used spread footings for 50% or more of their foundation designs. The Chang 2006 study is in error, as SD has used shallow foundations for bridges on rock. The remaining states reported that spread footings accounted for 5-40% of their foundations, as shown in Table 5.12.

Table 5.11 Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State (AASHTO 2012)

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al. 2001), in clay	0.50
		Theoretical method (Munfakh et al. 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al. 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil in soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

Paikowsky et al. (2010) also conducted a survey to collect information regarding foundation alternatives and shallow foundation design for bridge foundations, specifically with LRFD. Forty DOTs responded (39 US states and 1 Canadian province). Those responding were asked to consider the following questions for the period 2004 to 2006:

- Assess the percent usage of bridge foundation alternatives of the following types: shallow foundations, driven piles, and drilled foundations.
- Assess the percent usage of all constructed piers supported by shallow foundations. Out of the percentage supported on shallow foundations, assess the percentage supported on the following geomaterials: rock, intermediate geomaterials, granular soils, and cohesive soils.
- Assess the percent usage of all constructed abutments supported by shallow foundations. Out of the percentage supported on shallow foundations, assess the

percentage supported on the following geomaterials: rock, intermediate geomaterials (cemented soils/weathered rock), granular soils (sands and gravel), and cohesive soils (clays and silts).

Table 5.12 Foundation Usage Identified by Chang (2006)

States	Drilled Shaft (%)	Driven Pile (%)	Spread Footing (%)	States	Drilled Shaft (%)	Driven Pile (%)	Spread Footing (%)
Alabama	35	40	25	North Dakota	0	100	0
Arkansas	2	75	23	Ohio	15	80	5
Conn.	10	25	65	Oklahoma	95	5	0
Georgia	20	70	10	Oregon	5	75	20
Hawaii	60	20	20	Penn.	5	45	50
Indiana	5	80	15	Rhode Island	20	50	30
Kansas	45	45	10	South Carolina - Coastal	20	80	0
Louisiana	20	80	0	South Carolina – Piedmont	80	15	5
Maryland	10	50	40	South Dakota	20	80	0
Minnesota	5	85	10	Tennessee	10	50	40
Mississippi	5	95	0	Texas	70	30	0
Missouri	10	65	25	Utah[1]	N/A	N/A	N/A
New Jersey	20	40	40	Vermont	10	40	50
New York	10	40	40	Wyoming	15	30	55
Nevada	50	10	40				

1. Utah is known to use shallow foundations in only rare circumstances for bridge structures. Driven piles dominate compared to drilled shafts, although specific numbers are not available.

For further details of the survey questions and the responses obtained, please refer to Appendix C of *NCHRP Report 651* (Paikowsky et al. 2010).

On average, the respondents reported that the bridge foundation alternatives adopted in their state/province for the three years were as follows: 17% utilized shallow foundations, 59% utilized driven piles, and 24% utilized drilled shafts. However, the average usage of shallow foundations across the United States varied greatly from region to region. The survey results revealed that the use of shallow foundations in the Northeastern United States, which includes New York, New Jersey, Michigan, New Hampshire, Vermont, Massachusetts, Pennsylvania, and Connecticut, exceeded the use of shallow foundations in all other regions of the United States combined. Table 5.13 illustrates the percentage usage of shallow foundations to support bridges in the Northeastern states, as reported by Paikowsky et al. (2010).

Table 5.13 Use of Shallow Foundations for Bridges in the Northeastern States (Paikowsky et al., 2010)

State	Shallow Foundation Usage for Bridge Foundations
New York	40%
New Jersey	40%
Maine	40%
New Hampshire	47%
Vermont	50%
Massachusetts	53%
Pennsylvania	65%
Connecticut	67%

Abu-Hejleh et al. (2014) suggested that the reasons why the Northeastern states tended to utilize shallow foundations include the good long-term performance of existing bridges supported on shallow foundations, economic factors, the presence of competent natural soils or bedrock near the ground surface, and the implementation of good design and construction practices.

Other states that were using shallow foundations for bridges relatively often were Tennessee, Washington, Nevada, and Idaho, who used them 63%, 30%, 25%, and 20% of the time, respectively (Paikowsky et al. 2010). Six of the 39 states that responded to the survey did not use shallow foundations for bridges at all and a further 8 only used shallow foundations for bridges 5% of the time or less. The paper does not specify which states never used shallow foundations. Iowa and Arkansas indicated that they have used no spread footings on soils to support bridges.

The NCHRP survey results also indicated that 72.6% of the shallow foundations used were built on rock or IGMs; only 27.4% of shallow foundations were built on soil. Furthermore, 28 of the 39 states did not use shallow foundations for bridges on cohesive soils at all, as shown in Table 5.12. None of these state survey results include GRS-IBS system shallow foundations as the survey predates the use of the GRS-IBS shallow foundation system.

The FHWA published a report entitled *Implementation Guidance for Using Spread Footings on Soils to Support Highway Bridges* (Abu-Hejleh et al. 2014) with the goal of promoting the consideration and use of spread footings on soils when appropriate for highway bridges. The report analyzed the data gathered by two national surveys that were developed and conducted by the FHWA during 2007 and 2009, excluding GRS-IBS or similar specialty bridge foundation systems. Geotechnical engineers from 44 states responded to the 2007 survey, which focused specifically on state DOT geotechnical engineering practices, indicating that the average distribution of bridge foundation types across the United States based on information held by state DOTs are as shown in Table 5.14. These results are similar to the results reported in the *NCHRP Report 651* (Paikowsky et al. 2010).

Table 5.14 Average Distribution of Bridge Foundation Types Across United States (Abu-Hejleh et al. 2014)

TYPE OF FOUNDATION	PERCENTAGE USED	NOTES
Spread Footing	24%	11.5% on soils, 12.5% on rock
Deep Foundation	76%	56.5% are driven piles, 19.5% are drilled shafts

The 2007 FHWA national survey also identified the states with significant, moderate, limited (less than 5%), or no use of spread footings on soils to support highway bridges. Louisiana, Texas, and Kansas were not using spread footings on either soil or rock to support highway bridges. Table 5.15 lists the states leading the way in deploying spread footings for bridges in various regions of the United States. It shows that up to 50% of the bridges in the Northeast were using spread footings, in the Southwest it was up to 30%, the Northwest up to 20%, the Midwest up to 10%, and in the Southeast, it was none or limited. It should be noted that every region contained at least some states with limited or no use of spread footings on soil.

Table 5.15 Lead States in Deploying Spread Footings for Bridges (2007 FHWA National Survey Results) (Abu-Hejleh et al. 2014)

REGION	STATES	SPREAD FOOTINGS (%)		DEEP FOUNDATIONS (%)	
		Soil	**Rock**	**Driven Piles**	**Drilled Shafts**
Northeast	Connecticut	50	25	20	5
	Vermont	40	10	45	5
	Massachusetts	35	15	20	27
	New Hampshire	30	30	30	10
	New York	30	15	47	3
	New Jersey	30	20	40	5
Northwest	Idaho	20	10	60	10
	Oregon	20	10	60	10
Midwest	Michigan	10	5	80	5
Southwest	New Mexico	30	10	30	30
	Nevada	25	3	18	54

The 2009 FHWA national survey was conducted to gain additional insights from the state DOTs regarding their perceived obstacles when considering and implementing shallow foundations in their designs and on their experiences with the use of spread footings on soil to support highway bridges. The respondents identified the following perceived obstacles for those seeking to implement spread footing to support highway bridges (Abu-Hejleh et al. 2014):

- Scour of overburden soils needed for sliding resistance or other structural protection.
- Limited use and knowledge of FHWA and AASHTO technical resources and training courses for spread footing selection, LRFD design, construction, and performance.
- Limited knowledge of state DOTs that had successfully and economically used shallow foundations to support highway bridges with good performance.
- Concerns about using spread footings bearing on engineered granular fill and MSE fill.
- Limited use of load tests on spread footings to verify the design and performance of spread footings.
- Limited use of bridge instrumentation programs to verify the design and performance of spread footings.

- Inadequate subsurface investigation programs, which could lead to costly construction modifications if actual site conditions during construction differed from those assumed during the project design phase.
- Excessively conservative settlement analyses used for bridges supported on spread footings, leading to costlier designs for spread footings and hence making it more likely for spread footings to be excluded from consideration.
- LRFD implementation problems with spread footings.

As Tables 5.16-5.20 indicate, even though the preference for spread footings seems to vary by region, many states have utilized spread footings that are performing well and have no known performance issues. While states in the Northeastern United States tend to utilize spread footings the most, other regions have also implemented them with no known performance issues. Tables 5.14-5.18 compile the findings of the 2007 and 2009 FHWA national surveys and provide details of individual states' performances and experiences of shallow foundations for the various regions of the United States (Abu-Hejleh et al. 2014).

Table 5.16 Use and Performance of Bridges with Spread Footings on Soils: Northeast States.

State	FHWA 2007 National Survey Estimated Use (%) of Spread Footings on:		FHWA 2009 National Survey Use/Performance of Spread Footings on Soils
	Soil	**Rock**	
Connecticut	50	25	Used at every opportunity. Good performance
Vermont	40	10	Good performance
Massachusetts	35	15	Good performance
New Hampshire	30	30	To support abutments and piers. On MSE wall abutments. Used at every opportunity. Good performance.
New York	30	15	To support abutments and piers. On MSE wall abutments. Good performance.
New Jersey	30	20	Used at every opportunity. On MSE wall abutments. Good performance.
Delaware	13*	4*	Where feasible, more recent use. Good performance.
Pennsylvania	10	60	Good performance.
Rhode Island	10*	*	In glacial till. Good performance.
Maine	2	45	Used at every opportunity. Good performance.
Virginia	10	30	To support abutments and piers. On MSE wall abutments. Good performance.
Maryland	15*		Good performance.
West Virginia	0	20	No use.
District of Columbia	Rarely used*		

*From 2009 survey (data not reported in 2007 survey)

Table 5.17 Use and Performance of Bridges with Spread Footings on Soils: Northwest States (Abu-Hejleh et al. 2014).

State	FHWA 2007 National Survey Estimated Use (%) of Spread Footings on:		FHWA 2009 National Survey Use/Performance of Spread Footings on Soils
	Soil	**Rock**	
Idaho	20	10	Not aware of any performance issues with spread footings.
Oregon	20	10	For piers and abutments. Not aware of any performance issues with spread footings.
Washington	10	*	A very long history with successful use of spread footings on compacted granular embankments and with piers to support bridges. Dimillio (1982) reported very good conditions of 148 bridges. Currently not aware of any performance issues with spread footings.
Nebraska	10*	*	Not aware of any performance issues with spread footings.
Montana	5	5	For piers and abutments. Not aware of any performance issues with spread footings.
Wyoming	5	17	Not aware of any performance issues with spread footings.
Alaska	30% for abutments. <10% for piers*		Mostly to support abutments on MSE wall embankments. With piers on very dense glacial till. Not aware of any performance issues with spread footings.
Hawaii	7	2	For piers. Not aware of any performance issues with spread footings.
South Dakota	0	5	No use.
North Dakota	*	*	No use.

*From 2009 survey (data not reported in 2007 survey)

Table 5.18 Use and Performance of Bridges with Spread Footings on Soils: Midwest States (Abu-Hejleh et al. 2014).

STATE	FHWA 2007 NATIONAL SURVEY ESTIMATED USE (%) OF SPREAD FOOTINGS ON:		FHWA 2009 NATIONAL SURVEY USE/PERFORMANCE OF SPREAD FOOTINGS ON SOILS
	Soil	**Rock**	
Michigan	10	5	Hundreds of bridges with spread footings on soils constructed in Michigan (70% before 1980, reduced to 50% by 1990, and currently 10%).
Illinois	5	10	Use with piers on hard tills and dense sand; one bridge with MSE abutments.
Wisconsin	7.5*	10	Roughly 75 bridges supported on stiff natural soils constructed in the last 10 years. Very limited use with MSE walls. Use with multi-span bridges, piers, and abutments.
Indiana	1	5	Recently allowed spread footers for interior pier support on some grade separation bridges only in glacial tills, IGMs, and engineered fills. In process to allow them over MSE walls. Successfully used in the Accelerate I-465 project.
Minnesota	7	2	Recently used spread footing in simple span bridges (at abutments only) on dense sand and gravel, approximately four bridges per year, and with MSE walls. Use expected to increase in corridor and design-build projects. Employed with ground improvement in a value engineering project.
Ohio	5	1	Since 1998, built 244 bridges with spread footing on MSE walls and rocks. Presently, use of spread footings on MSE walls is not allowed. Current use is with dense sand and in few cases with very stiff clays.
Missouri	Little*	5	
Iowa	0	5	No use.

*From 2009 survey (data not reported in 2007 survey)

Table 5.19 Use and Performance of Bridges with Spread Footings on Soils: Southwest States (Abu-Hejleh et al. 2014).

STATE	FHWA 2007 NATIONAL SURVEY ESTIMATED USE (%) OF SPREAD FOOTINGS ON:		FHWA 2009 NATIONAL SURVEY USE/PERFORMANCE OF SPREAD FOOTINGS ON SOILS
	Soil	**Rock**	
New Mexico	30	10	Extensive use of spread footings on MSE walls (30 of 55 bridges in the I-25/I-40 interchange).
Nevada	25	3	Considered with all types of bridges. Saved money.
Arizona	20	5	Performance not reported but expected to be OK.
California	5 (30% - 50% in Southern California)*		Significant savings. Considered with all bridge types.
Utah	5	5	Mostly single-span bridges.
Colorado	3 bridges on soils*		Two bridges on MSE walls, third bridge on 2:1 approach embankment.
Oklahoma	Rarely used*		
Texas	0		No use
Kansas	0		No use

*From 2009 survey (data not reported in 2007 survey)

Table 5.20 Use of Bridges with Spread Footings on Soils: Southeast States (Abu-Hejleh et al. 2014).

STATE	FHWA 2007 NATIONAL SURVEY ESTIMATED USE (%) OF SPREAD FOOTINGS ON:	
	Soil	**Rock**
Tennessee	1	40
Florida	1	0
Alabama	5	10
North Carolina	0	10
Arkansas	1	22
Kentucky	<1*	40*
Georgia	2-3*	7-8*
Mississippi	5	0
South Carolina	Rarely used*	
Louisiana	0	0

*From 2009 survey (data not reported in 2007 survey)

Abu-Hejleh et al. (2014) also reported that according to the FHWA national survey results, the state DOTs that utilized spread footings on soils to support highway bridges did so under similar conditions to those recommended in the AASHTO LRFD Bridge Design Specifications and FHWA technical references, which consider the use of spread footings on:

- competent soils such as hard and very hard glacial tills, dense to very dense granular soils and gravel, cemented sand, and very stiff cohesive soils.
- improved natural soils; and
- compacted engineered granular fills and mechanically stabilized earth (MSE) fills

Some of the conditions typically considered to be favorable for utilizing shallow foundations for bridges were identified to be competent natural soils within shallow depths, smaller foundation widths, no groundwater issues, the availability of good quality of granular fill materials, lack of scour or liquefaction issues, and where shallow foundations can reduce cost and save time.

In addition to the above national surveys, the research team reviewed the current practices of other state DOTs involving shallow foundations and LRFD and determined how these states implement LRFD into their design methodologies in order to provide recommendations to SDDOT. To this end, many state bridge design manuals, geotechnical manuals, standard specifications, and reports published by other states were reviewed. The research group also reviewed publicly available online information such as workshop materials on LRFD for shallow foundations.

Based on this review, the states were divided into 3 groups based on their degree of LRFD implementation for shallow foundations. Group A consists of the states that indicate that they calibrate or modify the load and/or resistance factors found in the AASHTO LRFD (2017) when conducting or planning projects; Group B contains the states that indicate that they simply adopt AASHTO LRFD load and resistance factors for foundation design; and Group C comprises the states where no information on LRFD implementation was found by the research team. Only two states, Missouri, and Pennsylvania, have conducted or will conduct modification/calibration of the AASHTO LRFD parameters; most other states use the AASHTO LRFD parameters. A few states did not specifically indicate their use of LRFD for shallow foundations. Details of the information available on the various states' DOT websites as of June 2016 are presented in Table 5.21.

Table 5.21 State-level LRFD implementation status based on each state's DOT website information (as of June 2016).

Group	Description	States
A	Indicated the calibration or modification of the load and/or resistance factors found in the AASHTO LRFD Bridge Design Specifications	Missouri, Pennsylvania
B	Indicated load and resistance factors adopted from AASHTO LRFD Bridge Design Specifications	Alaska, Arizona, California, Colorado, Connecticut, Delaware, Florida*1 Georgia, Idaho, Illinois, Indiana*1, Iowa*1, Louisiana, Massachusetts, Michigan, Minnesota, Mississippi, Montana, Nebraska, Nevada, New Hampshire, New Jersey, New Mexico, New York, North Carolina, North Dakota, Oregon, South Carolina, South Dakota, Texas, Utah, Vermont, Washington, West Virginia
C	No information available/found	Alabama, Arkansas, Hawaii, Kansas, Kentucky, Maine, Maryland, Ohio*2, Oklahoma, Tennessee, Virginia

*1: Uses AASHTO LRFD Specification resistance factors for shallow foundations
*2: The official website of OH DOT was not accessible

Due to the limited information available online and the possibility of slow updating of the websites, the research team contacted some of the states directly to ask whether any updates not shown on their websites were available. Nineteen states were selected for further questions, and a short survey was distributed by email for this project to receive their input on their current LRFD implementation status specifically for shallow foundation design. The states surveyed were Alabama, Arizona, California, Colorado, Georgia, Hawaii, Iowa, Minnesota, Montana, Nebraska, North Carolina, North Dakota, Nevada, Oregon, South Carolina, Tennessee, Texas, West Virginia, and Wyoming.

The survey was conducted to gain useful insights into the following questions: 1) Does the surveyed DOT use AASHTO resistance factors or locally/regionally calibrated resistance factors? 2) If the surveyed DOT has calibrated its own resistance factors, were those resistance factors derived from experimental data or empirical experience? and 3) Does the surveyed DOT allow shallow foundations on soil or rock?

Eight out of the 19 states responded to the questionnaire: Alabama, Arizona, Georgia, Iowa, Montana, Nebraska, North Carolina, and Oregon; Table 4.3 summarizes their responses. Every state that responded to the survey uses LRFD resistance factors from AASHTO (2017), as expected due to the regulations on FHWA funded projects. Although it was confirmed through email communications that some DOTs do calibrate their own resistance factors for deep foundation design, none of the surveyed state DOTs that responded did so for shallow foundations. The data presented in Table 5.22 also show that a majority of the state DOTs that replied prefer to use shallow foundations on rock rather than soil.

Table 5.22 State Responses to Survey on LRFD Implementation for Shallow Foundations

STATE	AASHTO RESISTANCE FACTORS OR LOCALLY/REGIONALLY CALIBRATED RESISTANCE FACTORS FOR SHALLOW FOUNDATIONS?	USE SHALLOW FOUNDATIONS ON SOIL OR ROCK?
Alabama	AASHTO (but only implemented LRFD for piles at this time)	Rock
Arizona	AASHTO	No Response
Georgia	Secondary source by Paikowsky et al. (2010): "Recommended Resistance Factors for Shallow Foundations on Natural Deposited Granular Soil Conditions"	Primarily on rock for bridge design; Use shallow foundations on soil for walls
Iowa	AASHTO	Primarily on rock, smaller structures like retaining walls or sign trusses could be placed on soil
Montana	AASHTO	Used for both
Nebraska	AASHTO	No Response
North Carolina	AASHTO	Primarily on rock
Oregon	AASHTO	Used for both
Wyoming	AASHTO	Primarily on rock

Paikowsky et al. (2010) and Abu-Hejleh et al. (2014) both indicated that spread foundations supporting highway bridges are used frequently in many locations around the United States and are known to perform well using the AASHTO LRFD Specifications. Thus, the SDDOT may be missing an opportunity to save on construction costs and time by not considering the usage of spread footings on soils for LRFD design (Abu-Hejleh et al. 2014).

Using deep foundations where shallow foundations could safely be constructed can lead to additional design and construction complexity and thus additional construction costs and time.

Pennsylvania is one of the few places where the state's Department of Transportation (PennDOT) has implemented refined resistance factors in their Design Manuals (Pennsylvania Department of Transportation 2015). PennDOT currently utilizes bearing resistance factors for shallow foundations that are based on experience, sample designs developed when AASHTO was initially implemented, and from calibrations with LRFD resistance factors. (Paikowsky et al. 2010). Table 5.21 compares the values from three different sources for the resistance factors: PennDOT Design Manuals Part 4 Structures (Pennsylvania Department of Transportation 2015); AASHTO (2002); and AASHTO (2012). The refinements were made to incorporate resistance factors and to update the descriptions and categories of the limit states. There are both reduced and increased values of resistance factors, which suggests that these were updated after sufficient data was collected. The differences are mainly due to the department's extensive experience with rocks, which PennDOT has worked with more than many other states.

Missouri is the other state that has been actively working on the LRFD implementation for shallow foundations. The Missouri Department of Transportation (MoDOT) has conducted several research projects to implement LRFD methods for geotechnical designs. They also provide the MoDOT Engineering Policy Guide, which gives the LRFD design guidelines for bridges, and other relevant manuals online. The MoDOT suggests the use of resistance factors in AASHTO LRFD Bridge Design Specification for cohesionless soils. However, it then goes on to provide refinements for cohesive soils and rock, along with figures showing the variations of resistance factors corresponding to the coefficient of variation (COV) of soil/rock properties. These figures are based on the concept that the resistance factors are contingent upon the variability and uncertainty in the design properties and recommend the adoption of different target reliabilities. Therefore, for each project the different availability of test data and the different importance in terms of cost and service level can be applied to adjust the resistance factor appropriately. An example is shown in Figure 5.9.

Table 5.23 Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State (AASHTO 2002; AASHTO 2012; PennDOT 2015)

Method/Soil/Condition			Resistance Factor		
			AASHTO (2002)	**AASHTO (2012)**	**PennDOT (2015)**
Bearing Resistance	φ_b	Theoretical method (Munfakh et al. 2001), in clay	0.60[*1] 0.50[*2]	0.50	0.50
		Theoretical method (Munfakh et al. 2001), in sand, using *CPT*	0.45	0.50	0.50
		Theoretical method (Munfakh et al. 2001), in sand, using *SPT*	0.35	0.45	0.45
		Semi-empirical methods - sand using SPT - sand using CPT - clay sand using CPT	0.45 0.55 0.50	0.45 (all soils)	0.45 0.45 0.45
		Footings on rock	0.60	0.45	0.50
		Plate Load Test	N/A	0.55	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90	0.90	0.90
		Cast-in-Place Concrete on sand	0.90	0.80	0.80
		Cast-in-Place or precast Concrete on Clay	N/A	0.85	0.85
		Clay - shear strength < $0.5 \cdot \sigma_n$ (Lab or field vane) - shear strength < $0.5 \cdot \sigma_n$ (CPT) - shear strength ≥ $0.5 \cdot \sigma_n$	0.85 0.80 0.85	N/A	N/A
		Soil in soil	N/A	0.90	1.0
		Precast concrete place on rock - Using estimated δ[*3] - Using measured δ	N/A	N/A	1.00 0.90
		Cast in place on rock - Using estimated δ[*3] - Using measured δ	N/A	N/A	1.00 0.80
	φ_{ep}	Passive earth pressure component of sliding resistance	N/A	0.50	0.50

[*1]: Rational method with lab tests or field vane tests.
[*2]: Rational method with CPT
[*3]: Table 3.11.5.3-1—Friction Angle for Dissimilar Materials in AASHTO (2012)

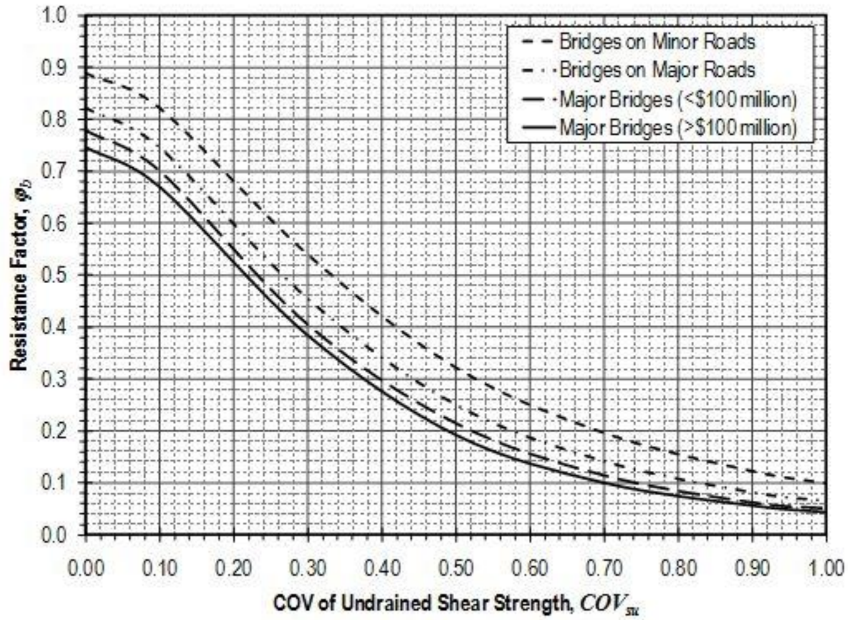

Figure 5.9 Resistance factors for bearing resistance for spread footings on cohesive soils (From MoDOT 2015)

5.2.3 Canada

Canada has two national design codes for structures and geotechnical applications: The National Building Code of Canada (NBCC) and the Canadian Highway Bridge Design Code (CHBDC) (Fenton et al. 2015). Both codes use reliability-based design for structural and geotechnical designs and the Canadian geotechnical community is moving further towards reliability-based design. However, the implementation of reliability-based design or limit state design (LSD) to geotechnical designs is slightly different to that adopted by AASHTO.

The approach used in the most recent version of the CHBDC expands on LRFD by applying a risk and consequences-based framework. This type of framework adjusts the target probability of failure depending on the type and potential severity of the consequences of a failure. This is achieved by adding a consequence factor to the LRFD design equation, which is included in the in the S6-14 2014 Canadian Highway Bridge Design Code (Fenton et al. 2011; Fenton et al. 2015). This gives a LRFD design equation of the form:

$$\Psi\varphi R \geq \sum \gamma L \tag{5.22}$$

Where all the symbols are as defined earlier, withΨ, being the consequence factor.

Three levels of consequence are considered for foundations or geotechnical system in the 2014 CHBDC (Fenton et al. 2015):

- High Consequence – designed for applications having large societal or economic impacts
- Typical Consequence – designed for applications medium to large volumes of traffic or having potential impacts on alternative transportation corridors or structures. This is the default level for structures.
- Low Consequence – designed for applications carrying low volumes of traffic and having limited impacts on alternative transportation corridors. This type of structure has a low chance of posing a threat to human safety such as storage facilities or temporary structures.

  Values for the consequence factor in each situation with a target maximum lifetime are given in Table 5.24.

Table 5.24 Consequence factors for different levels of consequence rating for structures (modified from Fenton et al. 2015).

Consequence Level	Target maximum lifetime (75 years) failure probability, p_m, for ULS (SLS)	Reliability index for ULS (SLS)	Consequence Factor Ψ
High	1/10,000 (1/1000)	3.7 (3.1)	0.9
Typical	1/5,000 (1/500)	3.5 (2.9)	1.0
Low	1/1,000 (1/100)	3.1 (2.3)	1.15

*ULS: Ultimate Limit State
*SLS: Service Limit State

In addition to consequence factors, sliding resistance factors are also being considered. Here, the resistance factor value is dependent on the degree of understanding of the site and model. As for the consequence factor, there are three levels of understanding for the site and model:

- High Understanding – This level of understanding is considered to apply when models are used that have been proven to achieve a high level of confidence with their predictions. A project where extensive site investigations that are project specific have been carried out would also fall into this category.

- Typical Understanding – Here, conventional prediction models are combined with a standard project-specific site investigation.
- Low Understanding – Conventional prediction models are combined with an understanding of the site based on previous experience or extrapolation from nearby or similar sites.

Table 5.25 illustrates how this would be combined with resistance factors for a project with shallow foundations. The values in the table should be viewed as illustrative only and the CHBDC must be consulted for the actual factors.

Table 5.25 Shallow foundation resistance factors depending on the degree of understanding of the site (modified from Fenton et al. 2015).

Shallow Foundation Resistance Factors	**Degree of Understanding**		
	Low	Typical	High
Bearing	0.45	0.50	0.60
Passive Resistance	0.40	0.50	0.55
Sliding Resistance -frictional	0.70	0.80	0.90
Sliding Resistance -cohesive	0.55	0.60	0.65
Settlement or lateral movement	0.70	0.80	0.9

To supplement the design codes applied in Canada, the Canadian Foundation Engineering Manual (CFEM) has been published. The CFEM is meant to help users interpret the intent and requirements of the national codes and provides additional material that is not covered in the national codes, including in depth information on the geotechnical aspects of foundation engineering.

5.2.4 European Union

The European Union (EU) was originally created as European Economic Community (EEC) after World War II to increase economic cooperation among six countries, and later, became a unique economic and political union changing its name to EU in 1993. Currently, there are 28-member countries who agree on unrestricted movement of goods, capital, services, and people. The EU has established integrated systems for immigration, finance, and currency, as well as standards in the construction industry. The Eurocode is the national standard for construction products and engineering services in the EU (European Union-European Commission 2016).

The Eurocode was first introduced in 1989 and currently consists of ten sections; Eurocode 7, which deals with Geotechnical Design, was adopted in 1997. The LRFD concepts that incorporate reliability and probability design were introduced in Eurocode 7, which differs from the other sections in that it focuses on the management of the design process, as opposed to the details of the design calculations. Although this focus has the drawback of not providing any calculation models in the core text, some information is provided in Informative Annexes. Most EU member countries are updating their national standards to complement Eurocode 7 (Orr 2012).

The major difference when applying LRFD methodology to foundation designs is how the factored capacity of the foundations is determined. As discussed earlier, the AASHTO specifications and Canadian code both use a factored resistance approach, also known as the total resistance factor approach, where capacity is calculated in the standard design manner and a resistance factor is applied for each limit state.

While the resistance factors are applied to yield the calculated bearing capacity in the AASHTO specifications (2012), Eurocode 7 instead uses the factored strength method, also known as the partial resistance factor approach, where the partial factors, which are equivalent to the resistance factors, are applied to geotechnical strength parameters such as cohesion and angle of internal friction, and then these factored strength parameters are used to determine the foundation capacity.

These two approaches are illustrated in Figure 5.10. Although the values of the factors are different, the partial factors can be compared by taking the inverse of the resistance factors. Note that due to the way resistance factors are typically calibrated based on conventional allowable stress design (ASD) designs, both methods achieve the same level of safety (Fenton et al. 2015).

Eurocode 7 recommends the quantities to be used for various partial factors, but these are only recommendations and countries in the European Union are free to define their own partial factors in their National Annex. Although countries have been given this option, almost all have chosen to use the recommended values from Eurocode 7.

The main differences between the designs in each country are largely due to the design approach used. Eurocode 7 outlines three design approaches (DA): DA 1, DA 2, and DA 3.

DA 1 applies partial factors to actions and ground strength parameters. DA 2 applies partial factors to actions and ground resistances, as in the total resistance factor approach used in North America (Fenton et al. 2015). There is a slight variation of this method, denoted as DA 2*, which applies partial factors to the effects of actions and ground resistances. DA 3 applies partial factors only to structural actions, but not to geotechnical actions or material properties. This can be represented as follows in Figure 5.11.

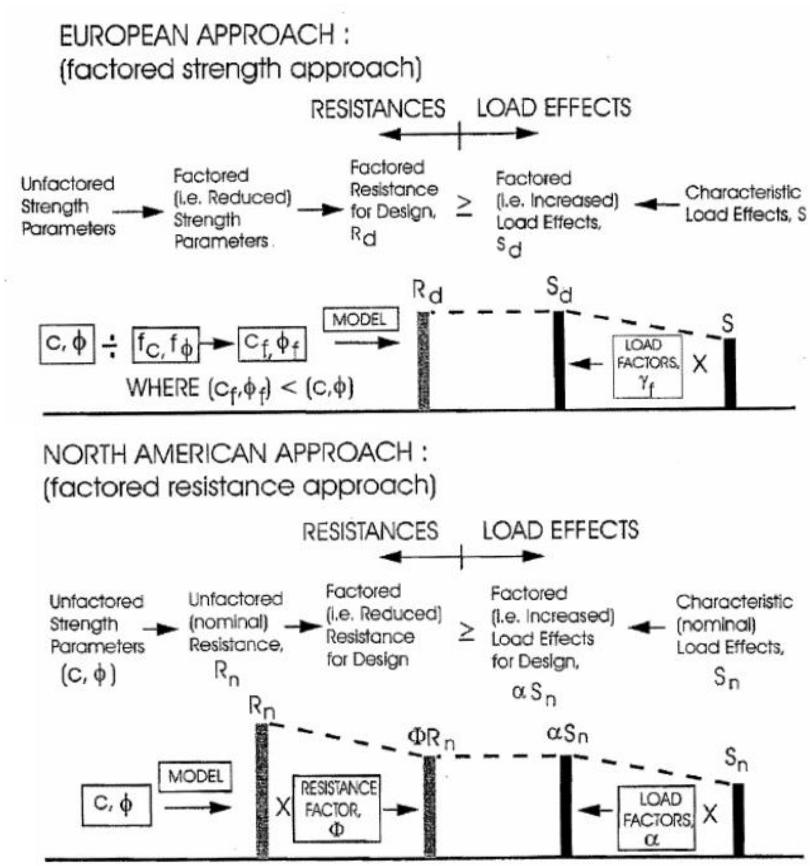

Figure 5.10 Graphic illustration of the two different LRFD design methods around the world. Images from the Canadian Foundation Engineering Manual 4th edition (Canadian Geotechnical Society 2006)

Here, A represents an action or the effects of an action, M represents soil parameters, and R represents the resistances. The "+" symbol signifies the use of a combination of these three sets of resistance factors in the design. Geotechnical actions are defined in Eurocode 7 as: the weight of the soil, stresses in the ground, seepage forces, surcharges, etc. Each design approach applies these resistance factors across two out of the three categories. This is represented mathematically by imposing a resistance factor of one. For example, DA1 does not apply resistance factors to resistances, so all the factors associated with "R1" are set to equal 1.0.

Most countries use different design approaches depending on the type of foundation (Orr 2012). Figure 5.11 shows the different design approaches each country uses for shallow foundations (Bond 2013). For shallow foundations, different Eurocode 7 design approaches can lead to quite different designs, making determination of the safety of the foundation unreliable, especially when complex loading is involved. It remains the case, however, that designs for shallow foundations that apply Eurocode 7 approaches are considered more reliable than those based on the allowable stress method (Forrest and Orr 2010).

As the figure shows, a wide range of different approaches are used across the EU when shallow foundations are involved. Though the approaches are different, they all share a common need for a solid understanding of the site and soil parameters. Unfortunately, thorough site investigations are not always feasible, so an alternative approach would be to develop a better understanding of precisely how resistance factors change with varying soil parameters. This would allow an easier alternative to a thorough site investigation.

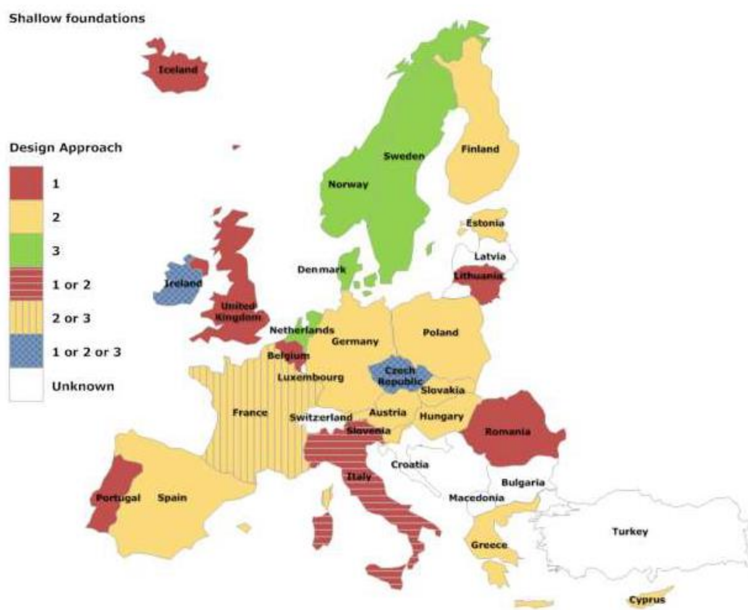

Figure 5.11: Map showing the chosen design approach for shallow foundation projects for each country in the European Union. (Bond 2013)

Table 5.26 summarizes the resistance factors relevant to shallow foundations provided in Eurocode 7. The values for each of the factors are listed under the relevant design approach (DA1 values are listed in the R1 column, etc.). Bold numbers in parentheses provide the inverted resistance factor for convenient comparison with the resistance factors used in North

American codes. Eurocode 7 resistance factors can be compared to North American resistance factors by inverting them (Fenton et al. 2015).

Table 5.26 Recommended resistance factors from Eurocode7

RESISTANCE	SYMBOL	SET (DESIGN APPROACH 1, 2, 3)		
		R1	R2	R3
Bearing Capacity	$\gamma_{R;v}$	1.0	1.4 **(0.71)**	1.0
Sliding Resistance	$\gamma_{R;h}$	1.0	1.1 **(0.91)**	1.0
Earth Resistance (for retaining structures)	$\gamma_{R;e}$	1.0	1.4 **(0.71)**	1.0
Soil Parameters	**Symbol**	**Set (Design Approach 1, 2, 3)**		
		M1	**M2**	
Angle of Shearing Resistance (**This is applied to tan(φ')**)	$\gamma_{\varphi'}$	1.0	1.25 **(0.8)**	
Effective Cohesion	$\gamma_{c'}$	1.0	1.25 **(0.8)**	
Undrained Shear Strength	γ_{cu}	1.0	1.40 **(0.7)**	

Overall, Eurocode 7 is less conservative than the North American codes but despite this, European countries maintain a high level of reliability and safety with their designs. This suggests that North American codes could potentially become slightly less conservative while still maintaining a high level of safety and reliability. For shallow foundations, an important aspect of this is site investigation. One aspect of this that has yet to be thoroughly explored is the way resistance factors change with varying soil parameters. A better understanding of this relationship could support a more confident and reliable reduction in the resistance factors that are currently provided in North American codes, with the potential for considerable cost savings for many projects.

5.2.5 Limitations and Performance of Shallow Foundations

In general, shallow foundations tend to be preferred for non-bridge structures because of the relatively short and simple construction required, making them the most economical foundation type. Deep foundations are generally preferred for bridge structures due to often superior performance in resisting loads and limiting settlement or agency risk. However, deep foundation systems tend to be more expensive than shallow foundation systems. Kimmerling (2002) estimated that the average bridge replacement cost in the U.S. to be about $500,000, with approximately 50% of the cost due to the substructure. Shallow foundations are known to reduce construction costs by 50 to 65% compared to deep foundations (Briaud and Gibbens 1999). This means that considerable sums of money can be saved by using shallow foundations instead of deep foundations wherever possible. Shallow foundations also have a proven record of safety for use in bridges; Washington State DOT alone constructed over 500 shallow foundation bridges between 1965 and 1980 (Dimillio 1982) and continues to do so today (Kimmerling 2002).

One key reason deep foundation systems have been predominantly favored for bridge foundations are the higher uncertainties regarding the long-term stability of shallow foundations, hence the application of LRFD to shallow foundations has been limited and relatively slow. Implementation of the LRFD method for deep foundations has been more active compared to that for shallow foundations due to the limited experimental shallow foundation system performance data available for determining reliable resistance factors. Technically, it is relatively easy to calibrate and determine resistance factors for deep foundations because static, dynamic or pile-driving instrumentation load tests are often required as a part of the construction project. More importantly, deep foundations are generally known to be preferred by state DOTs, especially in bridge substructures, due to their long-term reliability and proven records.

General concerns regarding the use of shallow foundations for bridges:

(1) The use of shallow foundations requires additional considerations at the design stage compared to deep foundation, as shallow foundations are more likely to experience changes in bearing capacity due to localized soil variability or time.

Some of the major concerns when using shallow foundations are:

- Scour – Hydraulic erosion processes can occur as a result of flowing water. This erosion can cause the structure to weaken and possibly collapse due to excessive deformation.
- Frost Heave – If a foundation is frost-susceptible, the footings must be embedded below the frost depth to prevent heave in the foundation due to soil expansion when freezing occurs.
- Moisture Sensitive Ground Conditions – If the soil is expansive and collapsible when wetted, it is not recommended for use in shallow foundations. This is because the volume changes due to absorption or expulsion of water can be large enough to cause structural damage.

- Angular Distortion – Differential settlement around the footing can develop excessive stresses in the foundation or displacement of the structure.
- Ground Water Table – The location of the ground water table also needs to be considered because this can affect both the stability and constructability of shallow foundations. If the ground water table rises, this will decrease the factored bearing capacity as it will reduce the effective vertical stress in the soil below a shallow foundation.
- Other DOTs have identified peat, organic soils, and liquefaction as major concerns.

(2) Number of data points for statistical analysis in LRFD

Compared to deep foundations, there has been considerably less work done on the use of shallow foundations for bridges. This creates a major issue for the proper implementation of LRFD with shallow foundations: lack of data. There is simply not enough data to permit statistically significant analysis and calibration. This results in the use of less precise values for the resistance factors, and hence designs with more conservatism and higher cost. Proper implementation of LRFD relies on reliability and probability theories. It is therefore the case that the reliability of the design depends on the number of statistical data points available for use in calibrating the resistance factors. Unlike deep foundations, load tests are rarely done for shallow foundations, and thus even though shallow foundations have been used for hundreds of years, in many cases load test data is very limited.

This lack of data has now been recognized in the geotechnical community and more effort is being devoted to addressing this issue. Calibration methods are moving towards using reliability-based methods, as opposed to calibration by fitting to ASD designs. To aid this process, databases for shallow foundations are being developed (Paikowsky 2011). These include a database for shallow foundations on soil, UML-GTR ShalFound07, as well as a database for shallow foundations on rock, UML-GTR RockFound07. Shalfound07 contains 549 cases, while RockFound07 contains load test data for 33 shallow foundations on rock surfaces and 28 shallow foundations below the surface.

5.3 Current Design Methods Used by SDDOT

SDDOT uses deep foundation systems for most bridge structures regardless of the geology. For bridges on shallow foundations, the occurrences are most common for when the foundations are bearing on rock or an IGM. SDDOT does not permit shallow foundations to bear on expansive shale formations due to the high risk of uplift and swell in these materials. Bearing of a shallow foundation for a bridge in South Dakota on soil is extremely rare, however the foundations for MSE walls are commonly on soil. SDDOT also designs shallow foundations for other walls and structures across the state that do not coincide with bridges.

As the bridge shallow foundations in South Dakota often bear on a rock or an IGM, shallow foundation design method used by SDDOT is based on partly on design equations and partly on past performance rather than presumptive allowable bearing capacities. This past performance driven design framework or approach is based on the idea that if sufficient high quality and reliable field and laboratory tests are not available (and rarely are, given the geologic materials in South Dakota), then rather than use inferior test data in a complicated calculation, or assume a bearing capacity, that bearing capacities estimated or back-calculated from past performance of other foundations should be used to constrain the results of calculations. This is reasonable as the bearing capacity equations and presumptive values used for conventional AASHTO designs assume consistent sand, clay, or hard rock below the shallow foundations, a situation which is rare in South Dakota. These consistent geologic materials tend to be easy to investigate, drill, sample, test and quantify. When shallow foundations are to be placed on thick sand or clay, SDDOT uses calculation-based designs in these materials which are more readily constrained from an engineering site characterization perspective.

Unfortunately, the soft rock and IGMs that are most common in South Dakota are not easy to investigate, drill, sample, test or quantify. As an example, in conventional clay soils, a Shelby Tube sample of undisturbed clay is sampled in the field and transported to a laboratory where the soil is extruded, trimmed and then tested in Unconfined Compression (UC), 1D consolidation, Unconsolidated Undrained Triaxial Compression, and other tests. In contrast, the IGMs and soft rock in South Dakota are difficult to sample, experiencing large amounts of disturbance during sampling, transportation, extrusion, trimming, and testing. This makes the field and laboratory test results less reliable and raises questions as to the veracity of the samples for conventional design calculations.

Past performance design methods on the other hand, follow the logic that the best predictor of future system performance is past system performance. If a thorough investigation is performed at a bridge site that characterizes the geologic material sufficient that the variability and uncertainty can be qualitatively or quantitatively known, then bearing capacities observed in previous footings in similar geologic materials will provide an upper-bound value for design of the current system, with calculations providing the lower bound. This method is not optimal, as it depends on sufficient past performance observations in a sufficient number of geologies. However, as the risks associated with designing using inferior laboratory or field data are high compared to the past performance approach while presumptive values are overly conservative, past performance approaches may be favored. Past performance design methods are part of the Observational Approach recommended by Terzaghi, Peck, Hansen, Thornburn, Casagrande and other pioneers of geotechnical and foundation engineering they combine both observations and calculations.

The past performance approach for rock in SD has resulted in SDDOT using allowable capacities of 4, 8, and 20 ksf for shale, schist, and quartzite formations respectively. SDDOT does not have past performance values for other formations such as sandstone. These past performance values are much less than the presumptive capacities in AASHTO, and have been shown to work well in SD by the virtue of no shallow foundation failures using these allowable capacities on rock. However, the margin of safety with these past performance allowable bearing capacities on rock is unknown, and will remain unknown until a foundation load test to failure is performed.

In 2007 when AASHTO and FHWA began requiring LRFD on federally funded transportation projects, SDDOT reached out to FHWA for assistance in converting existing ASD design methods to LRFD. The FHWA Service Office worked with SDDOT to develop a "soft calibration" of pre-existing ASD methods to LRFD. This was allowable under FHWA's guidance since SDDOT had not had any foundation failures and therefore the existing ASD methods were deemed sufficiently conservative. This "soft calibration" consisted of examining the FS and allowable capacities in LRFD and developing a resistance factor for each foundation and analysis or testing type to give a resulting LRFD factored load equivalent to the ASD allowable load. In this "soft calibration" a variety of resistance factors were developed, primarily for driven piles with equivalent LRFD resistance factors ranging from 0.4 to 0.7. For shallow foundation design, these "soft equivalent resistance factors" have been simplified to a resistance factor of 0.55 to 0.7 to be applied to the ultimate geotechnical resistance (i.e. divide ultimate resistance by 1.4 to 1.8).

The SDDOT current design methods can be summarized via the following list, or workflow, that shows how the past performance driven design framework is implemented:

- SDDOT begins the design with a drilling program:
    - Hollow or solid-stem augers are used in soil and IGMs. Rock coring equipment is used in competent rock formations. Drill tooling may be switched during drilling if competent rock is encountered below soil or IGMs.

    - Rock cores are transported to the laboratory for further analysis by a SDDOT foundation engineering department staff.
    - In soil and IGMs, a driven sampler with 2.5-inch outer diameter and 2.0-inch inner diameter (also known as a "modified California sampler" in other states) is driven using a standard SPT hammer to obtain samples. Soils and IGM tend to be too stiff and hard for Shelby tube sampling.
    - Number of blows measured to drive the sampler 18 inches are recorded over 6-inch increments. This record represents the "blowcount" of the sample.
    - Borings are planned to extend to a depth of 2 x the wall or bridge height below the ground surface. Deeper borings are used when deep foundation systems are anticipated. The focus for explorations is the upper 10-ft below foundation bearing grade, as this zone bears the highest stresses from the shallow foundation loading.
- During drilling, a sampling program is undertaken wherein samples are collected at regular intervals in all borings. Generally, SDDOT collects 6 samples in the upper 10-feet below ground surface and collects an additional 6 samples below 10-feet deep.
- At the time of drilling or soon afterwards, a geotechnical engineer identifies the geologic formation encountered during drilling.
    - If the geology consists of expansive shale, no shallow foundations for bridges are allowed. Walls at these sites are designed with a large under-cut below the wall and replacement of the over-excavated material with non-expansive structural fill. Drains are placed behind the wall in free-draining fill to minimize water intrusion below the wall into potentially expansive shale.
- Laboratory testing
    - Index tests are the primary laboratory test performed by SDDOT. These tests are used to constrain material types and distributions at the site. Several index tests are performed on samples from each boring and include one or more of the following: unit weight, water content, fines content, sieve analysis, hydrometer analysis, and Atterberg Limits.
    - Strength tests are then performed on a small number of driven samples. The number is chosen on a case-by-case basis based on the variability

of the site and the variability of field data. The most common tests are the Unconfined Compression (UC) Test and the Direct Shear Test.

- 1D collapse/swell and Consolidation tests are lastly performed to evaluate the potential of the material to collapse or swell upon inundation or saturation, or the potential of the material to settle under design loads. SDDOT chooses the number of tests for a given project based on the geology, variability, and results of field testing. Sites with expansive shales, silts, or clays receive more of these tests.

- Once all field and laboratory testing are completed, the design engineer performs a review and interpretation of data. This includes georeferencing exploration locations.
    - With georeferenced exploration locations and an understanding of grades, cuts and fills from the civil designer on the project, the design engineer develops a lithology model.
    - Development of the lithology model includes a qualitative evaluation of site variability, which leads the design engineer to better understand the risks to the bridge or wall structure posed by the site geology.
    - A well-developed lithology and understanding of site variability allow the design engineer to derive and select design parameters to be used in design calculations. At this stage, the design engineer calls upon the past performance of similar structures in similar conditions to assess and evaluate the design parameters, especially a unit bearing capacity for the geologic formation at the site.
- The design is then performed by the design engineer.
    - Standard calculations of foundation settlement and bearing capacity are performed for footings to bear on soil using the derived design inputs based on laboratory and field tests. If the footing bears on rock, then the SDDOT standard past performance allowable capacities are used.
    - Past performance is brought into the design again at this time as the design engineer uses the knowledge of the lithology, field and laboratory test data, and site variability to select an ultimate or nominal bearing capacity for the project.
    - An LRFD resistance factor is used to reduce the nominal bearing capacity to the allowable (factored) bearing capacity. The footing is then

sized to accommodate that factored bearing capacity based on the applied design structural loads.

    - SDDOT uses a resistance factor of 0.7 for rock and 0.55 for soil, from the "soft calibration" explained previously.

  - Over-excavation is specified if the subgrade is soft or expansive
  - Horizonal loads and moments are accounted for in footing eccentricity calculations which are performed by the structural engineer for the project, which often end up governing as foundations must be widened above that determined using the bearing capacity to resist overturning of the footing.

- Instrumentation is occasionally installed in the foundation or wall as it is being constructed, with monitoring of the instrumented settlements and deflections performed for several months after construction.

Contrast this past performance driven method to the use of presumptive bearing capacity values from table C10.6.2.6.1-1 in AASHTO. The major advantages of using a presumptive bearing capacity are that a foundation design can be selected quickly without requiring in-depth geotechnical test data and there is no need for repeated adjustments between structural engineers and geotechnical engineers regarding the size of the footings required. The limiting of the bearing capacity to a presumptive bearing capacity value has an additional advantage in minimizing the impact of eccentricity in the overturning calculations and final design. Eccentricity and overturning are generally known to control shallow foundation design on rock. However, this approach does mean that the design is usually conservative, with a higher safety margin (e.g. factor of safety) implicitly embedded in the design, as it cannot take into account local characteristics and ground conditions in a quantitative way, and thus, each design has different probability of failure even under same probability of occurrence of an event. Note AASHTO only allows the presumptive approach when rock is rated fair or better, which requires evaluation of the rock mass by an engineering geologist based on rock coring and geologic mapping. If a qualified engineering geologist has not performed geologic mapping and rock coring to evaluate rock classification, presumptive bearing capacities are suspect. As not all design work by SDDOT is afforded a geologist, the use of the more conservative past performance capacities in rock is more appropriate than use of the AASHTO presumptive values.

To document the design methods used by SDDOT over the last 60 years, the research team reviewed the paperwork for 27 geotechnical reports for 22 selected SDDOT construction projects carried out between 1957 and 2015, as shown in Table 5.27. The collection started with recent projects that may include shallow foundations. Due to the limited availability of eligible bridge projects, projects with retaining wall designs are included. Of the 27 projects, only 5 incorporated shallow foundations for bridges, all of which were designed on rock. The two projects executed in the 2010s suggested using a factored bearing resistance approach for the design, but instead appear to be based on past performance method values of 8,000 psf and 20,000 psf, respectively. Of the remainder, 14 projects are not bridge foundation projects but retaining wall projects with foundations for mechanically stabilized earth (MSE) walls.

In general, bearing resistance estimation is required when conducting an external stability check for an MSE wall design. However, most of the projects reviewed in this study did not document the design of the walls (performed by the wall vendor's designer and not included in the State's files), and only provided information on site and design parameters such as the unit weight, cohesion, and internal friction angle determined by boring logs, direct shear tests, and/or unconfined compression tests. Seven of the projects included a consolidation settlement analysis anticipating settlement issues and suggested some form of pretreatment of the foundation ground. Only 3 of the 14 MSE wall projects included details of a bearing capacity stability check; all used a factor of safety of 2.5. However, no further information was available on how this factor of safety was determined. The other 11 MSE wall projects documents did not disclose details on the bearing capacity and sliding checks. A summary of the complete findings of the review of current SDDOT design methods is attached as Appendix B.

Table 5.27 Summary of randomly selected SDDOT projects with foundation designs.

Project	Locations (County)	Project year	Foundation Types
PCN 01KL	Hamlin	2015	Driven pile
PCN 03A6	Spink	2015	Driven pile, drilled shaft
PCN 021X	Minnehaha	2015	Driven pile, spread footing on rock
PCN 01E2	Minnehaha	2013	Driven pile, spread footing on rock
PCN H100	Pennington	2006	Driven pile, drilled shaft
PCN 4259	Pennington	2005	MSE wall
PCN 5992	Edmunds	2004	MSE wall
PCN 5899	Brown	2004, 2003	MSE wall
PCN 3443	Minnehaha	2004	MSE wall
PCN A443	Minnehaha	2003	MSE wall
PCN 1177	Minnehaha	2002	MSE wall
PCN 6116	Pennington	2001	MSE wall
PCN 0664	Brown	2001, 2000	MSE wall
PCN 3453	Pennington	1999	MSE wall
PCN 4083	Brown	1999	MSE wall
PCN 3150	Minnehaha	1998	MSE wall
PCN 0547	Spink	1998	MSE wall
PCN 3574	Spink	1998,1997	Driven pile and MSE wall
PCN 1410	Clay	1994	MSE wall
N/A	Minnehaha	1959	Driven pile and spread footing on rock
N/A	Minnehaha	1958	Driven pile and spread footing on rock
N/A	Minnehaha	1957	Driven pile and spread footing on rock

Since 2015, all of the reviewed reports and documentation show that LRFD methods are being used. Prior to 2015, a mixture of LRFD and ASD designs were performed, with the first LRFD concepts documented in 2004. Prior to 2004, only ASD type designs were found. Geotechnical explorations were exclusively rock coring or borings with SPT or oversized Mod-Cal sampler blow counts as the only in-situ test for measurement of in-situ soil or rock properties.

Development of parameters for engineering calculations from available laboratory and in-situ testing is generally not clear in the available paperwork, with laboratory and in-situ test data that may conflict with the chosen design parameters with little justification. This is especially true for sites where rock is present at or near the bearing depth. Use of an engineering geologist or geologist to evaluate the rock competency is missing from all reports as SDDOT did not have an engineering geologist or geologist on staff in the foundation engineering group in the time period of review, rather SDDOT uses engineers to fill this role. In most cases for rock, past performance bearing capacities were used, though the rationale for use of the specific presumptive bearing capacity was often missing from the documentation. No projects appeared to directly consider differential settlements of foundations, MSE walls, or MSE wall foundations from variations in subsurface conditions. Documentation was generally poor in the reports reviewed as to soil corrosion potential for foundations or MSE reinforcing elements, with few recommendations provided for mitigating soil corrosion potential when identified.

Every project carefully considered potentially expansive soils or rock. Not all project documents included testing for expansive soils, but all projects considered it in design. All project documents contain recommendations and construction details for mitigation of expansive soil or rock if present.

Unfortunately for furthering the use of LRFD for the State of South Dakota, no field performance data are available for these 22 projects so a calibration to local conditions cannot be performed using historic designs. Likewise, no local full-scale load testing was identified to be used in full reliability-based calibration of LRFD factors for local conditions. Insufficient field and laboratory data were present for nearly all of the projects for statistical analysis of subsurface conditions. The authors of this report are unaware of any regional rigorous instrumentation or observational tracking program of presumptive shallow foundation design on rock [short to long-term] performance that has been undertaken to prove if the past performance driven approach is superior to other approaches.

5.3.1 Variance of SDDOT Method Compared to AASHTO

In general, the research team notes that SDDOT complies with much of the AASHTO requirements for LRFD under the 2017 provisions. There are a few exceptions, however, that are noted in this Section of this report. In some cases, SDDOT exceeds the standards outlain in AASHTO LRFD (2017), most notably in the design of foundations on expansive soils. One of the difficulties in determining the variance between SDDOT methods and AASHTO is that AASHTO allows the use of presumptive bearing capacities, which is a lower bar than SDDOT allows. However, the SDDOT use of past performance nominal bearing resistance for rock is often poorly documented in project files. Therefore, the project research team relied on interviews with SDDOT design engineers to ascertain the variances.

The primary variance between SDDOT methods and AASHTO is the use of past performance driven design compared to the soil/rock design-parameter driven design in AASHTO. This variance has been hitherto discussed in this report. Recommendations to merge the past performance driven design with AASHTO requirements is presented in the next Section of this report.

The second variance between current SDDOT methods and AASHTO LRFD (2017) methods is how to handle materials that classify neither as “classic” “rock” or “soil”, the IGMs. AASHTO separates the materials into three categories with differing methodologies and resistance factors. The project files that the research team reviewed included no explicit determination of soil versus IGM versus rock, despite field and laboratory data that indicated some materials may indeed be IGMs.

5.4 Suggested Shallow Foundation Design Improvements for SDDOT

The research team evaluated dozens of unique or innovative approaches to shallow foundation design that are being utilized in other states and nations. In review, several engineering means, methods, and approaches were shown to be promising for SDDOT to incorporate into their shallow foundation approach that will assist in use and implementation of AASHTO LRFD resistance factors and shallow foundation design. When coupled with the current design methods employed by SDDOT, this section provides a set of suggested shallow foundation design improvements for SDDOT engineers to use in the future.

1. AASHTO places equal emphasis on footing settlement (deflection) as it does geotechnical capacity. SDDOT should add the following to their approach:
    a. Total and differential settlement performance criteria that can apply to shallow foundations and to walls alike. A commonly used criteria by DOTs across the US that SDDOT could consider adopting would be developed based on the studies presented in Section 5.1.3:
        i. 2-inches total settlement post-construction, with no more than 0.5 inches settlement differential over 50 feet for footings and walls more than 50-ft from a bridge abutment.
        ii. 1-inch total settlement post-construction with no more than 0.25 inches settlement differential over 50 feet for footings and walls

within 50-ft of a bridge abutment. This would be a consistent settlement criterion as commonly used for deep foundation supported bents and abutments.

iii. No more than 12 total inches settlement for any walls during construction.

iv. No more than 2 inches settlement for any non-wall footing during construction (i.e. bridge abutments or bents).

b. For settlements of foundations and walls on soil, using the calculation methods presented in Section 5.1.3, or other settlement calculation methods used by other DOTs. The methods allowed by UDOT, CALTRANS, and WASHDOT are rigorous and well vetted and can be used for a wide range of geologic materials.

c. Foundation design experience of the research team shows that for footings on soil, that the allowable loads needed to meet these settlement performance criteria will govern over the strength capacities. For IGM and rock, capacity will govern over settlement.

2. Incorporate soil corrosion and chemical attack testing into every shallow foundation design. SDDOT typically uses NRCS soil maps for this, but regional soil mapping does not meet typical national standards of practice. The national standard of practice is site-specific soil analytical chemistry testing. The research team only found a few instances of these tests in the reviewed documents. More may be performed by SDDOT but were not documented. The soils in SD are noted for their aggressive chemical attack on concrete, asphalt and steel. These tests will be of critical use for pavement design and maintenance and deep foundation designs. This is accomplished through a set of analytical chemistry.

   a. These tests include:

      i. pH

      ii. Resistivity

      iii. Sulfates

      iv. Sulfides

      v. Soluble Salts (chlorides)

vi. Redox Potential

b. Criteria for when modifications for design are required can be compiled by SDDOT form a variety of sources. ACI, the American Water Works Association, and CALTRANS have excellent criteria that can be used by SDDOT to develop their own guidance on soil corrosion and chemical attack potential for use in design of remediations.

c. Remediations may include additional clear cover over rebar, epoxy coating rebar, specialty Portland cement mix designs or admixtures, and/or cathodic protection.

3. Use established AASHTO RMR procedures for development of geotechnical capacities for IGM shale, shale and other rock formations. These procedures are presented in Appendix A. These RMR procedures can be used by design engineers for development of rock capacities for all IGM and rock formations in SD. The benefits of the RMR approach for all rock and IGM is that the discrete strength and settlement parameters needed for the calculation-driven design of AASHTO LRFD can be developed rigorously for the entire formation. These RMR values can then be adjusted based on past performance and past experience with the formation. Ideally, these would be adjusted for SD specific conditions from a set of full scale load tests of shallow and deep foundations to calibrate both the LRFD resistance factors and material strengths with local load test data.

4. The majority of shallow foundation designs in the reviewed records were for cases where the foundations were in an area of expansive clay or shale, necessitating remediation with over-excavation and replacement. In these cases, the design engineer can take advantage of this over-excavation and replacement by including the superior strength and settlement characteristics of the frost-resistance granular structural fill specified by SDDOT for the replacement. SDDOT can do this by using the two-layer bearing capacity calculations presented in AASHTO and other foundation design manuals (such as FHWA and a myriad of textbooks). In these 2-layer calculations, the soil is not assumed to be a homogeneous and isotropic half-space, but as a layered system, with distinct strengths for two layers. The upper layer may be stiffer or softer than the lower layer. The design procedures outlined in AASHTO for layered systems can often result in increases of ultimate bearing capacities by as much as 200% in

the experience of the research team. Addition of a 5-ft layer of Phi = 38° material (for example) is a remarkable addition of strength and stiffness to the system that should be taken advantage of in designs. As a double advantage, if there is soil overburden over shallow rock at the site, these two-layer methods allow the rock to be added to the calculation, which may increase capacities significantly.

5. AASHTO LRFD presents a wide range of resistance factors for shallow foundations that have been presented herein for different analysis and investigation methods. Rather than use a “soft calibration” of older ASD designs, in which the actual margin of safety is unknown, it is preferable to use the recommended resistance factors from AASHTO, with two modifications:

    a. For IGMs, AASHTO does not provide LRFD resistance factors for shallow foundations. In this case, the “soft calibration” resistance factors should still be used until the time that a complete load test program and state-specific resistance factors can be developed.

    b. For sites with high or low uncertainty, the work of Fenton et al. (2008) showed that resistance factors can be adjusted “up” or “down” based on the design engineer’s understanding of the uncertainty in structural loading, site characterization, and lithology. The default AASHTO or IGM resistance factors should be adjusted in similar manner for sites in SD where the design engineer has a high or low degree of understanding of the site uncertainty. Figure 5.12 shows the recommendations of Fenton et al. (2008), and an associated increase or decrease of 0.05 to 0.1 for the degree of uncertainty at a site.

    c. Note that AASHTO allows significant increases in resistance factors for sites with full scale load testing. If SDDOT needs to increase resistance factors, a site load test or high-load plate load test can be a economical alternative to intensive exploration programs.

Shallow Foundation Resistance Factors	Degree of Understanding		
	Low	Typical	High
Bearing	0.45	0.50	0.60
Vertical (settlement) or lateral movement	0.70	0.80	0.9

• High Understanding – This level of understanding applies when models are used that have been proven to achieve a high level of confidence with their predictions; extensive site investigations.
• Typical Understanding – Here, conventional prediction models are combined with a standard project-specific site investigation.
• Low Understanding – Conventional prediction models are combined with an understanding of the site based on previous experience or extrapolation from nearby or similar sites

Figure 5.12. Modification of Resistance Factors for degree of understanding. After Fenton et al. (2008).

5.5 Conclusions

A literature review of the concepts, theories, applications, and implementation of LRFD for shallow foundation designs was conducted for this report, focusing particularly on the materials published by federal and state agencies, including SDDOT. The load and resistance factor design method considers uncertainties of load and resistance separately so that factors for each can be adjusted independently to take into account a different level of uncertainty during the lifetime of the structure. This approach was first introduced for structural designs and then extended to geotechnical designs because the characteristic properties of materials such as concrete or steel are considerably more homogeneous than those of soil or rock.

LRFD for bridge designs has been mandated by FHWA since 2007, and many studies have been conducted to help state engineers to make an appropriate and easy transition to the new approach. The first edition of AASHTO that includes resistance factors derived from factors of safety fitting to ASD was released in 2002, and this has been updated a number of times since; the current 8th edition was published in 2017.

Several studies have been conducted on the implementation of the LRFD method specifically in shallow foundations, investigating not only the introduction and application of LRFD but also the use of shallow foundations for bridges. The results of the surveys of state DOTs indicate that the use of shallow foundation in bridges is relatively limited, mostly due to designers' concerns regarding the long-term stability of the structures under harsh environments such as scour. Most states use the resistance factors provided in the AASHTO LRFD (2017) because of a lack of LRFD parameters that have been locally calibrated by via fitting them to the results obtained using past performance on previous projects or via reliability analyses of full-scale load test data.

Efforts to conduct local calibration of the resistance factors seem to have been few and far between, mostly because of the limited number of shallow foundation bridges that have been constructed and the consequent shortage of test data. Only Pennsylvania and Missouri, were found to have conducted any localization calibrations. Pennsylvania DOT has been using shallow foundations very actively compared to other states: 65% of their recently constructed bridges have shallow foundations, of which 60% are on rock, although this does mean that their calibration has been made mostly for resistance factors with rocks. In contrast, Missouri DOT has developed charts showing variations of resistance factors with coefficient of variation of the core parameters and target probability on both rock and cohesive soils. This method can consider the different reliabilities of direct, indirect, or combined test data, as well as the different levels of importance of the structures. However, they also used the AASHTO resistance factors for cohesionless soils.

The application of LRFD for service limit design has been less actively pursued. In AASHTO LRFD (2017), load and resistance factors for settlement analyses at the service limit state are assumed to equal 1.0 because reliability-based load and resistance factors have not yet developed. In general, most state specifications specify 1-inch of settlement for spread footings. FHWA (2002a) suggests ≤1 in and ≤ 1.5~2 in of settlement for continuous and simple span bridges, respectively. MoDOT suggested the use of resistance factors to modify the compression and recompression indices for cohesive soils and the uniaxial compressive strength of rocks.

The current study reviewed 22 of SDDOT's geotechnical projects that utilized shallow footings, most of which were conducted post-1990. The results show that the adoption of spread footings has been very limited in South Dakota for bridge structures, with most examples being foundations on rock. This dearth of shallow foundation applications limits the reliability of the localization of this method in the state through fitting to ASD. In addition, no plate load test or full-scale load test data was generated during any of the projects. Of the bridge and wall projects that used spread footings, project files obtained in this study indicated that bearing capacity nominal resistance for rock was based on the design engineer's prior experience and SDDOT past performance knowledge from previous projects. It was deemed unfeasible to conduct a localization exercise given the lack of suitable data. There is also some doubt as to whether the economic benefits would make such an approach worthwhile.

Several field protocols such as RMR have been developed in states with geology similar to South Dakota, wherein conventional field and laboratory tests are insufficient due to the challenging nature of the materials or where RMR can supplement or augment results of UC tests. This may be very beneficial to SDDOT, as shales can be difficult to sample for UC testing in the laboratory.

In order for SDDOT to continue using the past performance driven designs, it would be beneficial for SDDOT to develop a database of past performance so that the rationale of the past performance driven design can be documented in project files. A past performance database consists of individual case histories from each project. The case history includes all of the field data, lab data, construction drawings and performance data from the project. This database should include case histories of projects in neighboring states as well as case histories from other states and Canada when dealing with IGMs and rock geologies similar to conditions in South Dakota. Once the initial database is compiled, a data analysis should be performed to identify trends and to assess the variability and uncertainty across the state and neighboring states. SDDOT needs a way to quantitatively implement the past performance approach that is being qualitatively implemented. This database will allow SDDOT to perform the past performance approach quantitatively. The essential part to each case history is the geotechnical instrumentation and monitoring that shows the past performance with measured data. Without measured data, a complete case history cannot be compiled as there is no means to determine if the structure was marginally stable or overly stable. In other words, without the performance observations there is no means to identify if a design has been overly conservative or if collapse was only avoided by a narrow margin. Neither case is desirable for SDDOT to base future designs on.

Part of a past performance driven approach is to know both quantitatively and qualitatively how close a design is to a failure condition. In past performance monitoring of previous construction, it is very unlikely that SDDOT (or any DOT) purposefully allowed any structure or system to reach a failure state. Therefore, the database is likely to be heavily skewed to the conservative side. A database skewed is a database that is difficult to develop statistical relationships for. Thus, there is a need to couple the field data with full scale load tests that show where failure states actually are for the geologic conditions in South Dakota. Full scale load tests are much more common in deep foundations, and AASHTO LRFD (2017) allows for more aggressive resistance factors for sites/regions with full scale load testing. AASHTO allows these higher resistance factors as full-scale load tests prove the failure condition and reduce the uncertainties that drive the need for the resistance factors. If SDDOT were to perform a small number of full-scale shallow foundation load tests on well characterized, typical South Dakota, geologic materials, the uncertainties could be reducing, and higher resistance factors can then be recommended for use in the state.

One last item that the research team observed that can be easily adjusted is for SDDOT to improve shallow foundation design documentation. Documentation of shallow foundation design methods, rationale, assumptions, and justification of assumptions was poor in the project files that the research team reviewed. Fortunately, the research team was able to interview design engineers to fill in the gaps to obtain a complete picture of past design methods and use of LRFD resistance factors. It is essential in developing a database of case histories, with an eye to future calibration of local LRFD resistance factors, to document each and every step of the design process including the rationale for selecting the past performance driven bearing capacities AND the identification of geologic formation and declaration of soil versus IGM versus Rock. It may take several years to develop a complete database of shallow foundation case histories and full-scale load tests. With this database SDDOT could calibrate and derive LRFD resistance factors that constrain uncertainty in the materials within the state and the past performance driven method.

6.0 RECOMMENDATIONS

To implementing development and calibration of LRFD resistance factors for geotechnical or foundation design, a state DOT has three options: Level 1) adopt AASHTO's LRFD load and resistance factors, and reliability index; Level 2) develop a set of LRFD parameters that are locally calibrated by fitting them to the results obtained using the existing ASD method and field performance data of the designs during and after construction; and Level 3) develop a set of LRFD parameters that are locally calibrated through reliability analyses of load test data.

The SDDOT is currently designing foundations using the first of these options, Level 1, but asked the research team to develop the Level 2 option. However, due to the limited information available from previous projects that have adopted shallow foundation design, as well as the limited number of shallow foundation designs in the state, the refinement of LRFD resistance factors at Level 2 is not considered viable at this point. However, it is recommended that the South Dakota Department of Transportation should prepare a plan to implement Levels 2 AND 3 by carefully planning upcoming state projects to develop the inputs for conducting future refinements.

Levels 2 and 3 of LRFD resistance factor development both require comparison of field performance data with geologic material data and foundation design parameters. For Level 2, settlement performance data is critical from a host of projects so that variability in foundation performance can be compared to the variable soil conditions and design methods. For Level 3, full scale foundation load tests to failure add another level of complexity as these tests help constrain the ultimate bearing capacity of the soil-rock-foundation system.

The research team therefore suggests a list of items to be considered to support safe and economical design of spread footings. These recommendations are based on the level of adoption, gradually increasing the reliability of the design process by accumulating more data on the required parameters and thus support the implementation of local calibration.

6.1 SDDOT use AASHTO LRFD (2017) Bridge Design Specifications resistance factors of shallow foundation designs adjusted for uncertainty until appropriate local calibration and development of SDDOT specific resistance factors has been conducted.

SDDOT use AASHTO LRFD (2017) Bridge Design Specifications resistance factors of shallow foundation designs adjusted for site-specific uncertainty.

At this time insufficient data is available to develop Level 2 or Level 3 LRFD resistance factors for use in the State of South Dakota. Until this time that the subsequent recommendations in this report are followed for sufficient time that the requisite data are available, the project team recommends that SDDOT continue to use the AASHTO LRFD Bridge Design Specification Resistance Factors. However, with sufficient effort by SDDOT, both Level 2 and Level 3 datasets are attainable within 10 years that will facilitate future development of South Dakota specific Resistance Factors. Adjustments for uncertainty should be made on a site-specific basis.

6.2 SDDOT implement quantification of rock, soil, and Intermediate Geomaterials (IGM) on all projects, as recommended by AASHTO LRFD (2017).

SDDOT implement quantification of Intermediate Geomaterials (IGM).

One of the primary obstacles to Level 2 or Level 3 LRFD implementation in the State of South Dakota are the frequent occurrences of geologic materials not “soil” or “rock” under the definitions of AASHTO. These materials require different methods in AASHTO compared to “soil” or “rock”. In order to develop the Level 2 and/or Level 3 datasets, sufficient datapoints in the IGM category are required. To achieve this, the project team recommends that SDDOT carefully classify geologic materials statewide in a future research project per the guidelines of AASHTO (2017) so that general state-wide geologic data and foundation performance data can be appropriately coupled in future Level 2 and Level 3 databases. An additional benefit to SDDOT is the ability to use Level 1 Resistance Factors for IGMs that are generally less conservative than those for “soil” or very weak “rock.” See AASHTO 10.4.6.4.

6.3 SDDOT Implement Rock Mass Rating (RMR) Procedures in Shales and Other Rock

SDDOT implement RMR shale characterization procedures to augment the current usage of Unconfined Compression (UC) tests in shales. Widespread occurrences of IGM and other weak “rock” or strong “soils” in the State make widespread use of conventional geotechnical site characterization methods for obtaining material strength and compressibility difficult, if not impossible. Fortunately, RMR protocols for characterization of shales and soft rocks are able to overcome these shortcomings in material drilling, sampling, handling, and laboratory testing. RMR procedures have high reliability, relatively low cost, and have been shown to obtain excellent strength and compressibility results in IGM and similar geologic materials. In order to develop Level 2 or Level 3 Resistance Factors, reliable geologic data is essential, and these methods are ideal for populating the geologic datasets needed for Level 2 or Level 3 development and calibrations. These methods can be used immediately by SDDOT and increase the reliability of current past-performance driven methods and Level 1 Resistance Factors. See AASHTO 10.4.6.4.

6.4 SDDOT standardize the geotechnical exploration process and foundation design input parameter development for soil and rock for projects within the state and collect these into a centralized database of data and parameters, which will be an essential resource for future calibration/refinement.

SDDOT develop a centralized database of soil and rock parameters.

Essential to recommendations 2 and 3 of this report are that these improvements to the SDDOT current design process be documented and included into a centralized database of geologic conditions and data. This database must be coupled with additional datasets to perform Level 2 and/or Level 3 LRFD resistance factor development. A standardized process will facilitate compilation of the database. If all project designs have a consistent method to document raw field and laboratory data, selected design parameters, and past-performance based assumptions, these can be easily incorporated into a database that is added to with each new project. Data from deep foundation projects should be included in this database, as projects utilizing deep foundations have data that can be leveraged for shallow foundation LRFD resistance factor development.

6.5 SDDOT instrument and monitor bridge foundation and MSE wall settlements and develop a centralized database of shallow foundation performance that includes foundations and walls monitored previously by SDDOT and in neighboring states.

SDDOT develop a centralized database of shallow foundation performance.

Field and laboratory test data is not sufficient for LRFD resistance factor development at Level 2. Essential to the LRFD Resistance Factor are the concepts of risk, reliability, variability, and performance. Therefore, to perform Level 2 Resistance Factor development, a database of foundation and wall performance is needed. Ideal data for this database is settlement of walls and bridges. Settlement can be used directly to compare with geologic material compressibility data from field and laboratory testing. Settlement data can also be used to infer stability. Performance can then be compared to design expectations to show how existing Level 1 Resistance Factors fared in design. Answering if the design was sufficiently conservative or over-conservative. Additional value is gained by SDDOT in that in the current past-performance driven design framework, that will continue until Level 2 and Level 3 inputs are available, new engineers to SDDOT and new design consultants will have a set of actual performance data that is documented and reliable to draw upon when making past-performance driven design decisions. Settlement data for projects should be supplemented where appropriate with inclinometer, pore pressure, and other data types. Settlement data for non MSE wall or Bridge foundation projects are helpful to development of LRFD resistance factors for geologic material and should be considered for inclusion in this database. Geologic material properties recommended in 6.5 should be paired with performance data and as-built drawings so that performance can be properly evaluated.

6.6 SDDOT perform a set of well instrumented full-scale shallow foundation load tests, to failure, to add critical collapse condition data points to the foundation performance dataset.

SDDOT perform full-scale footing load tests on soil or IGMs.

Recommendations 2 to 5 are essential for Level 2 LRFD Resistance Factor development. In order for SDDOT to achieve state-specific Resistance Factors in Level 3, full scale shallow foundation load tests to failure on typical South Dakota geologic materials are essential. Full-scale load tests can be performed as part of project, but also performed as independent research projects. A full-scale load test for a shallow foundation is similar to that for a deep foundation, with reaction piles and a load frame used to apply a vertical load to a shallow foundation bearing on the appropriate material. The foundation itself is instrumented with strain gages, and the overall load: deflection of the shallow foundation behavior is measured along with internal strains. Full scale load tests show the developers of Level 3 Resistance Factors the probability of failure to be anticipated from the design methods employed. The data also show how the system performs through a range of strains all the way to failure, which is the state that LRFD seeks to avoid. Full scale load tests are in addition to performance data (that is not carried all the way to failure). Sites for full scale load tests should be characterized per recommendations 2 and 3 of this report if possible.

As a side note, a deep foundation is often a “tested” foundation. Any ongoing deep foundation load test or dynamic pile testing compiled into a deep foundation database for LRFD development will be useful for engineers developing Level 2 and/or 3 for shallow foundations.

The project team recommends that the deep foundation data be included as “full-scale” load test data to supplement the recommended shallow foundation load tests.

6.7 SDDOT use the centralized database of geology, geologic data, laboratory data, field monitoring and performance, and load tests to develop a set of South Dakota specific LRFD resistance factors for shallow foundations on rock, soil, or IGM.

SDDOT develop state specific LRFD resistance factors.

In order to reduce uncertainty on shallow foundation projects, to reduce unnecessary over-conservatism and associated costs, the project team recommends that SDDOT follow the previous recommendations of this report and compile the requisite database(s). At such time as sufficient data is available that statistical analyses can be performed on the database(s), the project team recommends that SDDOT perform the calibration for both Levels 2 and Level 3 of LRFD Resistance Factor development. The research team recommends Level 3 as the interesting and difficult nature of geologic materials in South Dakota require state-specific assessments. This recommendation is in-line with current SDDOT design methodologies that favor past-performance. This recommendation is really a formalization of the current method to be consistent with the intent of LRFD as recommended by AASHTO; to reduce both risk and costs for highway projects.

6.8 SDDOT evaluate the use of more shallow foundations systems for bridge structures for bridges without scour hazards, wherein foundations are placed on reinforced structural fill.

SDDOT evaluate use of innovative shallow foundation systems.

New and novel shallow foundation systems have been introduced within the last decade that circumvent the need for State Specific LRFD resistance factors. The primary example are shallow foundation systems that utilizes shallow foundations on geosynthetic reinforced structural fill. By placing the shallow foundation on structural fill that is heavily reinforced but ductile, the Level 1 LRFD Resistance Factors for dense imported A-1 granular fill remains adequate. These shallow foundation systems have additional benefits in terms of construction cost and schedule that make them attractive alternatives to conventional deep foundations for sites that have historically had too poor of soils for shallow foundations. Use of these novel foundation systems also avoids the need to developing large database(s) for Level 2 and 3 Resistance Factor development. The project team recommend that SDDOT select one new highway bridge or MSE wall with no scour hazard, and geologic materials where deep foundations are typically used, to test one of these novel foundation systems with reinforced structural fill. The test system should be instrumented and monitored for performance over a period of 5-years to demonstrate the effectiveness of the systems for South Dakota conditions. FHWA provides several excellent design guides and commentary for LRFD Resistance Factor selection for several of these systems.

6.9 SDDOT utilize layered bearing capacity equations for cases of over-excavation and replacement above weak subgrades; and

SDDOT utilize layered bearing capacity equations for cases of over-excavation and replacement above weak subgrades.

Current SDDOT shallow foundation design procedures for estimating factored bearing capacities are only single-layer calculations. Single-layer calculations assume that the subgrade is homogeneous and isotropic. In many designs, however, SDDOT specifies a thick over-excavation and replacement of weak subgrade materials with structural fill. For these cases, it would be advantageous to take advantage of the over-excavation and replacement with structural fill by using bearing capacity equations for two-layer systems. Examples are found in both AASHTO and in most foundation design textbooks. These methods allow the high friction angles of structural fill to be used for the upper layer, and the weak subgrade material properties are used as in current design methods. These methods are often able to raise nominal and factored bearing resistances by up to 50%!

6.10 SDDOT place more emphasis on settlement and deflection in shallow foundation design.

SDDOT place more emphasis on settlement and deflection in shallow foundation design.

SDDOT currently places almost all design emphasis on the development of factored bearing resistances for shallow foundations. However, shallow foundation performance in the field and in case histories is generally governed by Settlement or deflection of the foundation. SDDOT could place more emphasis on settlement estimates in their design work for shallow foundations.

7.0 RESEARCH BENEFITS

The research completed and the results of this research are interim. The review of SDDOT files, neighboring state practice, and interviews with Stakeholders has identified areas for improvement in implementation of LRFD but fell short of the ultimate goal of providing SDDOT a set of locally calibrated LRFD resistance factors for the unique geology and conditions within the State. This report sets forth the recommended tasks SDDOT should follow to enable the future local calibration of LRFD resistance factors. Unfortunately, this means that the primary benefits of this research will not be fully realized for some time. However, there are benefits to the recommendations of this research project that see immediate impact. The following enumerated list maps the recommendations of Section 6 to the short-term and long-term benefits to the State of South Dakota, its Department of Transportation, the taxpayers of the state, and the traveling public.

1. As SDDOT uses the AASHTO LRFD (2017) Bridge Design Specifications resistance factors of shallow foundation designs, as recommended, for the near future, new and ongoing design projects will benefit from increased reliability when compared with national trends of bridge and wall performance.
2. As SDDOT implements quantification of Intermediate Geomaterials (IGM) on projects, designers will benefit from increased utilization of nationally calibrated Resistance Factors for the challenging geologic conditions found within the State.
3. As SDDOT investigates the use of RMR field procedures to supplement borings for upcoming projects, these test methods will maximize the reliability of input parameters into design calculations, increasing the reliability and lowering risks of poor bridge or wall performance.
4. As SDDOT develops a centralized database of soil and rock parameters, wall and bridge performance, standardized calculation packages and well documented past performance driven designs, the institutional knowledge of highway structures designs will be able to be passed more effectively to the next generation(s) of design engineers. The centralized database will enable future LRFD resistance factor calibration. Without this database, no calibration will be possible.
5. As SDDOT performs a small set of full-scale foundation load tests, the documentation of past performance needed for current design methods and future LRFD calibration will be obtained. These data benefit the state in that now the levels of over- or under-conservatism in past-performance driven design and LRFD design can be constrained.
6. As SDDOT develops calibrated LRFD resistance factors local to the State, the costs associated with under- and -over conservatism will be ameliorated in bridge and wall foundation design and construction.
7. As SDDOT begins to use new and innovative shallow foundation options within the state, the ability of the designers and contractors to push forward Accelerated Bridge Construction within the state will be emboldened, allowing for decreased construction time on the urban and rural freeways in the state essential for commerce and community resilience.

8.0 REFERENCES

AASHTO (2002). "Standard Specifications for Highway Bridge, 17th Edition." American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2007). "AASHTO LRFD Bridge Design Specifications, 4th Edition." American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2012). "AASHTO LRFD Bridge Design Specifications, Customary U.S. Units (6th Edition)." American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

AASHTO (2014). "AASHTO LRFD Bridge Design Specifications, 7th Edition." American Association of State Highway and Transportation Officials. (AASHTO), Washington, D.C.

AASHTO (2017). "AASHTO LRFD Bridge Design Specifications, 8th Edition." American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.

Abu-Hejleh, N., DiMaggio, J. A., Kramer, W. M., Anderson, S., and Nichols, S. (2011). "Implementation of LRFD Geotechnical Design for Bridge Foundations." National Highway Institute, Washington, D.C.

Abu-Hejleh, N. M., Alzamora, D., Mohamed, K., Saad, T., and Anderson, S. (2014). "Implementation Guidance for Using Spread Footings on Soils to Support Highway Bridges ".

Alavi, A. H., and Sadrossadat, E. (2016). "New design equations for estimation of ultimate bearing capacity of shallow foundations resting on rock masses." *Geoscience Frontiers*, 7(1), 91-99.

Becker, D. E. (1996). "Eighteenth Canadian Geotechnical Colloquium: Limit States Design For Foundations. Part I. An overview of the foundation design process." *Canadian Geotechnical Journal*, 33(6), 956-983.

Bieniawski, Z.T. 1976. Rock mass classification in rock engineering. In *Exploration for rock engineering, proc. of the symp.*, (ed. Z.T. Bieniawski) 1, 97-106. Cape Town: Balkema.

Bieniawski, Z.T. 1989. *Engineering rock mass classifications*. New York: Wiley.

Bond, A. (2013). "Implementation and evolution of Eurocode 7." *Modern Geotechnical Design Codes of Practice: Implementation, Application and Development*, P. Arnold, ed.

Briaud, J.-L., and Gibbens, R. (1999). "Behavior of Five Large Spread Footings in Sand." *Journal of Geotechnical and Geoenvironmental Engineering*, 125(9), 787-796.

British Standards Institution (1986). *Code of Practice for Foundations*.

Canadian Geotechnical Society (2006). *Canadian Foundation Engineering Manual*.

Carter, J. P., and Kulhawy, F. H. (1988). "Analysis and design of drilled shaft foundations socketed into rock." Cornell University, Ithaca, NY.

Chang, N.-Y. (2006). "CDOT Foundation Design Practice and LRFD Strategic Plan." University of Colorado at Denver, Denver, CO.

Das, B. (2010). *Principles of Foundation Engineering, SI Edition*, Cengage Learning.

Davis, R.O., and Sevaldurai, A.P.S. (2002). *Plasticity and Geomechanics*, Cambridge Press, Cambridge, UK.

Densmore, D. H. (2000). "Load and Resistance Factor Design (LRFD) Memo." <https://www.fhwa.dot.gov/bridge/062800.cfm>. (01/08, 2015).

Dimillio, A. F. (1982). "Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill." Federal Highway Administration, Washington, D.C., 54.

Ditlevsen, O. (1981). *Uncertainty modeling with applications to multidimensional civil engineering systems*, McGraw-Hill International Book Co.

Ellingwood, B., Galambos, T. V., MacGregor, J. G., and Cornell, C. A. (1980). "Development of a probability based load criterion for American National Standard A58." National Bureau of Standards, 222.

European Union-European Commission (2016). "About the EU." <https://europa.eu/european-union/about-eu/eu-in-brief_en>. (06/22, 2015).

Fenton, G. A., and Griffiths, D. V. (2008). *Risk Assessment in Geotechnical Engineering*, Wiley.

Fenton, G. A., Griffiths, D. V., and Cavers, W. (2005). "Resistance factors for settlement design." *Canadian Geotechnical Journal*, 42(5), 1422-1436.

Fenton, G. A., Griffiths, D. V., and Ojomo, O. O. (2011). "Consequence factors in the ultimate limit state design of shallow foundations." *Canadian Geotechnical Journal*, 48(2), 265-279.

Fenton, G. A., Naghibi, F., Dundas, D., Bathurst, R. J., and Griffiths, D. V. (2015). "Reliability-based geotechnical design in 2014 Canadian Highway Bridge Design Code." *Canadian Geotechnical Journal*, 53(2), 236-251.

Fenton, G. A., Zhang, X., and Griffiths, D. V. "Load and resistance factor design of strip footings." *Proc., GeoCongress 2008: Geosustainability and Geohazard Mitigation, March 9, 2008 - March 12, 2008*, American Society of Civil Engineers, 106-113.

Federal Highway Administration, FHWA (2012). "Geosynthetic Reinforced Soil Integrated Bridge System interim implementation guide." *FHWA Publication No. FHWA-HRT-11-026*. Washington, D.C.

Federal Highway Administration, FHWA (2002). "Evaluation of soil and rock properties." *Geotechnical Engineering Circular No. 5, Report No. FHWA-IF-02-034*. Washington, D.C.

Forrest, W. S., and Orr, T. L. L. (2010). "Reliability of shallow foundations designed to Eurocode 7." *Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards*, 4(4), 186-207.

Foye, K. C., Salgado, R., and Scott, B. (2006a). "Assessment of Variable Uncertainties for Reliability-Based Design of Foundations." *Journal of Geotechnical and Geoenvironmental Engineering*, 132(9), 1197-1207.

Foye, K. C., Salgado, R., and Scott, B. (2006b). "Resistance Factors for Use in Shallow Foundation LRFD." *Journal of Geotechnical and Geoenvironmental Engineering*, 132(9), 1208-1218.

Gifford, D. G., Kraemer, S. R., Wheeler, J. R., and Mckown, A. F. (1987). "Spread footings for highway bridges." FHWA, U.S. Department of Transportation, Washington D.C., 229.

Goble, G. G. (1999). *Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods*, National Academy Press.

Goodman, R. E. (1989). *Introduction to Rock Mechanics*, Wiley.

Hasofer, A., and Lind, N. (1974). "An Exact and Invariant First-order Reliability Forma." *Journal of the Engineering Mechanics Division*, 100(EM1), 111-121.

Hubler, J.F., Zekkos, A., and Zekkos, D. (2018). "Monotonic and cyclic simple shear response of sand-gravel mixtures." *Soil Dynamics and Earthquake Engineering*, **115**, 291-304.

Kimmerling, R. E. (2002). *Geotechnical Engineering Circular No. 6: Shallow Foundations*, FHWA, Washington D.C.

Kulhawy, F. H., and Goodman, R. E. (1987). "Foundations in rock." *Ground Engineer's reference book*, F. G. Bell, ed., Butterworths, London, UK.

Kulhawy, F. H., and Phoon, K. K. "Observations on geotechnical reliability-based design development in North America." *Proc., Proceedings of International Workshop on Foundation Design Codes and Soil Investigation in View of International Harmonization and Performance Based Design*, 31-48.

Kulhawy, F. H., Trautmann, C. H., Beech, J. F., O'Rourke, T. D., McGuire, W., Wood, W. A., and Capano, C. (1983). "Transmission Line Structure Foundations for Uplift-Compression Loading." Electric Power Research Institute, Palo Alto, CA.

Lesny, K., and Paikowsky, S. G. (2011). "Developing a LRFD Procedure for Shallow Foundations." *Proceedings of the 3rd International Symposium on Geotechnical Safety and Risk*, N. Vogt, B. Schuppener, D. Straub, and G. Bräu, eds.Munich, Germany, 47-65.

Low, B. K., and Tang, W. H. (1997). "Probabilistic slope analysis using Janbu's generalized procedure of slices." *Computers and Geotechnics*, 21(2), 121-142.

Lwin, M. M. (1998). "Modern and Comprehensive Bridge Design Specifications." *Transportation Research News.* Washington D.C., 35-36.

Meyerhof, G. G. "The Ultimate Bearing Capacity of Foundations on Slopes." *Proc., Proceedings 4th International Conference on Soil Mechanics and Foundation Engineering*.

Meyerhof, G. G. (1963). "Some Recent Research on the Bearing Capacity of Foundations." *Canadian Geotechnical Journal*, 1(1), 16-26.

MODOT (2015). "Category:751.38 LRFD Bridge Design Guidelines - Spread Footings." *LRFD Bridge Design Guidelines*, <http://epg.modot.org/index.php?title=Category:751_LRFD_Bridge_Design_Guidelines>. (June 22, 2015).

Moulton, L. K., GangaRao, H. V. S., and Halvorsen, G. T. (1987). "Tolerable Movement Criteria for Highway Bridges." FHWA, U.S. Department of Transportation, Washington D.C.

Munfakh, G., Arman, A., Collin, J. G., Hung, J. C.-J., and Brouillette, R. P. (2001). "Shallow Foundations Reference Manual." FHWA, U.S. Department of Transportation,, Washington, DC.

Orr, T. L. L. (2012). "How Eurocode 7 has affected geotechnical design: a review." *Proceedings of the Institution of Civil Engineers - Geotechnical Engineering*, 165(6), 337-350.

Paikowsky, S. G., Canniff, M. C., Lesny, K., Kisse, A., Amatya, S., and Muganga, R. (2010). "LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures." *NCHRP Report 651*, Transportation Research Board, Washington, D.C.

Paikowsky, S. G., Fu, S., Amatya, Y., and Canniff, M. C. (2009). "Uncertainty in Shallow Foundations Settlement Analysis and Its Utilization in SLS Design Specifications." *Proc., Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering,* Alexandria, Egypt, 1317 - 1320.

Paikowsky, S. G., and Lu, Y. (2006). "Establishing Serviceability Limit State in the Design of Bridge Foundations." *Foundation Analysis and Design - Innovative Methods.* ASCE GSP No. 153 GeoShanghai International Conference, Shanghai, China, 49-58.

Pennsylvania Department of Transportation (2015). "Design Manual Part 4 Structures." Pennsylvania Department of Transportation, ed.

Phoon, K.-K., Kulhawy, F. H., and Grigoriu, M. D. (2003). "Multiple resistance factor design for shallow transmission line structure foundations." *Journal of Geotechnical and Geoenvironmental Engineering*, 129(9), 807-818.

Sabatini, P. J., Bachus, R. C., Mayne, P. W., Schneider, J. A., and Zettler, T. E. (2002). *Geotechnical Engineering Circular No. 5: Evaluation of Soil and Rock Properties* FHWA, Washington D.C.

Samtani, N. C., Nowatzki, E. A., Mertz, D. R., Anderson, S. A., and Federal Highway Administration Resource, C. (2010). *Selection of spread footings on soils to support highway bridge structures*, U.S. Dept. of Transportation, Federal Highway Administration Resource Center, Matteson, Ill.

Sargand, S. M., Masada, T., and Abdalla, B. (2003). "Evaluation of Cone Penetration Test-Based Settlement Prediction Methods for Shallow Foundations on Cohesionless Soils at Highway Bridge Construction Sites." *Journal of Geotechnical and Geoenvironmental Engineering*, 129(10), 900-908.

Schmertmann, J. H., Brown, P. R., and Hartman, J. P. (1978). "Improved Strain Influence Factor Diagrams." *Journal of the Geotechnical Engineering Division*, 104(GT8), 1131-1135.

Sowers, G. F., and Sowers, G. B. (1979). *Introductory soil mechanics and foundations: geotechnical engineering*, Macmillan.

Terzaghi, K. (1943). *Theoretical soil mechanics*, John Wiley and Sons, New York.

United States Department of the Navy Naval Facilities Engineering Command [NAVFAC] (1986). *Foundations and Earth Structures: Design Manual 7.2*, U.S. Navy, Naval Facilities Engineering Command, Alexandria, VA.

Wisconsin DOT (2016). "Bridge Manual." B. o. Structures, ed., Wisconsin DOT, Madison, WI.

Wyllie, D. C. (1992). *Foundations on Rock*, E & FN Spon, London, UK.

Xiao, M., Quit, T., Khosrojerdi, M., Basu, P., and Withiam, J.L. (2016). Synthesis and Evaluation of the Service Limit State of Engineered Fills for Bridge Support. *FHWA Report FHWA-HRT-15-080*. Federal Highway Administration, Washington, D.C.

Xue, J., and Nag, D. (2011). "Reliability analysis of shallow foundations subjected to varied inclined loads." *Geotechnical Safety and Risk*, N. Vogt, B. Schuppener, D. Straub, and G. Bräu, eds. Munich, Germany.

Appendix A: RMR Tables and Protocols from AASHTO (2007)

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses. AASHTO (2007)

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>8 MPa	4–8 MPa	2–4 MPa	1–2 MPa	For this low range, uniaxial compressive test		
		Uniaxial compressive strength	>200 MPa	100–200 MPa	50–100 MPa	25–50 MPa	10–25 MPa	3.5–10 MPa	1.0–3.5 MPa
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>3000 mm	900–3000 mm	300–900 mm	50–300 mm	<50 mm		
	Relative Rating		30	25	20	10	5		
4	Condition of joints		x Very rough surfaces x Not continuous x No separation x Hard joint wall rock	x Slightly rough surfaces x Separation <1.25 mm x Hard joint wall rock	x Slightly rough surfaces x Separation <1.25 mm x Soft joint wall rock	x Slicken-sided surfaces or x Gouge <5 mm thick or x Joints open 1.25–5 mm x Continuous joints	x Soft gouge >5 mm thick or x Joints open >5 mm x Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 10 000 mm tunnel length	None	<25 L/min.	25–125 L/min.	>125 L/min.			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations. AASHTO (2007)

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined from Total Ratings AASHTO (2007)

RMR Rating	100-81	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Table 10.4.6.4-4 Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining - Nonlinear Strength (*Hoek and Brown, 1988*) AASHTO (2007)

Rock Quality	Constants	Rock Type A = Carbonate rocks with well-developed crystal cleavage — *dolomite, limestone, and marble* B = Lithified argillaceous rocks—*mudstone, siltstone, shale, and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polyminerallic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite* E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 900–3000 mm *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 900–3000 mm *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 300—900 mm *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 50–300 mm; some gouge. Clean compacted waste rock. *RMR* = 23	*m* *s*	0.029	0.041	0.061	0.069	0.102

VERY POOR-QUALITY ROCK MASS Numerous heavily weathered joints spaced <50 mm with gouge. Waste rock with fines. *RMR* = 3	*m* *s*	0.007	0.010	0.015	0.017	0.025