SELECTION OF SPREAD FOOTINGS ON SOILS TO SUPPORT HIGHWAY BRIDGE STRUCTURES

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Global change

The following global change was made to make symbols (notations) consistent with those in *AASHTO LRFD* Bridge Design Specifications.

1. Use of symbol “S” in lieu of “d” when vertical movement (i.e., settlement) is discussed.

Specific changes

1. In Table of Contents, updated the title of Section 3.3 to make it consistent with the change noted in Item 4a below.
2. Updated List of Symbols and Abbreviations in the Table of Contents to reflect the change in symbols noted herein.
3. Section 3.1, page 14, first bullet item
   a. Modified paragraph to explain relationship between “Accuracy” and “Bias.” Included a new reference to Tan and Duncan (1991).
4. Section 3.3
   a. Included “THE S-0 CONCEPT OF” at the start of the title for this section.
   b. Modified Figure 3-2 based on the global change noted above.
   c. Modified Figure 3-3 to show Mode 1 and Mode 2 patterns.
   d. Deleted Table 3-1 and included it as part of Figure 3-3.
   e. Modified write-up related to Figure 3-2 and Figure 3-3.
   f. Renumbered Table 3-2 as Table 3-1 and made changes at all locations in the document where Table 3-2 was previously referenced.
5. Section 4.1
   a. Changed symbols in Figure 4-1, updated caption, and adjusted first paragraph to be consistent with revised notations.
   b. Modified Figure 4-3 to show Mode 1 and Mode 2 to be consistent with the modified Figure 3-3. Modified write-up in second full paragraph of page 23 to reflect modification of Figure 4-3.
   c. Repaginated other text in this section due to Item 5a.
6. Section 4.2
   a. In Item 4 on page 27, referenced information in Appendix C.3.3.3 to better define structural fill.
7. Appendix E
   a. Updated the limiting eccentricity criteria in Decision 2 box in Figure E.3-1, from “Is $e_{B\text{-}STR} < B_{f\text{-}SER}/4$?” to “Is $e_{B\text{-}STR} < B_{f\text{-}SER}/3$?” to reflect the latest *AASHTO LRFD* provisions.
   b. Made the above limiting eccentricity criteria update in Step 2 of the example problem in Section E.4.1.
   c. In Figure E.4-2, the symbol S_2 was used twice. Corrected the second occurrence to S_3.
8. Bibliography
   a. On page 33, corrected the title and report number for FHWA (1997) reference.
   b. On page 38, added reference of Tan and Duncan (1991) that was introduced in Item 3a.

FOREWORD

The FHWA believes that spread footings on soils are underutilized because designers encounter one or more of the following obstacles: (a) limited knowledge of AASHTO/FHWA technical references that pertain to spread footings on soils to support bridges; (b) limited knowledge of adequate performance data for spread footings; (c) unrealistic tolerable settlement criteria; (d) overestimation of loads used to calculate settlement; and (e) the use of conservative settlement prediction methods. These obstacles have resulted in institutional biases and overly conservative designs that lead to the unnecessary use of costlier deep foundation systems. The primary goal of this report is to promote the use of spread footings bearing on competent natural soils, improved soils, and engineered fill materials as a routine alternative to deep foundations for support of highway structures by addressing the factors identified above. The introduction of Load and Resistance Factor Design (LRFD) permits a rational approach to the consideration of spread footings on soils as a viable alternative to deep foundations. Using documented performance data, the report presents powerful concepts such as the use of construction-point analysis and angular distortions to demonstrate the efficacy of using spread footings. The report presents sources of information that agencies and designers can use as references in their project applications. It is hoped that this report will provide the impetus for agencies and transportation designers to evaluate the use of spread footings for all conditions other than those where their use is precluded (e.g., scour conditions).

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Notice

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16. Abstract

The FHWA believes that spread footings on soils are underutilized because designers encounter one or more of the following obstacles: (a) limited knowledge of AASHTO/FHWA technical references that pertain to spread footings on soils to support bridges; (b) limited knowledge of adequate performance data for spread footings; (c) unrealistic tolerable settlement criteria; (d) overestimation of loads used to calculate settlement; and (e) the use of conservative settlement prediction methods. These obstacles have resulted in institutional biases and overly conservative and excessively costly institutional processes that lead to the unnecessary use of costlier deep foundation systems.

The primary goal of this report is to promote the use of spread footings bearing on competent natural soils, improved soils, and engineered fill materials as a routine alternative to deep foundations for support of bridges by addressing the factors identified above. Documented performance data is used to make this case. The report also presents powerful concepts such as construction-point analysis and angular distortions to demonstrate the efficacy of using spread footings. Implementation of these concepts requires only that conventional computations be taken one step further without any requirement for advanced computational skills. The report presents sources of information that agencies and designers can use as references in their project applications. The report contains comprehensive appendices that treat in detail many of the topics discussed in the report. For example, one such appendix provides an introduction to Load and Resistance Factor Design (LRFD) that permits a rational approach to the consideration of spread footings on soils as a feasible alternative to deep foundations.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	Lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	Feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASD	Allowable Stress Design
ASTM	American Society for Testing and Materials
A_1	Calculated angular distortion in Span 1
A_2	Calculated angular distortion in Span 2
A_3	Calculated angular distortion in Span 3
A_4	Calculated angular distortion in Span 4
A'_f	Effective footing area
B	Direction of lesser plan dimension of footing
B_f	Total footing width
B'_f	Effective footing width
CPT	Cone Penetration Test
DA_{1-1}	Design angular distortion in Span 1 for Mode 1
DA_{1-2}	Design angular distortion in Span 1 for Mode 2
DA_{2-1}	Design angular distortion in Span 2 for Mode 1
DA_{2-2}	Design angular distortion in Span 2 for Mode 2
DA_{3-1}	Design angular distortion in Span 3 for Mode 1
DA_{3-2}	Design angular distortion in Span 3 for Mode 2
DA_{4-1}	Design angular distortion in Span 4 for Mode 1
DA_{4-2}	Design angular distortion in Span 4 for Mode 2
D_f	Depth of footing base below finished grade
DMT	Dilatometer Test
DOSI	Depth of Significant Influence
DOT	Department of Transportation
DS	Design differential settlement within a given span
DS_{1-1}	Design differential settlement in Span 1 for Mode 1
DS_{1-2}	Design differential settlement in Span 1 for Mode 2
DS_{2-1}	Design differential settlement in Span 2 for Mode 1
DS_{2-2}	Design differential settlement in Span 2 for Mode 2
DS_{3-1}	Design differential settlement in Span 3 for Mode 1

DS_{3-2}	Design differential settlement in Span 3 for Mode 2
DS_{4-1}	Design differential settlement in Span 4 for Mode 1
DS_{4-2}	Design differential settlement in Span 4 for Mode 2
e	Eccentricity
e_B	Eccentricity in B-direction
e_L	Eccentricity in L-direction
F	Factored Load (Strength Limit State)
FHWA	Federal Highway Administration
ft	foot (feet)
ft^2	square feet
GS	Geotechnical Specialist
in	inches
ksf	kips per square foot
kPa	kilo Pascal
L (in context)	Distance between 2 points over which the angular distortion is calculated
L (in context)	Direction of greater plan dimension of footing
L_f	Total footing length
L_1	Length of Span 1
L_2	Length of Span 2
L_3	Length of Span 3
L_4	Length of Span 4
LRFD	Load and Resistance Factor Design
M	Moment
MCFT	Modified Compression Field Theory
M_{SER}	Moment at Service Limit State I
M_{STR}	Moment at Strength Limit State I (max)
m	meter
m^2	square meters
MPa	Mega Pascal

MSE	Mechanically Stabilized Earth
NA	Not Available
NCHRP	National Cooperative Highway Research Program
NDOT	Nevada Department of Transportation
NIST	National Institute of Standards and Technology
N	SPT N-value (uncorrected)
N_{60}	SPT N-value corrected for hammer efficiency
N_{corr}	SPT N-value corrected for overburden pressure
$N1_{60}$	SPT N-value corrected for overburden pressure and hammer efficiency
OCR	Over Consolidation Ratio
ODOT	Ohio Department of Transportation
q_{max}	Maximum contact pressure
q_{nn}	Net nominal bearing resistance
q_{teuv}	Net equivalent uniform vertical bearing resistance
q_R	Factored net bearing resistance
q_{teuv}	Total equivalent uniform vertical bearing resistance
Q	Estimated dead load
Q_I	Load at initial limit
Q_F	Load at factored limit
Q_S	Load at service limit
Q_N	Load at nominal limit
S (in context)	Settlement
S (in context)	Service Load (Service Limit State)
S_1	Calculated settlement at construction-point 1
S_2	Calculated settlement at construction-point 2
S_3	Calculated settlement at construction-point 3
S_4	Calculated settlement at construction-point 4
S_{A1}	Total settlement at Abutment 1
S_{A2}	Total settlement at Abutment 2
S_I	Settlement at initial limit
S_F	Settlement at factored limit
S_m	Measured settlement
S_N	Settlement at nominal limit

S_{P1}	Total settlement at Pier 1
S_{P2}	Total settlement at Pier 2
S_{P3}	Total settlement at Pier 3
S_S	Settlement at service limit
S_{tol}	Tolerable Settlement
S_W	Settlement under load W
S_X	Settlement under load X
S_Y	Settlement under load Y
S_Z	Settlement under load Z
SE	Load factor for SE Load type in AASHTO LRFD
SPT	Standard Penetration Test
SS	Structural Specialist
TRB	Transportation Research Board
US	United States
V	Vertical component of the resultant load
V_{SER}	Vertical component of the resultant load at Service Limit State I
V_{STR}	Vertical component of the resultant load at Strength Limit State I (max)
W	Load after footing construction
WSDOT	Washington State Department of Transportation
X	Load after pier column/wall construction
Y	Load after superstructure construction
Z	Load after wearing surface construction
γ_p	Load factor for permanent earth load
γ_s	Unit weight of soil within the depth of footing embedment, D_f
δ	Calculated foundation movement
ϕ_b	Resistance factor for bearing

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CHAPTER 1 – INTRODUCTION

In general terms, foundations for bridges and their appurtenant structures such as retaining walls can be categorized as shallow or deep. The duty of a foundation design specialist is to establish the most economical design that safely satisfies prescribed structural criteria and properly accounts for the intended function of the structure. However, in reality the foundation specialist often chooses a deep foundation system because of a variety of reasons that include institutional culture and biases and/or lack of knowledge about rational methods for analysis and design.

Based on the research of Department of Transportation (DOT) practices, it appears that the primary obstacles to the wider use of shallow foundations on soils for highway bridges are:

1. Limited knowledge of documents from the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) that provide guidelines for the selection, and safe and economical design and construction of shallow foundations on soils to support bridges.
2. Limited knowledge of adequate performance information of spread footings on soils from (a) documented results of axial load tests, and (b) case histories of the successful and economical use of spread footings on soils to support bridges.
3. Use of conservative settlement prediction methods.
4. Restrictive tolerable settlement criteria, which have led some agencies to set unrealistic performance criteria for foundations (e.g., zero or ¼ inch of settlement).
5. Overestimation of loads that generate settlement.
6. Institutional culture and biases; for example, “It’s the way we have always done it.”

The cumulative effect of these obstacles often results in the indiscriminate use of costlier and more complex deep foundations. FHWA (1997) estimated a potential cost savings to the public of approximately $135 million, if one assumes that only 25% of the approximately 600,000 existing bridges in the United States were to use spread footings when they are replaced due to age or other reasons. If that amount is adjusted for inflation, the potential cost savings in terms of 2009 dollars would be more than $180 million. Because of this significant cost saving the FHWA is making efforts to ensure that spread footings are considered as a viable foundation alternative for all bridges.

The introduction of the Load and Resistance Factor Design (LRFD) methodology permits a rational approach to the consideration of spread footings on soils as a viable alternative to deep foundations. Using documented performance data, this report presents powerful concepts, such as the use of construction-point analysis and angular distortions, to demonstrate the benefits of using spread footings. Implementation of these concepts requires only that conventional computations be taken one step further without any requirement for advanced computational skills. This report also presents sources of information that agencies and designers can use as references in their project applications. The hope is that this report will provide the impetus for agencies to evaluate the use of cost-effective spread footings for all conditions other than those where their use may be suspect or is clearly precluded (e.g., scour conditions).

1.1 GOAL

The primary goal of this publication is to promote the mainstream use of spread footings on soils as a routine foundation alternative to deep foundations for the support of bridges and other highway structures. To attain this goal, the following chapters will demonstrate that spread footings may be an excellent foundation alternative for fast and low cost, low risk construction on competent native soils, improved in situ soils, and compacted engineered embankment fill materials. Spread footings are not always appropriate, and this publication can help one identify those situations as well.

1.2 SCOPE OF THE WORK

The scope of the work for the present study to attain the stated goal is as follows:

- Provide performance data that demonstrate the successful use of spread footings,
- Provide guidance for calculating settlements based on an appropriate loading sequence,
- Provide references to resources that can be used to evaluate spread footings and overcome the factors listed above, and
- Recommend mechanisms to analyze and design spread footings in the context of the LRFD methodology.

The authors assume that the reader has a basic understanding and knowledge of the analysis and design of shallow foundation systems as well as the LRFD approach. The reader is referred to the bibliography chapter that contains an extensive list of references for additional information.

1.3 PAST STUDIES AND THE CURRENT STUDY

The topic of shallow foundations is not new and neither is the subject matter of this study. Several studies have been sponsored by the FHWA and other agencies such as the Transportation Research Board (TRB) through their National Cooperative Highway Research Program (NCHRP), the Departments of Transportation (DOT) of various states (e.g., Ohio, South Carolina), the National Institute of Standards and Technology (NIST), and the American Society of Civil Engineers (ASCE). Internationally, there are other similar studies. A list of the studies relevant to highway bridge structures is provided in the Bibliography of this report, which is organized according to primary and secondary sources. Primary sources include publications of the FHWA, TRB, NCHRP, AASHTO, ASTM, NIST, DOTs and various university transportation research organizations. Secondary sources include articles published in peer-reviewed technical journals or in conference proceedings.

This effort differs from past studies in that it is phrased in the context of the Load and Resistance Factor Design (LRFD) approach, which is now mandatory for the design of highway bridge structures that are built with Federal funds. The full benefit of the LRFD methodology will be realized in time through achieving better performance with less overall resources by 'balancing' the design for desired reliability and by improving communication between design specialists working on different parts of the design. Part of the communication improvement will come through the use of a common language; one that relies on defined limit states and criteria for performance. As practice moves towards this goal, now is the time to reevaluate the performance of spread footings on soils and the process of foundation selection and design for highway bridge structures. This study provides guidance in an LRFD language common to both geotechnical and structural specialists.

1.4 AVAILABLE RESOURCES

Limited knowledge of available resources was identified earlier as the first obstacle to greater use of spread footings on soils. The FHWA has a reference manual (FHWA, 2001), a Geotechnical Engineering Circular (FHWA, 2002), and several research reports and training courses that provide guidance on the appropriate use of spread footings on soils. The primary resources are listed in the bibliography. However, most of these were developed based on the formerly used Allowable Stress Design (ASD) methodology. The NHI LRFD Substructures Course and Reference Manual (FHWA, 2005), as well as the manual for mechanically stabilized earth (MSE) walls (FHWA, 2009) provide guidance for shallow foundations in the context of the LRFD methodology. The AASHTO LRFD specification (AASHTO, 2007 with 2009 Interims) itself provides valuable commentary on the use of spread footings on soils. Appendix A describes other resources that are also available from FHWA. In this report, comprehensive guidance for the use of the LRFD methodology is included in Appendix E, which concentrates on the evaluation of spread footings in the LRFD context. The Appendix pulls together previous guidance and is intended to mitigate the obstacle of limited knowledge of available resources.

1.5 SUMMARY

Chapter 1 identifies six obstacles that may be impediments to the wider use of spread footings on soil for highway bridge structures. The first obstacle pertains to the availability of FHWA and other resources to designers. Section 1.4 of this chapter and Appendix A provide information in this regard. A number of the FHWA publications currently available are listed in the Bibliography. Thus, the first obstacle is addressed explicitly in this chapter. Similarly, the remaining five obstacles will be systematically addressed in the following chapters.

CHAPTER 2 - PERFORMANCE

Limited understanding of documented performance is the second obstacle listed in Chapter 1 as an impediment to wider use of spread footings. Performance is documented here through reported experience from state DOTs, documented data, and cost considerations. As shown in this chapter, spread footing performance for highway bridges is determined by deformation criteria, not soil bearing failure. Furthermore, spread footings have generally been less expensive than other foundations, and the bridges founded on them have performed very well.

2.1 REPORTED EXPERIENCE OF STATE DOTs

As part of this study two independent surveys were performed to obtain a broad view of the extent to which state DOTs used spread footings in the past for highway bridge foundations and the extent of their use in current practices related to pier and abutment foundations. Abu-Hejleh (2009) surveyed predominantly geotechnical specialists while Mertz (2009) concentrated on bridge specialists. A combined total of more than 40 states participated in the surveys. Following are the key findings based on these surveys:

- The results of both surveys were remarkably similar and revealed that, for the most part, structural specialists currently use spread footings for bridge substructures only at non-stream crossings and when the substructures can be founded on bedrock or intermediate geomaterials (IGMs), i.e., geomaterials whose engineering properties are intermediate between soil and rock.
- Use of spread footings varied significantly between states with some states reporting that they "never" use spread footings.
- Even if only anecdotally, it was found that bridges on spread footings were performing as well as those on deep foundations.
- None of the respondents reported significant problems with bridges founded on spread footings when the footings were technically viable and designed and constructed properly.
- Many state DOTs have started migrating towards the use of integral or semi-integral abutments founded on driven piles.
- Spread footings were reported to be used in a variety of configurations and for conditions ranging from fills, granular soils, overconsolidated clays, on top of MSE walls (i.e., as "true" bridge abutments) and, to a lesser degree, for integral abutments.

2.2 DOCUMENTED PERFORMANCE OF BRIDGES

Significant documentation for the successful performance of bridges is available. The Bibliography provides a list of references where such information can be found. This section presents some pertinent findings from previous reports on spread footing use for highway bridge structures. The key point is that highway bridges can tolerate significant deformations and that spread footings have been used successfully for highway bridge structures.

- FHWA (1985) presents the results of work that was based on a nationwide study of 314 bridges and arrives at the following conclusions:

  “The results of this study have shown that, depending on type of spans, length and stiffness of spans, and the type of construction material, many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without becoming seriously overstressed, sustaining serious structural damage, or suffering impaired riding quality. In particular, it was found that a longitudinal angular distortion (differential settlement/span length) of 0.004 would most likely be tolerable for continuous bridges of both steel and concrete, while a value of angular distortion of 0.005 would be a more suitable limit for simply supported bridges."

- NCHRP (1983) concluded the following:

  "In summary, it is very clear that the tolerable settlement criteria currently used by most transportation agencies are extremely conservative and are needlessly restricting the use of spread footings for bridge foundations on many soils. Angular distortions of 1/250 of the span length and differential vertical movements of 2 to 4 inches (50 to 100 mm), depending on span length, appear to be acceptable, assuming that approach slabs or other provisions are made to minimize the effects of any differential movements between abutments and approach embankments.”

- FHWA (1982) and the Washington State Department of Transportation conducted a performance evaluation of 148 highway bridges supported by spread footings on engineered fills. This review, in conjunction with detailed survey investigations of the foundation movement of 28 selected bridges, concluded that spread footings can provide a satisfactory alternative to deep foundations, especially when high embankments of good quality borrow materials are constructed over satisfactory foundation soils. Studies of foundation movement showed that bridges easily tolerated differential settlements of 1 to 3 inches without significant distress.

In addition to the above studies, there are numerous instances of the successful use of spread footings on top of MSE walls at abutments (so-called “true bridge abutments”) with resultant cost savings. For example, true bridge abutments were used on 30 bridges as part of the reconstruction of the “BIG-I” traffic interchange at the intersection of I-25 and I-40 in Albuquerque, New Mexico. In service since 2000, these bridges are performing well. In Colorado, true bridge semi-integral abutments were constructed at the Founders/Meadows Bridge over I-25. Geogrids were used for soil reinforcements in these structures, which were extensively instrumented (Abu-Hejleh et al., 2001). Another successful application is in Tucson, Arizona, where use of true bridge abutments for a 5-span bridge over a local parkway and multiple tracks of the Union Pacific Rail Road resulted in significant cost savings for the owner. In service since 2004, these abutments are performing well based on the results of monitoring over 500 survey markers placed on the MSE abutments.

Unfortunately, many agencies continue to disregard spread footings as a viable foundation alternative to deep foundations for highway structures. Section 2.3 presents data in a format that demonstrates the basic behavior of spread footings leading to their successful performance as foundation systems for highway bridge structures.

2.3 CHARACTERISTICS OF LOAD-DEFORMATION PERFORMANCE

The reported excellent performance of spread footings for highway bridge structures can be understood better by looking at instrumented test cases and the characteristics of measured load-deformation behavior for spread footings. Note that due to the large size of spread footings for highway bridges, soil bearing failure is not likely (FHWA, 2006). Therefore, the performance of spread footings in highway bridge design is evaluated primarily on the basis of vertical displacement, i.e., settlement, and how differential settlements affect angular distortion. Figure 2-1 shows the schematic of a typical axial load (Q) versus vertical displacement (S) curve for a spread footing of width B_f. The Q-S curve contains three distinct parts as follows:

- Initial (Part O-I)
- Transition (Part I-N)
- Nominal (Beyond N)

The Q-S curve is linear within the initial and nominal parts, while in the transition part it is non-linear as shown in Figure 2-1. Point I is often called the elastic limit because the curve is linear up to that point. However, soils very rarely act elastically upon initial loading even though their Q-S curve may be linear. Therefore, in this report, Point I will be referred to as the initial limit. Point N corresponds to the nominal resistance that is based on the shear resistance of the soil on which the footing of width B_f is bearing. In conventional terms, this point corresponds to the ultimate bearing capacity, which in LRFD is referred to as the nominal bearing resistance.

Based on an evaluation of 167 axial compression field load tests on spread footings of widths (B_f) ranging from approximately 0.8-ft to 13-ft, Akbas and Kulhawy (2009) report the values of vertical displacement, δ, in terms of footing width, B_f, at Points I and N as shown in Tables 2-1 and 2-2. Table 2-3 presents the ratio of loads, Q_N/Q_I, at Points I and N based on the same work. All the data in Tables 2-1 to 2-3 were developed by Akbas and Kulhawy (2009) based on the L_1-L_2 method of analysis of a load settlement curve, which is described in their paper.

Following are the notable points from the information in Tables 2-1 and 2-2:

- The mean value of settlement at the initial limit (Point I), S_I, in Figure 2-1 is 0.26% B_f
- The mean value settlement at the nominal limit (Point N), S_N, in Figure 2-1 is 5.88% B_f
- The mean value of Q_N/Q_I is 6.74, i.e., the load at nominal limit (Point N in Figure 2-1) is 6.74 times the load at initial limit (Point I in Figure 2-1)

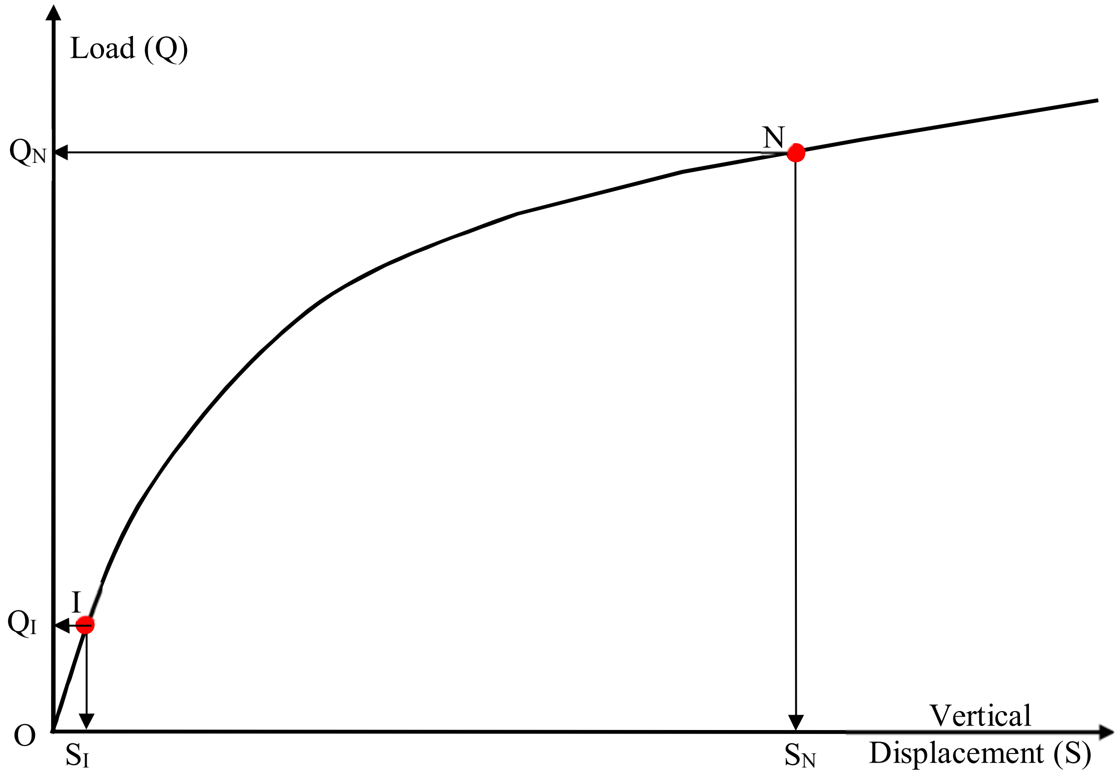


Figure 2-1. Schematic. Typical axial load versus vertical displacement curve (Q-δ) for spread footing of width B_f.

Table 2-1. Vertical displacement values for points I and N in Figure 2-1 (After Akbas and Kulhawy, 2009).

Point	S/B_f (%)			Coefficient of Variation, COV (%)
	Maximum (Upper Bound)	Minimum (Lower Bound)	Mean	
At Point I, S_I	1.33[0.34]	0.05[0.12]	0.26[0.23]	71[23]
At Point N, S_N	14.9[6.63]	3.30[3.99]	5.88[5.39]	26[13]
Values in [ ] parenthesis are for uncemented transported soils. Other values include data from residual, cemented as well as uncemented soils.				

Table 2-2. Ratio of loads for points I and N in Figure 2-1 (After Akbas and Kulhawy, 2009).

Axial Load, Q	Q_N/Q_I
Maximum	14.3 [10.3]
Minimum	1.70 [3.67]
Mean	6.74 [6.88]
Coefficient of Variation, COV (%)	38.0 [24.0]
Values in [ ] parenthesis are for uncemented transported soils. Other values include data from residual, cemented as well as uncemented soils.	

Table 2-3 presents calculated values of S/B_f for common ranges of sizes of spread footings and a range of settlements. By comparing the values in Table 2-3 with the mean values in Table 2-1, it can be observed that on the load-settlement curve shown in Figure 2-1, most of the footing designs would be performed in the vicinity of Point I. This clearly shows the extreme conservatism in typical spread footing designs for highway bridges.

Table 2-3. Range of typical S/B_f for spread footings on soils for highway bridges.

Footing Width, B_f (ft)	S/B_f (%)				
	S = 0.50-in	S = 0.75-in	S = 1.00-in	S = 1.5-in	S = 2.00-in
10	0.42	0.63	0.83	1.25	1.67
15	0.28	0.42	0.56	0.83	1.11
20	0.21	0.31	0.42	0.63	0.83
25	0.17	0.25	0.33	0.50	0.67
30	0.14	0.21	0.28	0.42	0.56
35	0.12	0.18	0.24	0.36	0.48

The geotechnical literature contains a large amount of field performance data for spread footings for constructed facilities. However, much of the data are related to smaller size footings for buildings. As part of the development of this report, an attempt was made by the project team to identify case histories of large-scale field load tests and/or instrumented footings in actual highway bridges. Appendix B provides a summary of case histories that were identified in the open literature (English language) that contains data related to large spread footings typical of highway bridges. It is acknowledged that there are other data bases that are not available in the public domain or in languages other than English that are not included in Appendix B. However, the information presented in Appendix B is sufficient to develop the following general observations:

- The typical contact pressure under a highway bridge footing varies from 3 to 6 ksf.
- When expressed in terms of ratio of measured settlement, S_m, to total footing width, B_f, the S_m/B_f values range from 0.1% to 1.0% for footing widths greater than 10-ft. For smaller footing widths values of S_m/B_f as large as 2.50% can occur.

Comparing this range of observed S_m/B_f values with those cited in Table 2-1 for δ/B_f, it can seen that in the context of Figure 2-1 the large spread footings for highway bridges are generally designed at or near Point I or the initial limit. This observation was also made with respect to the information in Table 2-3.

Based on the information in this chapter, it is clear that spread footings on soils for highway bridges perform well with respect to settlement and have a large margin of safety with respect to soil strength since the loads are well below the nominal resistance (Point N in Figure 2-1). However, this does not mean that larger settlements will be acceptable from the viewpoint of structural integrity of the bridge superstructure or aesthetics. The issue of settlements with respect to their effects on bridge superstructure will be further evaluated in later chapters.

2.4 COST CONSIDERATIONS

Except for the cases discussed in Section 2.4.1, it is generally accepted that spread footings are more cost-effective than deep foundations. There are many reasons for this but perhaps the most pertinent reasons are that the construction of spread footings does not require specialty contractors or equipment and that all the construction is performed at shallow depths.

Even though spread footings are more cost-effective, there are not many published detailed cost analyses documenting the cost savings associated with the spread footing alternative. Following are some observations from three significant formal cost analyses:

- FHWA (1982) presents 3 case histories of bridges constructed by Washington State DOT (WSDOT) which demonstrate that the spread footing alternative was 46% to 67% less expensive compared to the deep foundation alternative.
- FHWA (1987) indicates that for bridge abutment and pier loads of less than 1,000 tons (2,000 kips), which are typical of highway overpass structures, the cost of pile foundations in 1986 dollars is on the order of $40 to $80 per ton of pier load depending on the pile length, as compared with a spread footing cost on the order of $15 per ton. For heavier loads the cost of the pile foundation is likely to be on the order of 2 to 3 times that of spread footings.
- Based on their work for Ohio DOT (ODOT), Sargand and Masada (2006) report that cost savings of 58% can be realized by ODOT by using spread footings instead of pile foundations.

From these studies, it is evident that spread footings, where technically feasible, can provide significant cost-savings. Therefore, it is recommended that for all projects where spread footings are feasible, i.e., where unfavorable situations as identified in this report do not exist, owners perform a detailed cost analyses to justify not using spread footings. Following are guidelines for performing the cost analyses so that costs are not biased towards use of deep foundations (FHWA, 2006):

- Express the total cost of a foundation system in terms of dollars per kip (or ton) of load that will be supported; this is the approach used in the FHWA (1987) study cited above.
- For an equitable comparison, the total foundation cost should include all costs associated with a given foundation system including the need for excavation or retention systems, environmental restrictions on construction activities, e.g., vibrations, noise, disposal of contaminated spoils, pile caps and cap size, etc.
- For major projects, if the estimated costs of alternative foundation systems during the design stage are within 15 percent of each other, the alternate foundation designs should be considered for inclusion in the contract documents. If alternate designs are included in the contract documents, both designs should be adequately detailed. Otherwise, material costs and not the installed foundation cost will likely determine the low bid.

As noted in Chapter 1, FHWA (1997) estimated a potential cost savings to the public of approximately $135 million, if one assumes that only 25% of the approximately 600,000 existing bridges in the United States were to use spread footings when they are replaced due to age or other reasons. If that amount is adjusted for inflation, the potential cost savings in terms of 2009 dollars would be more than $180 million. This is likely a lower bound estimate of the real cost-savings.

2.4.1 Use of Deep Foundations where Spread Footings are Technically Viable

As part of the surveys reported in Section 2.1, it was found that some states such as Texas and Colorado often use drilled shafts even though spread footings are technically feasible. Such decisions are made based on local conditions such as significant competition among drilled shaft contractors and the generally favorable soil and groundwater conditions for open-hole shaft construction. In these situations, drilled shafts may be cost-competitive and could be considered feasible. Site constraints and construction scheduling are other considerations that could favor the use of deep foundations where spread footings would be otherwise technically feasible. For example, on the I-10/I-19 traffic interchange in Tucson, AZ, spread footings were viable, but drilled shafts were used because the large open excavations for spread footings in close proximity to each other created site constraints from a construction viewpoint.

2.5 SUMMARY

Chapter 2 provides information to mitigate the second obstacle to the wider use of spread footings on soil for highway bridge structures. As noted in Chapter 1, the second obstacle is "Limited knowledge of adequate performance information of spread footings on soils from (a) documented results of axial load tests, and (b) case histories of the successful and economical use of spread footings on soils to support bridges." There is ample evidence of successful use and performance of spread footings on soils in highway bridge design practice. The key points in this regard are as follows:

(a) Most spread footing designs are very economical and very conservative in that the design load is far less than the nominal resistance, and
(b) Spread footings perform as well as deep foundations for conditions where both are technically feasible.

CHAPTER 3 – EVALUATION OF TOLERABLE SETTLEMENTS FOR HIGHWAY BRIDGE DESIGN

The controlling factor in the design of spread footings for bridges is usually tolerable settlement. Use of conservative methods for the estimation of settlement was identified in Chapter 1 as the third obstacle to the wider use of spread footings on soils for highway bridge structures. In addition, the overly restrictive and unrealistic tolerable settlement criteria often used by agencies, e.g., zero or ¼ inch of settlement was identified in Chapter 1 as the fourth obstacle to the wider use of spread footings on soil for highway bridge structures. To help mitigate these two obstacles, Chapter 3 discusses settlement prediction methods and the settlement criteria that should be used in the context of bridge design.

3.1 SETTLEMENT PREDICTION METHODS

The settlement of a footing is the result of the compression of soils within the "depth of significant influence" (DOSI), i.e., the depth below the base of the footing to which significant applied stresses are experienced by the soil. The DOSI varies from 2 to 4 times the effective width of the footing depending on the shape of the footing. Appendix C contains further discussion of the DOSI. The total settlement under a load is comprised of two major components: (a) immediate (so-called elastic) settlements, and (b) long-term time-dependent consolidation (primary and secondary) type of settlements. While the methods to determine long-term consolidation type settlements are fairly standard (FHWA, 2006), there are a number of different settlement prediction methods available for computing immediate settlement. Each analytical method used for calculating immediate settlements is based on a set of assumptions specific to that method. Therefore, there is an inherent uncertainty associated with the calculated values of movements. Several studies have been undertaken by various researchers that have compared measured settlements to calculated settlements, e.g., Burland and Burbridge (1984), FHWA (1987), Baus (1992), FHWA (1997), Sargand and Masada (2006), etc. Figure 3-1 shows the data from the study performed by FHWA (1987). Other studies show similar trends.

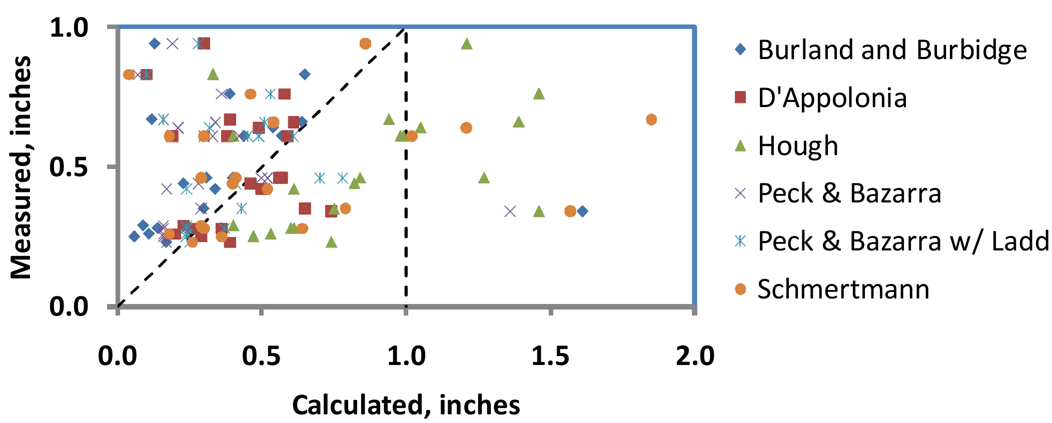


Figure 3-1. Graph. Comparison of measured and calculated settlements based on data in FHWA (1987).

The key points to be noted from the data in Figure 3-1 are:

- The more accurate the method, the closer the data points will plot to the 1:1 diagonal line. Two definitions, **Accuracy** (also sometimes referred to as **Settlement Ratio**) and **Bias**, can be used to evaluate different methods. **Accuracy**, which is often used in the geotechnical literature, is defined by Tan and Duncan (1991) as the value of the ratio of calculated to measured settlements. **Bias**, which is often used in LRFD context, is defined as the value of the ratio of the measured to the calculated settlements. Thus, Bias is the inverse of Accuracy and in either case the closer the value of these ratios to 1, the more accurate the calculation method. The average (arithmetic mean) values of accuracy and bias data are defined as Accuracy Factor and Bias Factor, respectively. Since both the definitions are with respect to the average of the data points, the method which has the value of these factors closest to 1.0 may result in calculated values being more or less than the actual (measured) value.

- Some methods are more reliable than others. Reliability should not be confused with accuracy. Reliability can be considered as the probability that the measured (actual) settlements would be <u>less than</u> those calculated by a specific method. It is a measure of conservativeness of a settlement prediction method. With respect to Figure 3-1, the method that yields the most data points below the 1:1 diagonal line is the most reliable, but not necessarily the most accurate.

From the above discussion, it can be surmised that a method that is accurate is not necessarily as reliable as the method that is less accurate. For example, a method with a an Accuracy Factor of 2.0 (or alternatively a Bias Factor of 0.5) is more reliable than that with an Accuracy Factor or Bias Factor of 1.0 because it would give a conservative estimate of settlement, i.e., the measured value would be half the calculated value as opposed to being equal to the calculated value.

3.2 FHWA RECOMMENDED METHODS FOR CALCULATING SETTLEMENTS

While accuracy and reliability considerations are important, the choice of a method should also be based on whether or not it offers a rational evaluation of stress and strain distribution with depth for various footing shapes and the elastic properties of the foundation soils. Based on all of these considerations, FHWA (2006) recommends the use of the method proposed by Schmertmann et al. (1978) for computing immediate settlements. Since AASHTO (2007 with 2009 Interims) recommends the use of specific correlations to estimate the elastic properties of the soils, it is recommended that the version of the method as presented in FHWA (2006), which is calibrated to the AASHTO correlations, be used for calculating immediate settlement. Use of the FHWA modified version of the method of Schmertmann et al. (1978) is illustrated in FHWA (2006).

If long-term, time-dependent consolidation settlements are anticipated, then FHWA recommends the use of the conventional consolidation theory to estimate the long-term settlements. These long-term settlements would have to be added to the immediate settlements to obtain total settlements. Procedures for computing the long-term consolidation type of settlements can be found in FHWA (2006) or other well-known geotechnical textbooks, such as Perloff and Baron (1976), Lambe and Whitman (1979), Holtz and Kovacs (1981) and Terzaghi et al. (1996).

It is important to note that it is simply not enough that the designer is using the FHWA recommended methods for computing settlements. Incumbent in this process is the need to perform a minimum standard of investigations as outlined in FHWA (2006) and Section 10 of AASHTO (2007 with 2009 Interims) to develop the appropriate geotechnical parameters for use in settlement analyses. In the context of LRFD, AASHTO allows the use of larger resistance factors if better investigation techniques, such as cone penetration tests or plate load tests, are used. For critical highway bridges on life-line routes (e.g., interstate highways), consideration may be given to increasing the level of field and laboratory investigations particularly in those areas where prior information and/or experience is poor or not available.

3.3 THE S-0 CONCEPT OF EVALUATION OF CALCULATED SETTLEMENTS IN THE CONTEXT OF BRIDGE DESIGN

Although calculation of total settlement is an important and necessary exercise, it is only the first step in evaluating the feasibility of using spread footings on soils for bridge foundations. Whether the calculated settlements will be detrimental to a bridge structure will depend on the configuration of the bridge and the materials used to construct the bridge, e.g., steel or concrete. This is because differential settlements and angular distortions are more pertinent parameters in the evaluation of damage to bridge facilities than the absolute settlements alone. In a larger context, the issues of accuracy and reliability as well as the use of a specific method of calculating settlements are mitigated when settlements are expressed in terms of differential settlements and angular distortions. Using the concepts introduced in earlier chapters, the published performance data, and knowledge of the general behavior of bridges, this section and the remainder of this report will demonstrate that the use of spread footings on soils will often lead to an acceptable behavior of bridge structures in terms of deformations.

While all methods have a certain degree of uncertainty in estimating settlements, the uncertainty of the calculated differential settlement is larger than the uncertainty of the calculated total settlement at each of the two support elements used to calculate the differential settlement, e.g., between an abutment and a pier, or between two adjacent piers. For example, if one support element actually settles less than the amount calculated while the other support element actually settles the amount calculated, the actual differential settlement will be larger than the difference between the two values of calculated settlement at the support elements.

Based on the above considerations and guidance in FHWA (2006) and NCHRP (1991), the following criteria are suggested to estimate a realistic value of differential settlement and angular distortion:

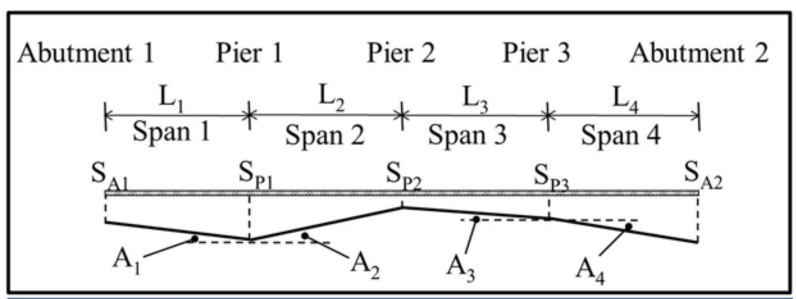
- The actual settlement of any support element could be as large as the value calculated by using a given method, and

- The actual settlement of the adjacent support element could be less, taken as zero in the limit, instead of the value calculated by using the same given method.

The above criteria lead to the "S-0" concept, with a value of S representing full settlement at one support of a span and a value of "0" representing zero settlement at an adjacent support. Use of the S-0 concept would result in an estimated maximum possible differential settlement equal to the larger of the two total settlements calculated at either end of any span. The use of S-0 approach may be prudent for cases where site variability is "high" based on judgment, criteria noted in commentary to Article 10.5.5.2.3 of AASHTO (2007 with 2009 Interims), and/or discussions of subsurface variability in Article 10.4 of AASHTO (2007 with 2009 Interims).

The application of the S-0 concept can be illustrated by considering a hypothetical case shown in Figure 3-2 of a 4-span bridge structure with 5 support elements (2 abutments and 3 piers) wherein the calculated settlement, S, at each support is different ($S_{A1} < S_{P1} > S_{P2} < S_{P3} < S_{A2}$). Differential settlements induce bending moments and shear in the bridge superstructure when the spans are continuous over supports. These moments and shears can potentially cause structural damage. To a lesser extent, differential settlements can also cause damage to a bridge consisting of simple spans. However, the major concern with simple span bridges is the quality of the riding surface and aesthetics. Due to a lack of continuity over the supports, the changes in slope of the riding surface near the supports of a simple span bridge induced by differential settlements may be more severe than those in a continuous span bridge.

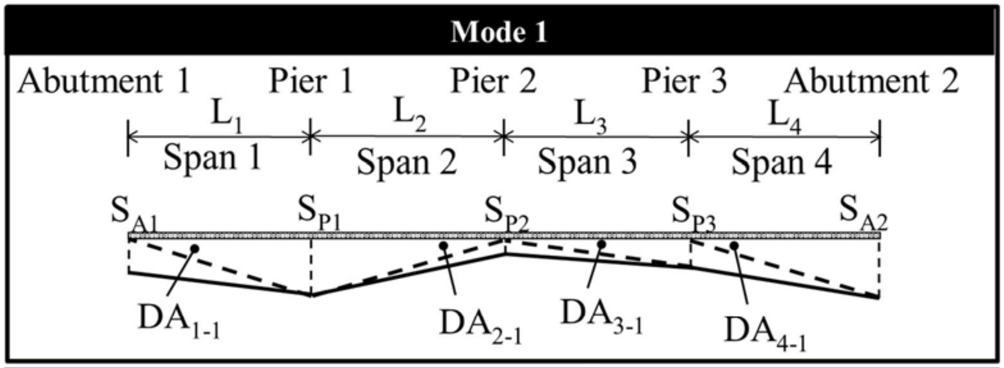
Support	Settlement
Abutment 1	S_{A1}
Pier 1	S_{P1}
Pier 2	S_{P2}
Pier 3	S_{P3}
Abutment 2	S_{A2}



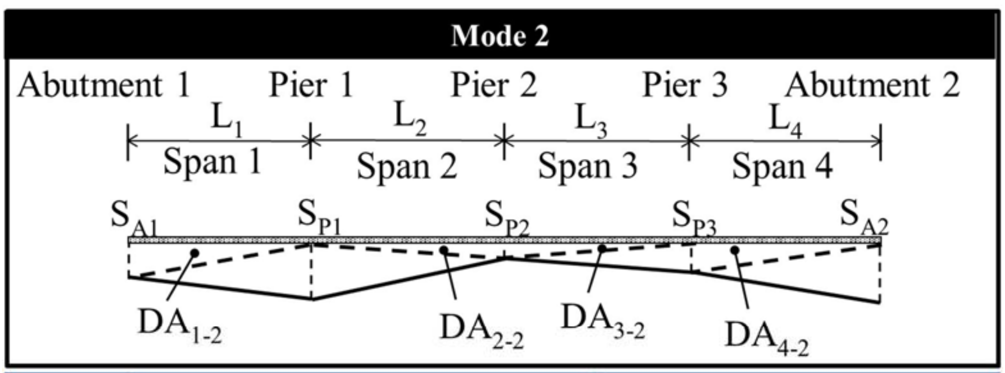
Span	Differential Settlement	Angular Distortion
1	$\lvert S_{A1} - S_{P1}\rvert$	$A_1 = (\lvert S_{A1} - S_{P1}\rvert)/L_1$
2	$\lvert S_{P1} - S_{P2}\rvert$	$A_2 = (\lvert S_{P1} - S_{P2}\rvert)/L_2$
3	$\lvert S_{P2} - S_{P3}\rvert$	$A_3 = (\lvert S_{P2} - S_{P3}\rvert)/L_3$
4	$\lvert S_{P3} - S_{A2}\rvert$	$A_4 = (\lvert S_{P3} - S_{A2}\rvert)/L_4$

Figure 3-2. Schematic. Concept of settlement and angular distortion in bridges.

For the hypothetical case shown in Figure 3-2, the differential settlements and angular distortions for design are evaluated as shown in Figure 3-3 based on the S-0 concept. As shown, two viable modes of deformation shapes, Mode 1 and Mode 2, are possible depending on which support settlement is assumed to be zero. The symbols for design differential settlement and angular distortion are $DS_{i\text{-}j}$ and $DA_{i\text{-}j}$, respectively, where i represents the span number (1 to 4) and j represents the mode (1 and 2). The hypothetical settlement profile assumed for computation of the maximum design angular distortion for each span is represented by the dashed lines in Figure 3-3. It should not be confused with the calculated total settlement profile that is represented by the solid lines. The values of design differential settlements and design angular distortions in Figure 3-3 are the maximum values for each span according to the criteria above. In AASHTO LRFD context, the "design" values are based on factored settlement values using the *SE* load factor.



Span	Design Differential Settlement	Design Angular Distortion
1	$DS_{1-1} = S_{P1}$ (assume $S_{A1} = 0$)	$DA_{1-1} = DS_{1-1}/L_1$
2	$DS_{2-1} = S_{P1}$ (assume $S_{P2} = 0$)	$DA_{2-1} = DS_{2-1}/L_2$
3	$DS_{3-1} = S_{P3}$ (assume $S_{P2} = 0$)	$DA_{3-1} = DS_{3-1}/L_3$
4	$DS_{4-1} = S_{A2}$ (assume $S_{P3} = 0$)	$DA_{4-1} = DS_{4-1}/L_4$



Span	Design Differential Settlement	Design Angular Distortion
1	$DS_{1-2} = S_{A1}$ (assume $S_{P1} = 0$)	$DA_{1-2} = DS_{1-2}/L_1$
2	$DS_{2-2} = S_{P2}$ (assume $S_{P1} = 0$)	$DA_{2-2} = DS_{2-2}/L_2$
3	$DS_{3-2} = S_{P2}$ (assume $S_{P3} = 0$)	$DA_{3-2} = DS_{3-2}/L_3$
4	$DS_{4-2} = S_{p3}$ (assume $S_{A2} = 0$)	$DA_{4-2} = DS_{4-2}/L_4$

Figure 3-3. Schematic. Estimation of maximum angular distortion in bridges.

3.3.1 Tolerable Angular Distortions

Based on the FHWA (1985) cited earlier in Section 2.2 of Chapter 2, AASHTO presents the guidance summarized in Table 3-1 for the evaluation of angular distortions in a bridge structure.

Table 3-1. Tolerable movement criteria for highway bridges (AASHTO 2002, 2007 with 2009 Interims)

Limiting Angular Distortion, DS/L (radians)	Type of Span
0.004	Continuous span
0.008	Simple span

The criteria in Table 3-1 suggest that for a 100-ft span, a differential settlement of 4.8-inches is acceptable for a continuous span and 9.6-inches is acceptable for a simple span. Such relatively large values of differential settlement create concern for structural designers, who often arbitrarily limit the criteria to one-half to one-quarter of the values listed in Table 3-1. While from a structural integrity viewpoint, there are no technical reasons for structural designers to set such arbitrary additional limits to the criteria listed in Table 3-1, there are often practical reasons based on the tolerable limits of deformation of other structures associated with a bridge, e.g., approach slabs, wingwalls, pavement structures, drainage grades, utilities on the bridge, deformations that adversely affect quality of ride, etc. Thus, the relatively large differential settlements based on Table 3-1, should be considered in conjunction with functional or performance criteria not only for the bridge structure itself but for all of the associated facilities. The following steps are suggested in this regard (FHWA 2006):

Step 1: Identify all possible facilities associated with the bridge structure, and the tolerance of those facilities to movements.

Step 2: Due to the inherent uncertainty associated with estimated values of settlement, determine the differential settlement by using conservative assumptions. It is important that the estimation of angular distortion is based on a realistic evaluation of the construction sequence and the magnitude of loads.

Step 3: Compare the angular distortion from Step 2 with the various tolerances identified in Step 1 and in Table 3-1. Based on this comparison identify the critical component of the facility. Review this critical component to check if it can be relocated or if it can be designed to more relaxed tolerances. Repeat this process as necessary for other facilities. In some cases, a simple re-sequencing of the construction of the facility based on the construction sequence of the bridge structures may help mitigate the issues associated with intolerable movements.

The above approach will help to develop project-specific limiting angular distortion criteria that may differ from the general guidelines listed in Table 3-1.

With respect to the hypothetical case shown in Figure 3-2, although the criteria for limiting angular distortions listed in Table 3-1 may be exceeded in one or more spans as estimated in Figure 3-3, they will generally be satisfied if the construction-point concept discussed in Chapter 4 is applied and the design angular distortions are computed based on loads placed after the construction of the substructure units. It will be shown that angular distortion after construction of the substructure units may be within tolerable limits.

3.3.2 Footings Proportioned for Equal Settlement

Often geotechnical and structural specialists will try to proportion footings for equal settlement. In this case, the argument is made that there will be no differential settlement or angular distortion. While this concept may work for a building structure because the footprint is localized, it is a fallacy to assume zero differential settlement for a long linear highway structure such as a bridge or a retaining wall because the properties of the geomaterials along the length of the structure will inevitably vary too much. Furthermore, as noted earlier, the prediction of settlements from any given method is uncertain by itself. Thus, for highway structures even when the footings are proportioned for equal settlement it is advisable to evaluate differential settlement assuming that the actual settlement of any support element could be as large as the value calculated by using a given method while at the same time, the actual settlement of the adjacent support element could be zero.

3.4 SUMMARY

Chapter 3 provides information that will help mitigate the third and fourth obstacles to the wider use of spread footings on soil for highway bridge structures. As noted in Chapter 1, the third obstacle is, “Use of conservative settlement prediction methods” and the fourth obstacle is, “Restrictive tolerable settlement criteria, which have led some agencies to set unrealistic performance criteria for foundations (e.g., zero or ¼ inch of settlement).” The key points in this regard are as follows:

(a) Guidance on the selection and use of appropriate settlement prediction methods is available.

(b) Since a bridge structure is much more tolerant to deformations than a conventional building structure, larger total footing settlements may be acceptable for highway bridges. Thus, there are no technical reasons for using restrictive and often unrealistic criteria for tolerable settlement such as zero or ¼ inch of settlement.

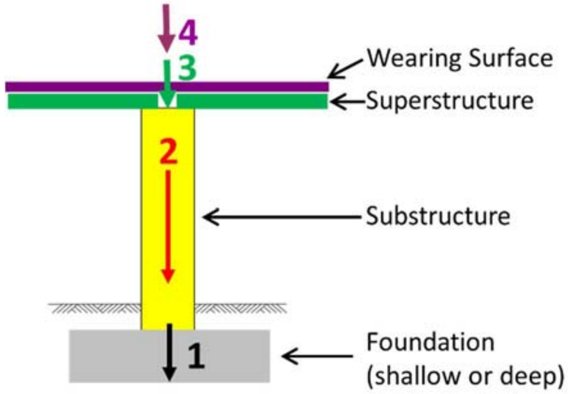
CHAPTER 4 – LOAD CONSIDERATIONS IN EVALUATION OF SETTLEMENTS

The estimation of settlement may be routinely accomplished with adequate geotechnical data and a realistic estimation of the structural loads that cause potentially damaging settlements. The overestimation of loads in the evaluation of settlements was identified in Chapter 1 as the fifth obstacle to the wider use of spread footings on soil for highway bridge structures. Chapter 4 attempts to mitigate this obstacle by discussing the construction-point concept to illustrate how settlements develop during construction of a bridge structure. Chapter 4 also discusses various other considerations that must be taken into account by both the geotechnical specialist and the structural specialist to reduce unnecessary conservatism in the estimation of settlements.

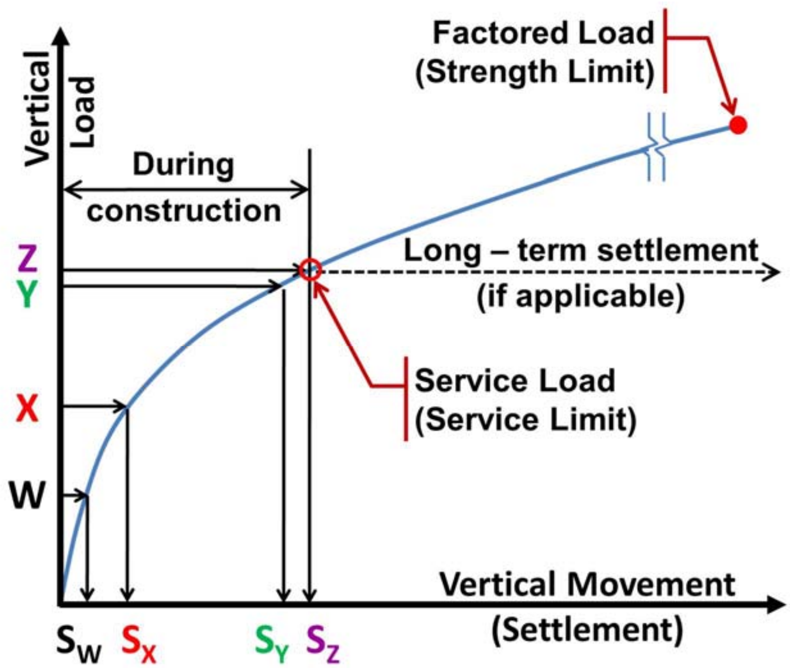
4.1 CONSTRUCTION-POINT CONCEPT

Most designers use the criteria for tolerable movements described in Chapter 3 as if a weightless bridge structure is instantaneously wished into place and all the loads are applied at the same time. In reality, loads are applied gradually as construction proceeds. Consequently, foundation movements will also occur gradually as construction proceeds. There are several critical construction points or stages during construction that should be evaluated separately by the designer. For the case of vertical loads and settlements, Figure 4-1a shows the critical construction stages (1, 2, 3, and 4) and Figure 4-1b shows the associated load-settlement behavior. In this case, the total foundation settlement, S_t, is equal to the settlement corresponding to the total construction Load, Z (= Load 1 + Load 2 + Load 3 + Load 4). Thus, $S_t = S_Z$. The settlements that occur before placement of the superstructure may not be relevant to the design of the superstructure. Thus, the settlements between application of loads in construction stages 2 and 4 are the most relevant. For continuous-span bridges the settlement, S_X, corresponding to Load X (= Load 1 + Load 2) may be applicable for computing relevant total settlement, S_{tr}. In this case, the total foundation settlement prior to construction of bridge superstructure, S_{tp}, is equal to the settlement, S_X, corresponding to Load X. Thus, for this example, $S_{tr} = S_Z - S_X$. Similarly, for simple-span bridges where settlement, S_Y, corresponding to Load Y (= Load 1 + Load 2 + Load 3) may be applicable for computing relevant total settlement, $S_{tr} = S_Z – S_Y$. Formulation of settlements in a manner shown in Figure 4-1b permits an assessment of settlements up to that point that can affect the bridge superstructure. Although Figure 4-1 illustrates the construction-point concept for the case of a pier, vertical loads and settlements (vertical movement), the concepts apply to other elements of bridge structure (e.g., abutments), load types (shears, moments, etc.) and deformation types (lateral movements, rotations, etc.).

Consideration of relevant settlements in conjunction with the type of superstructure and bearings can lead to a more rational consideration of spread footings on soils for highway bridges rather than an uninformed decision to select a costlier deep foundation system. Alternatively, if a deep foundation system is chosen due to reasons noted in Section 2.4.1 and Appendix C.3.1, the size of the deep foundation can be optimized, e.g., reduction in number of piles, reduction in size of piles, reduction in pile spacing with associated reduction in size of pile cap, and/or reduction in length of piles.



(a)



(b)

Legend

W	Load after footing construction	S_W	Settlement under load W
X	Load after pier column/wall construction	S_X	Settlement under load X
Y	Load after superstructure construction	S_Y	Settlement under load Y
Z	Load after wearing surface construction	S_Z	Settlement under load Z

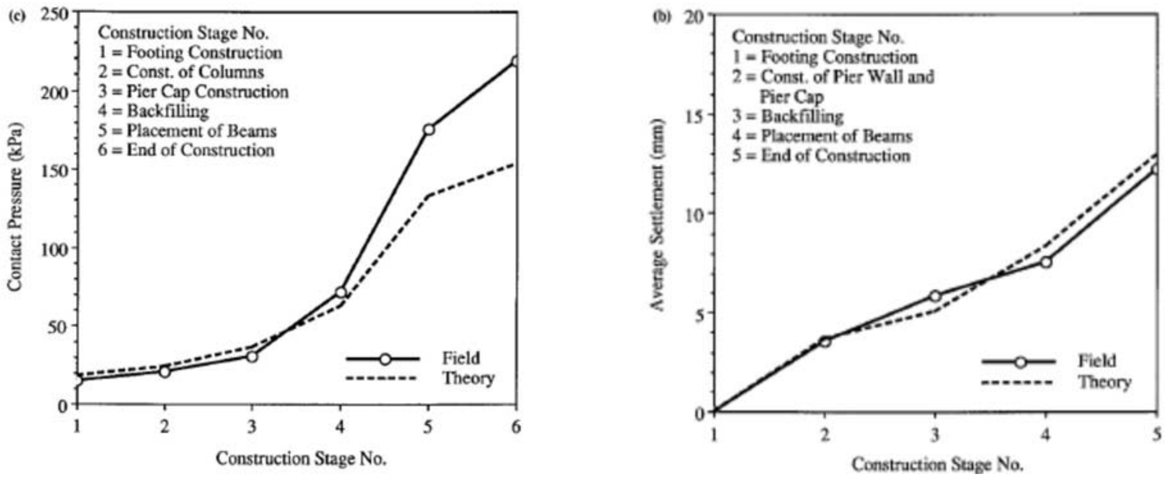
Figure 4-1. Schematic. Construction-point concept for a bridge pier, vertical loads, and vertical movement (settlement). (a) Identification of critical construction points, (b) Conceptual load-displacement pattern for a given footing width.

Sargand et al. (1999) and Sargand and Masada (2006) report the measurements of contact pressure, settlements, and tilting at various significant construction points for over 50 spread footings at 7 highway bridges in Ohio. Based on their study, Sargand and Masada (2006) made the following observations:

- The average contact pressure was typically less than 1 ksf at the end of footing construction, less than 4 ksf after placement of the beams and less than 5 ksf shortly after the construction of the bridge deck.
- The average settlement was typically less than 0.7 inches by the time the footing is backfilled, less than 1 inch after placement of beams, and less than 1.5 inches shortly after the bridge deck construction.
- The post-deck settlement varies from 0 to 0.4 inches with an average of 0.14 inches, which corresponds to 0 to 37.5% (average 19.0%) of the total settlement recorded in the field.
- The average degree of rotation was typically less than 0.15 degrees (0.0026 radians) by the time the beams were placed and less than 0.3 degrees (0.0052 radians) after the bridge deck construction.

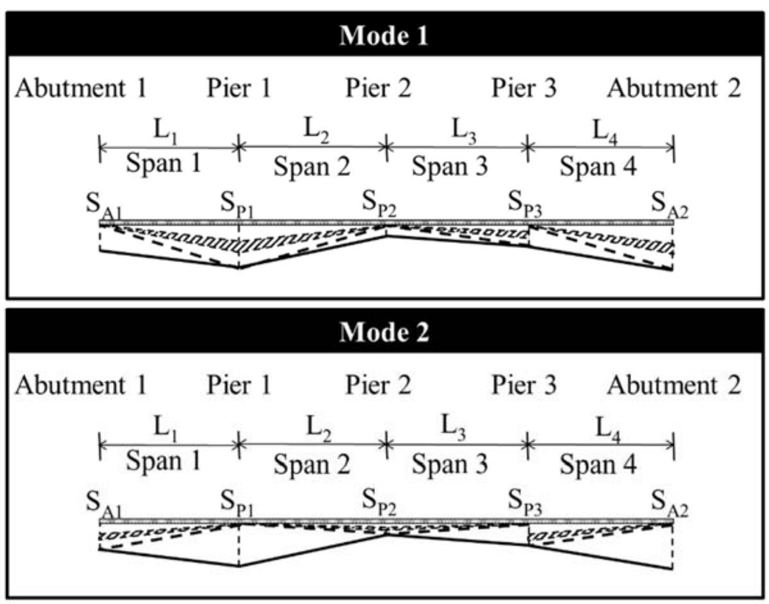
Figure 4-2 presents example plots from measurements of contact pressure and average settlement at various stages of construction of one of the bridges in the study by Sargand et al. (1999). It can be seen that the settlement between placement of beams and end-of-construction is less than 50 percent of the total settlement. This percentage is not unique and can vary from 25 to 75 percent depending on the type of the superstructure and the construction sequence. With respect to the example of the 4-span bridge and the angular distortions in Table 3-1, the use of the construction-point concept would result in smaller angular distortions to be considered in structural design. This will be true of any bridge evaluation. Using Figure 3-3 as a reference, Figure 4-3 shows a comparison of the profiles of the calculated settlements (solid lines), hypothetical maximum angular distortions (dashed lines) and the range of actual angular distortions (hatched pattern zones) based on the construction-point concept. For a given project and site-specific conditions, the actual relevant angular distortion profile will be represented by a dashed line within the hatched pattern zone. FHWA (2006) presents a detailed example for a bridge where application of this concept permitted the use of spread footings even though a total settlement of 7.5-inches was calculated.

Evaluation of incremental displacements between various construction points, when taken in conjunction with guidance on angular distortions provided in Table 3-1, can permit a more efficient design of the substructure as well as the superstructure. For example, adjusting the bearing levels can generally compensate for the settlements that occur before the placement of the superstructure. Therefore, such settlements may be irrelevant with respect to their effect on the design of the superstructure itself. Properly accounting for such settlements will lead to smaller settlements for the construction stages that follow, which may be of more interest from the viewpoint of differential settlement and angular distortion, e.g., the angular distortion between end-of-construction of a pier and after placement of the superstructure. Such considerations may lead to more efficient designs for both the substructure and the superstructure.



(a) (b)

Figure 4-2. Figure. Examples of data from measurements of contact pressure and settlement at various stages of bridge construction (Sargand, et al., 1999) [100 kPa = 1 tsf; 25.4 mm = 1 inch]. [Note: Figure (a) is for a pier, and (b) is for an abutment].



Legend:

Calculated settlement profile (refer to Figure 3-2)

Hypothetical settlement profile assumed for computation of maximum angular distortion.

Range of relevant angular distortions using construction-point concept

Figure 4-3. Schematic. Angular distortion in bridges based on construction-point concept.

While using the construction-point concept it is important to realize that the various quantities are being measured at discrete construction stages and the associated settlements are considered to be immediate. However, the evaluation of total settlement and the maximum (design) angular distortion as discussed previously must also account for long-term settlements. For example, significant long-term settlements may occur if footings are founded on saturated clay deposits or if a layer of saturated clay falls within the DOSI of the footing, even though the footing itself is founded on competent soil. In such cases long-term settlements will continue under the total construction load (Z) as shown by the dashed line in Figure 4-1. Continued settlements during the service life of the structure will tend to reduce the vertical clearance under the bridge with associated problems of high profile vehicles impacting the bridge superstructure. Therefore, the geotechnical specialist must estimate and report to the structural specialist the magnitude of the long-term settlement that will occur during the design life of the bridge. As noted in Chapter 3, the geotechnical specialist can estimate the long-term settlement by using consolidation theories described in texts such as Perloff and Baron (1976), Lambe and Whitman (1979), Holtz and Kovacs (1981) and Terzaghi et al. (1996). Finally, a key point in evaluating settlements at critical construction points is close coordination between the structural and geotechnical specialists.

4.2 OTHER CONSIDERATIONS

The accuracy of calculated settlements, regardless of the method used, is only as good as the quality of the geotechnical data and the estimation of the actual loads. Settlements of spread footings are frequently overestimated by designers for the following reasons:

1. *The structural load causing the settlement is not estimated properly*

   - In the absence of actual structural loads during the design process, geotechnical specialists conservatively assume that the bearing pressure is equal to the allowable bearing capacity (or factored bearing resistance in LRFD terminology). This scenario occurs when the geotechnical specialist develops the settlement estimate for a given bearing pressure. Use of the bearing resistance chart approach discussed in Appendix E alleviates this problem since the structural specialist can then use the bearing resistance chart as necessary at any stage of his/her design process when load refinements may be done. The geotechnical specialist is no longer justified to provide the structural specialist with only a single value of bearing resistance and an associated calculated settlement expressed as being less than a certain amount.

   - Structural specialists often estimate the loads at the upper plane of the footing. As noted earlier, spread footings for bridge structures are large and the weight of the footing can be significant. By estimating the loads on the upper plane of the footing, the weight of the footing is neglected, which has the effect of increasing the computed eccentricity. This method of estimating loads affects the effective width of the footing and the calculated settlements since the DOSI is a function of the effective footing width. Although computing loads at the upper plane of the footing is convenient to the structural specialist because the initial footing size is not known, some simple guidelines may help in identifying an initial footing size as follows (FHWA, 2006):

- For piers, assume a total footing width equal to ⅓ of the pier column height.
- For abutments, assume a total footing width equal to ½ of the abutment height.

As an alternative, the initial footing width may be assumed to be equal to 6 times the computed eccentricity based on service (working) loads at the upper plane of the footing. For initial load estimation purposes, the thickness of the footing for both of the above cases may be assumed to be 3- to 4-ft.

- Structural specialists should consider the net pressure at the base of the footing when estimating bearing pressures. Considering that the base of the footing for highway bridge structures is often embedded 5-ft to 6-ft below finished grade, the net pressure at the base of the footing is approximately 0.60 ksf to 0.72 ksf less than the gross footing pressure because of the effect of removing the soil. Since settlements occur under net bearing pressure, accounting for the effect of embedment will result in reduced calculated settlements. For spread footings in fill, the use of the net pressure is a function of whether the footing is placed in an excavated slot after the final configuration of the fill has been constructed or whether the final configuration of the fill is established after the footing has been constructed at a certain elevation within the fill. In the latter case the use of net pressure is not advisable.

2. *Preloading of the subsoil is not accounted for in the analysis.*

Preloading may be due to a geologic load applied in the past or to the removal of significant amounts of soil in construction prior to placement of the foundation. Not accounting for preloading can cause a grossly overestimated settlement. This is true for both granular soils and fine-grained cohesive soils. In the latter case, the preload effect is expressed in terms of over-consolidation ratio (OCR) and the effect of preloading is explicitly accounted for in consolidation theory (FHWA 2006). For sands, Schmertmann et al. (1978) recommended a reduction in settlement after preloading or other means of compaction of half the predicted settlement. Alternatively, in the case of preloaded soil deposits, the settlement can be computed by using the method proposed by D'Appolonia (1968, 1970), which includes explicit consideration of preloading.

3. *Settlement occurring during construction is not subtracted from total predicted amounts.*

By adoption of the construction-point concept discussed earlier, this consideration can be easily accounted for in the bridge design.

4. *Proper parameters are not used for estimation of settlement of footings on structural fills.*

Calculation of the settlement of a spread footing supported in or on an engineered fill requires an assumption about the compressibility of the fill material. Because structural fills should be constructed of good-quality granular materials and by following good construction techniques, the estimation of settlement lends itself to the application of the methods discussed in this report. To estimate settlements of footings in structural fills, an assumption must be made about the SPT N-value that is representative of the engineered fill. For fills constructed using good-quality granular materials, as discussed in Section C.3.3.3 of Appendix C, FHWA (2006) suggests a corrected SPT N-value ($N1_{60}$) of 23 blows per foot as a representative value for estimating settlement in structural fills if a relative compaction of 95% based on Modified Proctor (ASTM D 1557) compaction energy is used. The $N1_{60}$ value can be increased to 32 blows per foot if 97% relative compaction based on Modified Proctor compaction energy is used.

5. *Construction Inspection and Monitoring*

As is the case with any foundation system, the successful in-service performance is dependent on the care taken during construction. It is important that the project plans and specifications carefully detail any specific items that need to be inspected and monitored. Appendix D presents information on construction inspection and monitoring for spread footings.

4.3 SUMMARY

Chapter 4 provides information that will help mitigate the fifth obstacle to the wider use of spread footings on soil for highway bridge structures. As noted in Chapter 1, the fifth obstacle is "overestimation of loads that generate settlement." The key points in this regard are as follows:

(a) Settlements occur progressively and a significant portion of the immediate (elastic) settlement may be realized before it becomes an issue with respect to the structural performance of the bridge superstructure.

(b) In the absence of long-term (time-dependent) consolidation settlements, structural designers should properly estimate the loads that are consistent with the construction sequence of the bridge structures. Only the immediate settlements resulting from loads imposed by the superstructure construction sequence should be used to evaluate the effects of settlement and angular distortion on the bridge superstructure.

CHAPTER 5 –LRFD AND SPREAD FOOTINGS

In Chapter 1, institutional culture and bias was listed as the sixth obstacle to the wider use of spread footings on soil for highway bridge structures. This obstacle is likely a result of the cumulative effect of the first five obstacles listed in Chapter 1, and a long period of doing "business as usual." However, the mandatory implementation of LRFD methodology since October 1, 2007 for states seeking federal funding for new highway bridge projects is forcing designers to re-evaluate their "business as usual" current practices. The change in design platform from allowable stress design (ASD) to LRFD is not easy and requires significant efforts by agencies, their consultants and the contractors doing the construction work to learn about LRFD and document new approaches. The complete revision to LRFD also provides agencies, consultants and contractors an ideal opportunity to revisit past practice and to make appropriate changes. With this in mind, the purpose of this chapter is to introduce significant guidance on spread footing design using the AASHTO LRFD approach. This guidance will also permit the designers to take a fresh look at current practice for evaluating and selecting foundation alternatives based on performance experience and expectations.

5.1 ANALYSIS OF SPREAD FOOTINGS USING THE LRFD APPROACH

The LRFD concepts and methodology provide a rational mechanism to incorporate into the design process most of the remedies to the obstacles to the wider use of spread footings on soil for highway bridge structures listed in Chapter 1. As discussed previously in this report, these remedies included a proper understanding of the use of spread footings based on performance data, appropriate settlement estimation methods, the use of the construction-point analysis, and a realistic estimate of angular distortions. In addition to presenting information on each of these remedies, a major component of the scope of work of this study is to provide guidance for the evaluation of spread footings in the LRFD context. Appendix E provides a comprehensive treatment of the analysis of spread footings based on the LRFD approach. The following information can be found in Appendix E.

- Discussion of the LRFD-based limit states applicable to spread footings.
- Streamlined evaluation of bearing resistance and settlement through use of a convenient bearing resistance chart that incorporates LRFD limit states.
- A step-by-step design process flow chart.
- A comprehensive numerical example that demonstrates through detailed calculations the various concepts introduced in this report. This numerical example is based on an actual bridge structure.

Upon review of Appendix E, it will be readily apparent to a designer that now there are explicit mechanisms available through the LRFD approach to address the concerns of structural specialists regarding geotechnical data presentation and design input developed by geotechnical specialists. In the past the geotechnical specialist's input was generally limited to a broad evaluation of settlements and the effect of footing configurations, i.e., width and depth, on such settlements. The LRFD approach requires much closer collaboration between the geotechnical and structural specialists than has existed in the past and thereby assures opportunity for a more realistic and reliable design of feasible foundation elements, especially in the case of spread footings on soils.

5.2 OPPORTUNITY FOR NEW LRFD-BASED GUIDANCE

As state DOTs and others implement the LRFD platform, they will be revisiting their procedures for foundation design. This entire report, and in particular Appendix E, provides guidance on how this can be done for spread footings on soils. The performance data, previous publications, and the experiences of state DOTs reported here provide the technical support to develop LRFD-based guidance that considers the widest possible use of spread footings on soils. Indeed, this is an excellent opportunity for states to move away from past practice that is based perhaps too heavily on "It's the way we have always done it" to a practice that is based on a wider body of performance data.

5.3 SUMMARY

Chapter 5 provides information that will help mitigate the sixth obstacle to the wider use of spread footings on soil for highway bridge structures. As noted in Chapter 1, the sixth obstacle is "institutional culture and bias." The key points in this regard are as follows:

(a) This report provides a clear and rational approach to the evaluation of spread footings by the LRFD methodology through inclusion of comprehensive LRFD guidance in Appendix E. This approach provides opportunities to designers for overcoming institutional culture and biases that have been and continue to be an obstacle to the wider use of spread footings on soil for highway bridge structures.

(b) The migration from allowable stress design (ASD) to LRFD is a great opportunity for agencies to redraft their foundation design guidance to ensure that spread footings are given due consideration in the foundation selection process.

CHAPTER 6 – SUMMARY AND CONCLUSIONS

The primary goal of this report is to promote spread footings bearing on natural soils (native or improved) and compacted engineered fill materials as a routine alternative to deep foundations for the support of highway bridges. The following approach was taken to attain that goal:

1. Identify the obstacles that create an impediment to the wider use of spread footings on soil for highway bridge structures,
2. Recognize that the obstacles can be overcome without extensive research. Rather they can be mitigated by studying past work, conducting careful analyses, and having good communication between geotechnical and structural specialists, and
3. Provide documentation and guidance here that can help mitigate and overcome those obstacles.

In the above framework, using documented performance data, the report presents powerful concepts such as the use of construction-point analysis and angular distortions to demonstrate the efficacy of using spread footings on soils. Implementation of these concepts requires only that conventional computations be taken one step further without any requirement for advanced computational skills. The report presents sources of information that agencies and designers can use as references in their project applications. Based on the information presented in Chapters 1 to 5 and Appendices A and B the following key points and conclusions can be summarized:

- Significant resources are available to bridge designers to evaluate the performance of spread footings on soils and determine the feasibility of their use for highway bridge structures.

- The size of spread footings on soils for highway bridges is controlled by settlement considerations including differential settlements and associated angular distortions. Field measurements of actual bearing pressures suggest that spread footings have a large margin of safety with respect to soil strength since the loads are well below the nominal bearing resistance in LRFD terminology or ultimate bearing capacity in ASD terminology.

- By properly accounting for the construction sequence of a bridge structure, settlements that are relevant to the performance of the bridge structure are often only 25 to 50 percent of the total settlements. This statement is supported analytically and by measurements on actual bridge footings in the field. The implications are that actual angular distortions will be similarly reduced so that the AASHTO tolerable movement criteria for highway bridges will rarely be exceeded for conventional bridge loads carried by spread footings founded on soils.

- Significant documentation for the successful implementation of spread footings on soils is available. For example, based on field measurements it has been shown that the typical contact pressure under a highway bridge footing varies from 3 to 6 ksf. Also, when expressed in terms of the ratio of measured settlement, S_m, to total footing width, B_f, the S_m/B_f values range from 0.1% to 1.0% for footing widths greater than 10-ft. For smaller footing widths values of S_m/B_f as large as 2.50% can occur. These values have been independently verified by a number of researchers. Thus, the field performance data clearly demonstrate that the spread footings for highway bridges operate in a safe manner.

Significant additional information regarding the use of spread footings on soils is presented in Appendices C and D. Based on the information presented in these appendices, the following significant points and conclusions can be summarized:

- Spread footings in structural fills at abutments have been constructed successfully and are currently performing satisfactorily. Appendix C of the report presents material specifications and suggested fill configurations based on successful applications by various agencies.

- The successful use of spread footings to support abutments on top of MSE walls is documented for many projects. This concept can result in significant savings besides mitigating the "bump at the end of the bridge."

- Spread footings can and have been used to support semi-integral and integral abutments. Section C.2.2 discusses these applications and provides an example to demonstrate the successful application of spread footings for such applications.

- Detailed construction inspection, monitoring and instrumentation guidelines are available in Appendix D that can be implemented on a routine basis in a cost-effective manner.

Finally, a detailed approach for the evaluation of spread footings in the LRFD framework is presented in Appendix E of the report. This stand-alone appendix presents procedures requiring that only conventional computations be taken one step further without the need for advanced computational skills or analytical tools. Using the methods in Appendix E, bridge designers can now rationally evaluate spread footings on soils when making decisions on the choice of the most cost-effective foundation systems for bridges.

In conclusion, this report has tackled the obstacles (perceived or otherwise) that are an impediment to the wider use of spread footings on soils for highway bridge structures. Using documented performance data and the now mandatory LRFD methodology, the report presents detailed guidance on the evaluation of spread footings on soils in a rational and unbiased manner. FHWA resources are identified and a comprehensive bibliography is presented to assist structural designers in the evaluation of spread footings on soils. Between the resources, bibliography and comprehensive documentation and guidance, this report provides a clear avenue for agencies and designers to make an informed evaluation of the use of cost-effective spread footings for all conditions other than those where their use may be suspect or is clearly precluded (e.g., scour conditions). This type of evaluation should be done as a matter of routine as part of the foundation type selection process.

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APPENDIX A – AVAILABLE RESOURCES

This work was sponsored by the FHWA Resource Center's Technology Deployment program and the Geotechnical and Hydraulics Technical Services Team (TST). Geotechnical specialists from the TST are engaged with the FHWA Division offices in each state and territory, and with the Divisions of the FHWA Federal Lands Highway program, as well as with state highway agencies and local agencies through the Local and Tribal Technology Assistance Programs (LTAP and TTAP). Through this engagement the team has an understanding of standards of practice (SOP), regional practices, and successes and failures, and is in a unique position to pass on these observations and foster deployment of good technologies where they are not currently used.

Working with both the FHWA Headquarters Office and the FHWA Turner Fairbank Highway Research Center, the TST is also involved with ongoing research, and in setting current policy and guidance. Research is being conducted internally through federal funded programs, through pooled fund studies, and through NCHRP. FHWA geotechnical engineers serve in at least a liaison role in all of these efforts to guide and stay appraised of developments. Policy and guidance is set through the Headquarters Office of Infrastructure in the form of policy memoranda or technical guidance documents, such as Geotechnical Engineering Circulars (GECs), and National Highway Institute (NHI) training materials and reference manuals. Policy and guidance is set through collaboration with AASHTO, TRB and national industry groups such as ASCE, and geotechnical team members are active in the committees of these organizations.

Readers of this report are encouraged to use the FHWA resources to help implement the findings of this work as appropriate for their state or region. The TST members and resources can be accessed at the following site (or by internet search for FHWA, geotechnical, resource center):

http://www.fhwa.dot.gov/resourcecenter/teams/geohydraulics/

The TST can help states and Divisions, as well as LTAPs and TTAPs through delivering online or in person training, through technical assistance at the foundation selection stage of a project, or through conducting what is called a 'Process Review' of the foundation selection process. The review would consider the current process in light of the experiences of others, and might reveal specific obstacles for the fair evaluation and selection of spread footings on soils that could be overcome, or it might confirm that the current process is reasonable and appropriate.

Readers are also directed to the agency home page for geotechnical engineering at the following site (or by internet search for FHWA geotechnical):

http://www.fhwa.dot.gov/engineering/geotech/

This site provides links to references, and to the home pages of the Resource Center and Turner Fairbank Highway Research Center, for example.

APPENDIX B – FIELD PERFORMANCE DATA FOR LARGE SIZE SPREAD FOOTINGS ON SOILS

As part of this study, an attempt was made to locate field performance data for large size spread footings on soils because footing sizes for highway bridges are large as noted in Chapter 2. For such large size footings, settlement controls the footing design. Although there are several significant data bases in the geotechnical literature that present performance data, most of these data are related to building foundations that are comparatively smaller in size. For smaller size footings bearing resistance is the controlling factor rather than settlements. Examples of these data bases include the work performed by Burland and Burbridge (1984), FHWA (1987), Baus (1992), FHWA (1997), Sargand and Masada (2006), etc.

After screening the available open literature, 84 cases from 6 data bases were found to contain useful documented information relevant to this study. It is acknowledged that there are several other good data bases for the performance of large spread footings, but these are not available in the public domain or are published in languages other than English. In addition, there are anecdotal case histories, but they do not have documented data that is useful.

A summary of the relevant data from the sources found in the open literature is included in this Appendix in a tabular format. The summary includes the dimensions of the footings (B_f, L_f, D_f), maximum contact pressure (q_{max}), the measured (uncorrected) SPT N-values (N), overburden corrected N-values (N_{corr}), and measured settlement (S_m). The computed values of S_m/B_f expressed as a percentage are presented in the last column of Tables B-1, B-2, B-4 and B-5. The range of computed values of S_m/B_f expressed as a percentage is presented in the last two columns of Table B-3. These values of S_m/B_f are used in Chapter 2 to evaluate the performance data.

Table B-1. Data established by FHWA (1987).

Structure		B_f (ft)	L_f/B_f (dim)	D_f/B_f (dim)	N_{corr} (dim)	q_{max} (ksf)	S_m (inch)	S_m/B_f (%)
1	Bridge 1 – Abutment 1	17.0	3.7	NA	44.0	3.2	0.35	0.17
2	Bridge 1 – Abutment 2	17.0	3.7	NA	58.0	2.6	0.67	0.33
3	Bridge 2 – Abutment 1	15.3	3.4	NA	43.0	2.4	0.94	0.51
4	Bridge 2 – Abutment 2	16.8	3.1	0.2	19.0	2.4	0.76	0.38
5	Bridge 2 – Pier	12.5	3.3	0.4	12.0	1.8	0.61	0.41
6	Bridge 3 – W. Abutment	11.0	6.8	NA	34.0	1.8	0.42	0.32
7	Bridge 3 – E. Abutment	18.5	4.3	0.3	22.0	2.4	0.61	0.27
8	Bridge 3 – Pier 1 North	21.0	1.0	0.2	18.0	2.0	0.28	0.11
9	Bridge 3 – Pier 1 South	21.0	1.5	0.2	18.0	1.6	0.26	0.10
10	Bridge 3 – Pier 2 North	16.0	1.7	0.3	20.0	2.4	0.29	0.15
11	Bridge 3 – Pier 2 South	16.0	1.2	0.3	22.0	2.4	0.25	0.13
12	Bridge 3 – Pier 3 North	21.0	1.6	0.2	15.0	1.4	0.97	0.38
13	Bridge 3 – Pier 3 South	21.0	1.4	0.2	25.0	1.6	0.98	0.39
14	Bridge 4 – S. Abutment	8.1	5.3	NA	21.0	3.4	0.46	0.47
15	Bridge 4 – N. Abutment	8.1	5.3	NA	8.0	3.4	0.34	0.35
16	Bridge 5 – N. Abutment	16.8	4.6	0.4	42.0	2.4	0.23	0.11
17	Bridge 5 – S. Abutment	15.3	5.0	0.4	24.0	2.4	0.44	0.24
18	Bridge 6 – Abutment 1	15.3	4.0	0.6	55.0	1.8	2.26	1.23
19	Bridge 6 – Abutment 2	15.3	4.4	0.6	39.0	1.8	0.83	0.45
20	Bridge 7 – Abutment 2	28.0	1.0	0.0	24.0	2.2	0.64	0.19
21	Bridge 8 – Abutment 1	20.0	5.0	1.1	23.0	3.0	0.46	0.19
22	Bridge 8 – Abutment 2	20.0	5.0	0.3	38.0	3.2	0.66	0.28
23	Bridge 9 – Abutment 1	21.8	2.0	NA	39.0	3.6	0.61	0.23
24	Bridge 9 – Abutment 2	16.0	2.8	0.0	49.0	3.4	0.28	0.15

Note: NA – Not Available

Table B-2. Data compiled from literature in FHWA (1987).

1	Bergdahl & Ottosson (1982)	16.4	1.7	0.5	24.0	3.8	0.50	0.25
2	DeBeer (1948)	19.0	4.2	0.4	17.0	1.6	0.50	0.22
3	DeBeer (1948)	19.7	2.7	0.5	42.0	3.2	0.30	0.13
4	DeBeer (1948)	19.7	2.7	0.6	42.0	4.4	0.20	0.08
5	DeBeer (1948)	23.0	5.1	0.3	42.0	2.8	0.50	0.18
6	DeBeer (1948)	17.0	5.4	0.4	42.0	2.0	0.40	0.20
7	DeBeer & Martens (1956)	9.8	3.4	1.0	50.0	4.8	0.80	0.68
8	DeBeer & Martens (1956)	8.5	8.1	0.8	9.0	4.2	1.30	1.27
9	Levy & Morton (1974)	13.0	1.8	1.3	32.0	10.6	0.50	0.32

Table B-3. Data established by Sargand et al. (1999) and Sargand and Masada (2006).

Structure		B_f (ft)	L_f/B_f (dim)	D_f (ft)	N_{corr} [(a)] (dim)	q_{max} (ksf)	S_m (inch)	S_m/B_f (%)		
1	Bridge A – Panel A&B	12.0 – 16.0	3.00 – 3.25	3	39/48	3.0	0.40	0.28	-	0.21
2	Bridge A – Panel C						0.60	0.42	-	0.31
3	Bridge A – Panel D						0.70	0.49	-	0.36
4	Bridge A – Panel E						0.40	0.28	-	0.21
5	Bridge A – Panel F						0.60	0.42	-	0.31
6	Bridge A – Panel G/H						0.80	0.56	-	0.42
7	Bridge B – Abutment 1	16.8	3.80 – 4.60	4	41/44	3.4	0.90	0.45	-	
8	Bridge B – Center Pier						0.70	0.35	-	
9	Bridge B – Abutment 2						1.00	0.50	-	
10	Bridge C [(b)] – West Footing	4.0	12.20	12	18/16	4.8	1.20	2.50	-	
11	Bridge C [(b)] – East Footing						1.00	2.08	-	
12	Bridge D – Pier 1-N	7.50 – 10.50	2.50 – 3.60	4	37/35	6.4	0.90	1.00	-	0.71
13	Bridge D – Pier 1-S						1.00	1.11	-	0.79
14	Bridge D – Pier 2-N						1.20	1.33	-	0.95
15	Bridge D – Pier 2-S						1.10	1.22	-	0.87
16	Bridge D – Pier 3-N						0.60	0.67	-	0.48
17	Bridge D – Pier 3-S						0.60	0.67	-	0.48
18	Bridge D – Pier 4-N						0.70	0.78	-	0.56
19	Bridge D – Pier 4-S						0.60	0.67	-	0.48
20	Bridge D – Pier 5-N						0.40	0.44	-	0.32
21	Bridge D – Pier 5-S						0.45	0.50	-	0.36
22	Bridge E – Rear Abutment	11.0 – 15.0	1.90	4	22/22	4.0	1.40	1.06	-	0.78
23	Bridge E – Pier 1						0.80	0.61	-	0.44
24	Bridge E – Pier 3						0.55	0.42	-	0.31
25	Bridge E – Pier 4						0.60	0.45	-	0.33
26	Bridge E – Pier 5						0.20	0.15	-	0.11
27	Bridge E – Pier 6						0.34	0.26	-	0.19
28	Bridge E – Pier 7						0.42	0.32	-	0.23
29	Bridge E – Pier 8						0.50	0.38	-	0.28
30	Bridge E – Forward Abutment						0.48	0.36	-	0.27
31	Bridge F – Center Pier	8.0	5.00	8	50/49	4.0	0.20	0.21	-	
32	Bridge G – Pier 18	21.0 – 24.0	2.10 – 3.20	9	27/38	2.8	0.70	0.28	-	0.24
33	Bridge G – Pier 19				47/51		0.96	0.38	-	0.33
34	Bridge G – Forward Abutment						1.68	0.67	-	0.58

Notes:

(a) Two values of N_{corr} are presented. The first value is the average corrected N-value over a depth of $1B_f$ and the second value is the average corrected N-value over a depth of $2B_f$.

(b) Bridge C is a 3-sided culvert with side walls on footings. For this structure as well with other structures with footing widths smaller than 10-ft, the S_m/B_f values are larger.

Table B-4. Data established by Baus (1992).

Structure		B_f (ft)	L_f/B_f (dim)	D_f (ft)	N(a) (dim)	Q(b) (kips)	S_m (inch)	S_m/B_f (%)
1	Bridge 1, Bent 3 – Footing 1	15.0	1.000	5.9	29	678	0.426	0.24
2	Bridge 1, Bent 3 – Footing 4	15.0	1.000	4.7	39	678	0.520	0.29
3	Bridge 2, Bent 3, EBL – Footing 1	12.5	1.080	6.2	23	496	0.408	0.27
4	Bridge 2, Bent 3, EBL – Footing 3	12.5	1.080	6.3	23	496	0.468	0.31
5	Bridge 2, Bent 3, WBL – Footing 1	12.5	1.080	5.6	23	496	0.300	0.20
6	Bridge 2, Bent 3, WBL – Footing 3	12.5	1.080	5.7	23	496	0.324	0.22
7	Bridge 2, Bent 4, EBL – Footing 1	12.0	1.125	9.0	25	495	1.728	1.20
8	Bridge 2, Bent 4, EBL – Footing 3	12.0	1.125	9.0	25	495	1.631	1.13
9	Bridge 3, Bent 2, SBL – Footing 1	11.5	1.348	15.0	22	520	0.336	0.24
10	Bridge 3, Bent 2, NBL – Footing 1	11.5	1.348	9.0	34	520	0.636	0.46
11	Bridge 3, Bent 2, NBL – Footing 4	11.5	1.348	9.0	47	520	0.444	0.32

Notes:

(a) N-value is an average measured (uncorrected) N-value within a depth of approximately $2B_f$. Baus (1992) also report CPT results.

(b) Q is the estimated dead load.

Table B-5. Data established by FHWA (1997) [Same as by Briaud and Gibbens, 1999].

Structure		B_f (ft)	L_f/B_f (dim)	D_f (ft)	N(a) (dim)	Q(b) (kips)	S_m (inch)	S_m/B_f (%)
1	Footing 1 (3-m North Footing)	9.85	1.00	2.50	20/50	1011	1.00	0.85
2	Footing 2 (1.5-m footing)	4.89	1.01	2.50	20/50	337	1.00	1.70
3	Footing 3 (3-m South Footing)	9.89	1.00	2.92	20/50	1011	1.00	0.85
4	Footing 4 (2.5-m Footing)	8.19	1.00	2.50	20/50	809	1.00	1.02
5	Footing 5 (1.0-m Footing)	3.25	1.00	2.33	20/50	191	1.00	2.56

Notes:

(a) Two values of measured (uncorrected) SPT N-value are presented. The first value is the average N-value over a depth of 30-ft and the second value is the average N-value below 30-ft to approximately 36-ft which was the bottom of exploration. The report also presents subsurface data from a number of different methods, e.g., pressuremeter, CPT, DMT, etc. Full load settlement curves are also reported. Refer to the report for details.

(b) Q is the applied load and is obtained from load-settlement measurements based on application of a concentric load and corresponds to a measured settlement, S_m, of 1.00-in.

Table B-6. Data from FHWA archives.

Structure		B_f (ft)	L_f/B_f (dim)	D_f/B_f (dim)	N_{corr} (dim)	q_{max} (ksf)	S_m (inch)	S_m/B_f (%)
1	I-359, Tuscaloosa, Alabama	12.0	1.00	NA	NA	8	0.10	0.07

Note: NA – Not Available

APPENDIX C – GENERAL DESIGN CONSIDERATIONS FOR SPREAD FOOTINGS

The purpose of this appendix is to present general considerations for the design of spread footings. The appendix is organized to present information that addresses the following items:

- Terminology (Section C.1) – this section presents definitions and terminology for spread footings in general, e.g., isolated, strip, depth of significant influence (DOSI), etc.
- Uses (Section C.2) – this section presents the various potential uses of spread footings for highway bridges.
- Feasibility evaluation (Section C.3) – this section presents various considerations for evaluating the feasibility of using spread footings on soils for highway bridges.
- Horizontal deformations (Section C.4) – this section presents mechanisms that lead to horizontal deformations of spread footings for highway bridges.

C.1 TERMINOLOGY – GENERAL TYPES OF SPREAD FOOTINGS

The geometry of a typical spread footing foundation is shown in Figure C.1-1. As the name suggests, a spread footing spreads the applied load over the area of contact between the footing and the soil and eventually into the subsurface soils beneath the footing. Thus, the spread footing relies on bearing resistance in contrast to deep foundations where frequently a large component of the applied load is carried by side resistance. The depth of embedment, D_f, of spread footings is small compared to their cross-sectional size (width, B_f, or length, L_f).

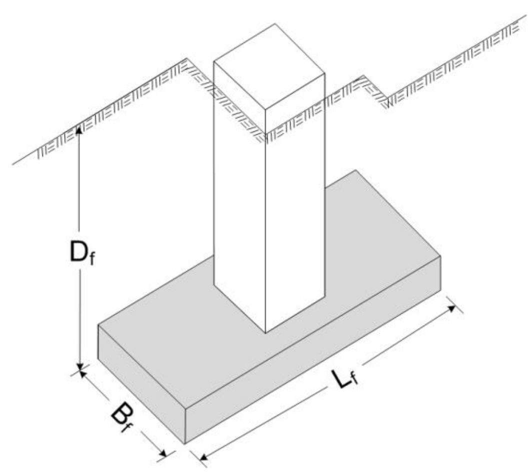


Figure C.1-1. Schematic. Geometry of a typical spread footing foundation (AASHTO 2002).

Common types of spread footings are shown in Figures C.1-2 through C.1-4. Following are certain size definitions and considerations that help in characterizing spread footing foundations (FHWA 2006):

- Footings with L_f/B_f less than 10 are considered to be **isolated spread footings**. As shown in Figure C.1-2, isolated spread footings are designed to distribute the concentrated loads delivered by a single column to prevent shear failure of the soil beneath the footing. The size of the footing is a function of the loads imposed by the supported column and the stiffness (strength and compressibility) characteristics of the bearing materials beneath the footing. For bridge columns, B_f is typically greater than 10 ft (3 m) and $L_f/B_f < 10$. B_f, and/or L_f increase when eccentric loads are applied to the footing. The depth of significant influence (DOSI) for settlement analysis, i.e., the depth below the base of the footing to which applied stresses are significantly felt in the soil, varies from $2*B_f$ for $L_f/B_f = 1$ to $4*B_f$ for $L_f/B_f \geq 10$. Structural design of isolated spread footings includes consideration for flexural resistance at the face of the column as well as shear and punching around the column.

- Footings with $L_f/B_f \geq 10$ are considered to be **continuous** or **strip footings**. As shown in Figure C.1-3, strip footings typically support a bearing wall to reduce the pressure on the bearing materials. Sizing and structural design considerations are similar to those for isolated spread footings with the exception that plane strain conditions are assumed to exist in the direction parallel to the long axis of the footing. This assumption affects the DOSI for settlement analysis. In contrast with isolated footings where the DOSI is between $2*B_f$ to $4*B_f$, the DOSI in the case of strip footings will always be $4*B_f$. The structural design of strip footings is generally governed by beam shear and bending moments.

- **Combined footings** are similar to isolated spread footings or continuous footings except that they support two or more columns and are rectangular or trapezoidal in shape (Figure C.1-4). They are used primarily when the column spacing is non-uniform (Bowles, 1996) or when isolated spread footings become so closely spaced that a combination footing is simpler to form and construct. Due to the frame action that develops with combined footings, they can be used to resist large overturning or rotational moments in the longitudinal direction of the column row. When L_f/B_f of the combined footing is ≥ 10, it should be treated analytically like a continuous footing; when L_f/B_f of the combined footing is < 10, it should be treated analytically like an isolated footing.

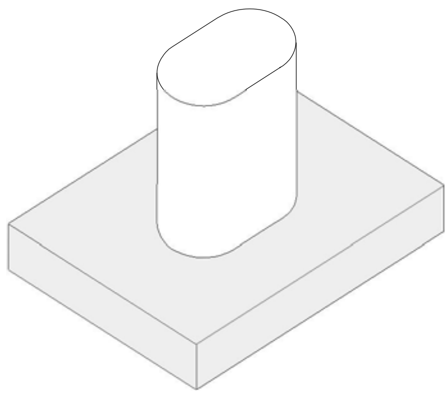


Figure C.1-2. Schematic. Isolated spread footing.

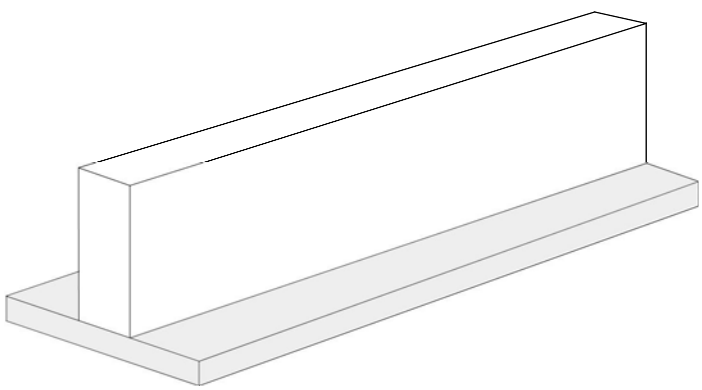


Figure C.1-3. Schematic. Continuous strip footing.

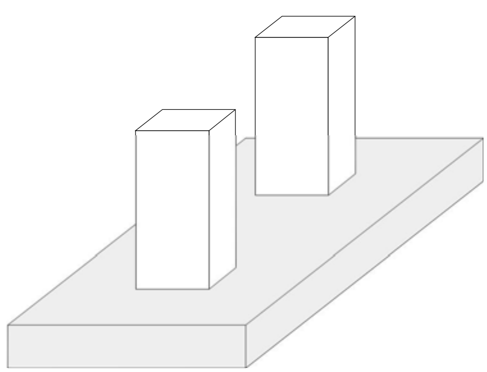
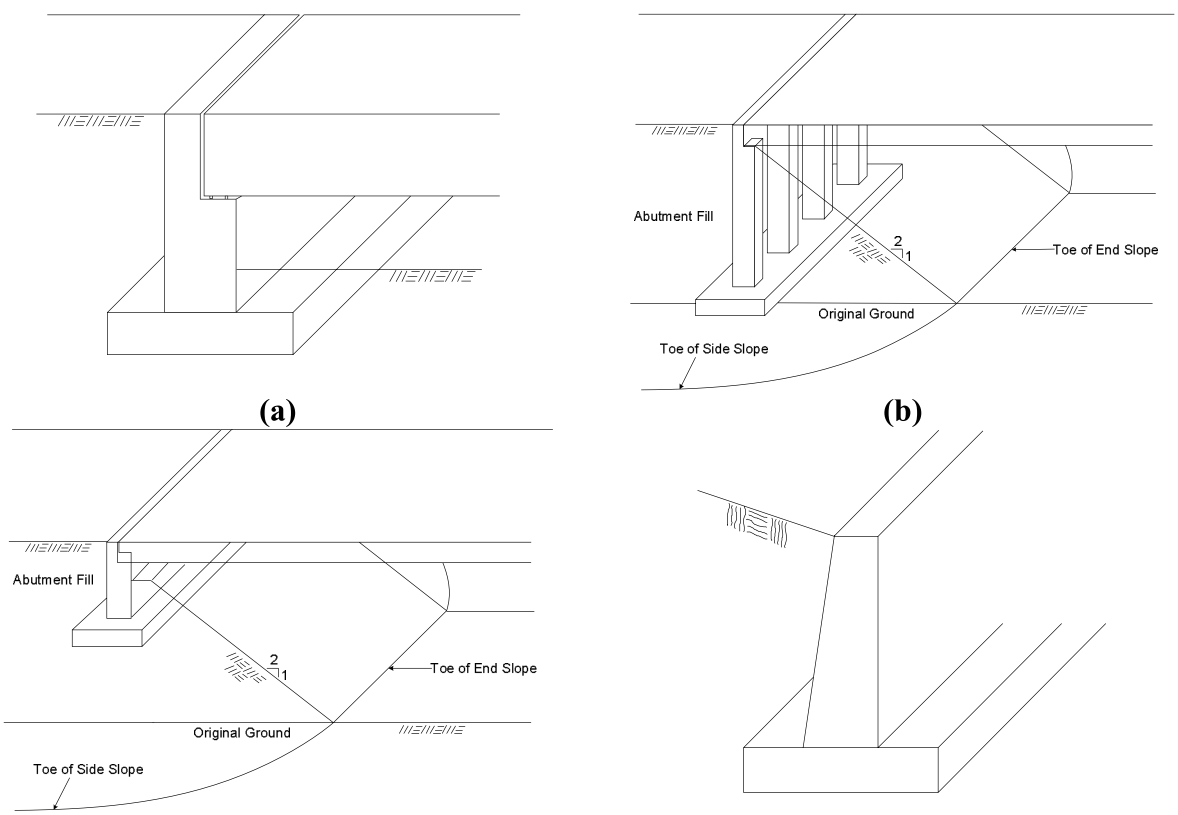


Figure C.1-4. Schematic. Combined footing.

C.2 USE OF SPREAD FOOTINGS FOR PIERS, ABUTMENTS AND RETAINING WALLS

Spread footings have been used in a variety of configurations to support bridge abutments and piers. The common uses of spread footings for abutments at highway bridges and cantilever retaining walls are shown in Figures C.2-1. At piers, the spread footings can be isolated footings as shown Figure C.1-2 or combined as shown in Figure C.1-4. Other applications of spread footings, e.g., footings on top of MSE walls and for integral (or semi-integral) abutments on soils for highway bridges are discussed in Section C.2.1 and Section C.2.2, respectively.

Compared to spread footings used for buildings, the size of spread footings for highway bridge applications is large. For example, the width of spread footings for continuous strip applications may range from 10- to 15-ft with lengths as great as 60- to 120-ft or more. For pier applications the size of isolated square footings ($L_f/B_f = 1$) may range from 15-ft x 15-ft to 30-ft x 30-ft with rectangular footings in the same size range. The thickness of footings for highway bridges can range from 3- to 6-ft.



(a) (b) (c) (d)

Figure C.2-1. Schematics. (a) Continuous footing for a cantilever stemwall, (b) Combined footing for spill-through abutment, (c) Continuous footing for perched abutment, (d) Continuous footing for a cantilever retaining wall.

C.2.1 Use of Spread Footing Abutments on Top of MSE Walls

Often referred to as a "true" bridge abutment, this configuration recognizes the significantly increased bearing resistance offered by the properly constructed reinforced backfill of an MSE wall acting as the founding material for a spread footing. A typical configuration is shown in Figure C.2-2.

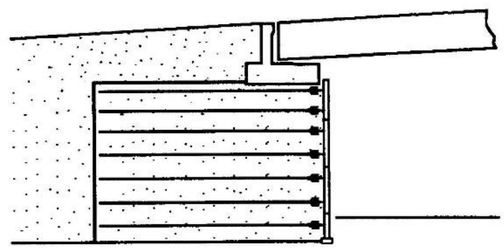


Figure C.2-2. Schematic. "True" bridge abutment with continuous footing on top of a mechanically stabilized earth (MSE) wall.

Where the native ground below the MSE wall prism is not anticipated to experience long-term settlements, the following are the pertinent criteria for the evaluation of spread footings on top of MSE walls (FHWA 2009):

- Factored bearing resistance for strength limit state = 7 ksf
- Service limit state bearing resistance at ½-inch settlement = 4 ksf

These values of bearing resistance are larger than (often double) the values that are conventionally reported by geotechnical specialists for footings on native soils or improved ground. As a matter of fact, these values are similar to those reported for IGMs or weak rock, because the soil is reinforced with mechanical inclusions such as steel or geosynthetics. As noted earlier, typical bearing pressures under footings for highway bridges range from 3 to 6 ksf. With the bearing resistance values noted above, these bearing pressures can be easily accommodated.

Use of true bridge abutments can result in significant cost savings. True bridge abutments also have significant advantages over conventional abutments as follows:

- The proverbial bump at the end of the bridge is eliminated because the footing settles along with the MSE wall in contrast to a deep foundation that does not settle at the same rate as the surrounding MSE walls.
- Approach slabs are not necessary because of the elimination of conditions that would lead to the bump at the end of the bridge. The elimination of approach slabs results in significant cost savings.

C.2.2 Use of Spread Footings for Integral Abutments

Integral and semi-integral abutments are being increasingly used in an attempt to eliminate the maintenance-intensive and troublesome joints and bearings. There are also no abutment seats to collect debris and no damage caused by leaking expansion joints. In the case of integral abutments, the upper portion of the abutment, sometimes called an end-wall, encases the beam ends and is integral with the bottom part of the abutment or pile cap. For semi-integral abutments, the upper portion of the abutment is entirely isolated by expansion material and/or a bearing pad, allowing it to move relative to the bottom portion.

Dunker and Liu (2007) present a summary of various foundation types for integral abutments. Virtually all integral abutments currently rely on flexible deep foundations in the form of H-piles with the weaker axis in the direction of the longitudinal axis of the bridge to permit the necessary thermal movement at the end of the bridge. Such is not the case with semi-integral abutments, where the end-wall can move independently of the bottom portion of the abutment and they can be founded on shallow or deep foundations. Instead of placing the integral abutment on a deep foundation, the Nevada Department of Transportation (NDOT) successfully places it on a spread footing as shown in Figure C.2-3. This proven method should be considered by other agencies to realize the significant synergistic cost savings of integral abutments and spread footings. The Colorado Department of Transportation has had success with a true bridge semi integral abutment on an MSE wall at the Founders/Meadows Bridge over I-25 (Abu-Hejleh et al., 2001).

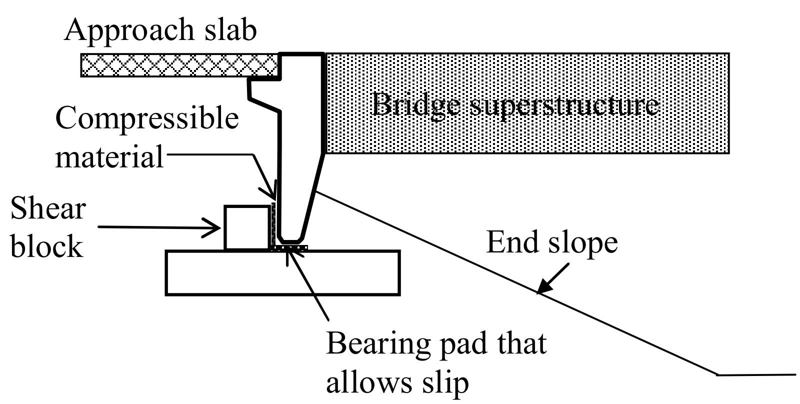


Figure C.2-3. Schematic. "Diaphragm-on-footing" type integral abutment on spread footing in an end slope used by Nevada Department of Transportation.

C.2.3 Quality Control and Drainage

The two applications of spread footings described in sections C.2.2 and C.2.3 involve construction of spread footings on or in compacted fill materials. Improperly compacted fill materials will have variable stiffness that may result in uneven settlement and rotation of a footing founded in or on the fill. In addition, improper drainage will permit ingress of moisture into the subsoil under the footing thereby leading to softening of the subsoil and associated settlement problems. Therefore, for all applications of spread footings on or in compacted fill materials, it is critical to ensure proper quality control of compaction during construction and implementation of proper surface and subsurface drainage measures. It should be noted that these recommendations do not constitute special or extraordinary requirements, but are simply the best practice for successful applications of the spread footings to MSE wall and integral abutments. Further discussion on the recommended properties of compacted fill materials is provided in Section C.3.3 of this appendix.

C.3 FEASIBILITY EVALUATION

As indicated in Section C.2, spread footings can be used to support bridge elements a number of different ways. In all such uses the foundation soils must be competent enough to support the bridge loads without undergoing catastrophic soil shear failure or excessive deformations that would result in unacceptable settlements. Section C.3 first discusses the unfavorable situations in which spread footings are not viable or should not be considered. Then, as a prelude to the discussion of the favorable situations for spread footing applications, a summary is presented of the subsurface investigations required to perform a proper evaluation of the load-displacement characteristics of the footing-soil system. As indicated in Chapters 3 and 4 of this report, the load-displacement characteristics are of primary importance in assessing the feasibility of using spread footings for a given application. Finally, the use of spread footings on competent native soils, improved soils, and compacted fill materials is discussed in detail. Thus, this section presents information for the bridge designer to evaluate the feasibility of using spread footings on soils for highway bridges.

C.3.1 Unfavorable Situations

Similar to any foundation type, spread footings have limitations with respect to their applications for highway bridges. For a feasibility evaluation, the following two key points must be understood:

1. Since the base of a spread footing is generally within a few feet of the finished grade, consideration should be given to all external conditions that can alter the conditions within the depth of embedment.

2. As noted in Section C.1, the DOSI below the footing is a function of the footing width. Therefore, any adverse geologic conditions or potential future changes within the DOSI should be considered while evaluating the feasibility of spread footings.

Based on these considerations, spread footings are not feasible for the following situations:

- Stream crossings where scour is a concern
- Liquefiable soils
- Deep collapsible soil deposits
- Soils with swell pressure larger than footing pressure
- Karstic deposits
- Permafrost areas
- Deep frost penetration
- Areas of tidal fluctuations
- Possibility of future unsupported excavations below the base of the footing
- Significant long-term settlements that would affect the structural integrity of the bridge

Other conditions which might limit the use of spread footings are as follows:

- Limited right-of-way which would control the size of the footing
- Excavation of contaminated soils
- Significant dewatering for cases where the water table is within the depth of embedment
- Situations where groundwater may rise within the DOSI in the future (e.g., in areas of groundwater recharge or adjacent significant unlined water bodies/facilities, which may leak)

C.3.2 Subsurface Investigations

Once the unfavorable situations are screened for a given project and a spread footing system has been identified as a potential candidate for foundations of a bridge structure, then an adequate subsurface investigation program must be implemented to evaluate the continued feasibility of the spread footing system. As noted in Section C.1, a "spread footing" spreads the applied load over the area of contact between the footing and the soil and eventually into the subsurface soils beneath the footing. This is in contrast to deep foundations where a substantial portion of load is commonly supported by side friction, which is concentrated in the immediate vicinity of the deep foundation element. The contact pressure under a spread footing is spread vertically and horizontally below the base of the footing with its magnitude decreasing with depth in multiples of the footing width. The performance of a spread footing will be directly affected by the stiffness characteristics of the subsurface soils within the DOSI. Therefore, to evaluate the feasibility of spread footings particular care should be taken in evaluating the subsurface soil characteristics within the DOSI. This report assumes that a minimum standard of investigations as outlined in FHWA (2006) and Section 10 of AASHTO (2007 with 2009 Interims) is rigorously implemented. For critical highway bridges, such as those on life-line routes (e.g., interstate highways), consideration may be given to increasing the level of field and laboratory investigations particularly in those areas where prior information and/or experience is poor or not available.

Performing an adequate subsurface investigation program does not guarantee proper performance of a spread footing. As with any foundation system, it is incumbent on the owner to implement an adequate construction inspection and monitoring program. Appendix D provides information on such a program and this report assumes that the minimum standard of care noted in Appendix D is implemented. Thus, the aspects of subsurface investigations, construction inspection and monitoring should be part of the feasibility evaluation of spread footings. It should be noted that these recommendations do not constitute special or extraordinary requirements, but are simply best practice for routine applications.

C.3.3 Categories for Use of Spread Footings on Soils for Highway Bridges

Once the subsurface investigations have been performed, the stiffness of the soils within the DOSI should be evaluated to verify if the spread footing system can be feasible with respect to the tolerable deformations established by the structural specialist. In this context the use of spread footings can be categorized based on the types of ground as follows:

- Spread footings on competent native soils – in this category the stiffness of the native soils is deemed to be adequate to support the loads within tolerable settlements.
- Spread footings on improved in situ soils – in this category it is deemed that the in situ soils need mechanical and/or chemical treatment to improve their stiffness so that the loads can be supported within tolerable settlements.
- Spread footings on compacted engineered embankment fill materials – in this category the footings are supported in fills constructed of select soils placed under engineered conditions.

C.3.3.1 Spread Footings on Competent Native Soils

Existing native soils generally suitable for the use of spread footings include dense to very dense sands, gravels, and sand and gravel mixtures; medium- to highly-cemented silty sands that are not susceptible to hydro-compaction; glacial tills and highly overconsolidated lean clays. Spread footings are feasible in such soils provided they exist to a depth below the base of the footing at least equal the DOSI as discussed in Section C.1. To assure uniformity below the base of the footing, these soils are generally scarified at least 12-inches at the foundation base elevation and compacted to a minimum relative compaction of 95% based on Modified Proctor compaction energy. Spread footings are also feasible for conditions where native soils are underlain at shallow depth by competent rock that is not subject to severe degradation by weathering, e.g., shales. In such cases the overlying soil is usually excavated and the spread footings are founded on the competent rock. The use of spread footings for the support of bridge structures in the soils described above is feasible only if there is no danger of significant erosion or scour.

C.3.3.2 Spread Footings on Improved In-Situ Soils

In general, in-situ soils that are not suitable for the use of spread footings include: alluvial soils (fans), fluvial soils (flood plain), colluvial soils, collapse-susceptible soils including loess, expansive soils, loose sands and silts, soft or organic clays, formations where large near-surface voids are present (karst topography, sinkholes), former landfills, disposal sites for building rubble and construction debris and other non-engineered fills. In order to carry the loads imposed by bridge structures the load-displacement characteristics of such soils must be improved mechanically or chemically.

The feasibility of using spread footings on improved in-situ soils generally depends upon the thickness of the soils that need to be improved and the depth to which the improvement can be undertaken economically. There is a practical limit to this depth that depends upon a number of factors including footing size, loading, the ability of the unsuitable soils to be stabilized, the economy of the stabilization method versus the cost of bypassing the poor soils entirely by deep foundations, etc. The methods most commonly used to stabilize near surface soils so that shallow spread footings are a feasible option for bridge abutments and piers are:

- surcharging,
- vibrocompaction,

- deep dynamic compaction,
- excavation and recompaction,
- excavation with removal and replacement, and
- chemical stabilization by mixed-in-place admixtures such as lime and Portland cement.

Information on the above methods for ground stabilization can be found in FHWA (2006a).

C.3.3.3 Spread Footings on Compacted Engineered Embankment Fill Materials

Figure C.3-1 shows a comparison of stress distribution under earth embankment and pier loads. As can be clearly seen, the weight of the embankment is predominant except at very shallow depths. Based on this comparison, geotechnical specialists have long recognized the feasibility of placing footings on engineered fills. If adequate time is allowed for the foundation soils to settle under the fill load, subsequent application of a smaller structural load will result in negligible settlement of the structure. In bridge construction, common practice is to build the approach embankment excluding the area to be occupied by the abutment and allow settlement to occur prior to abutment construction. Details of the evaluation of settlement of approach embankment fills are presented in FHWA (2006).

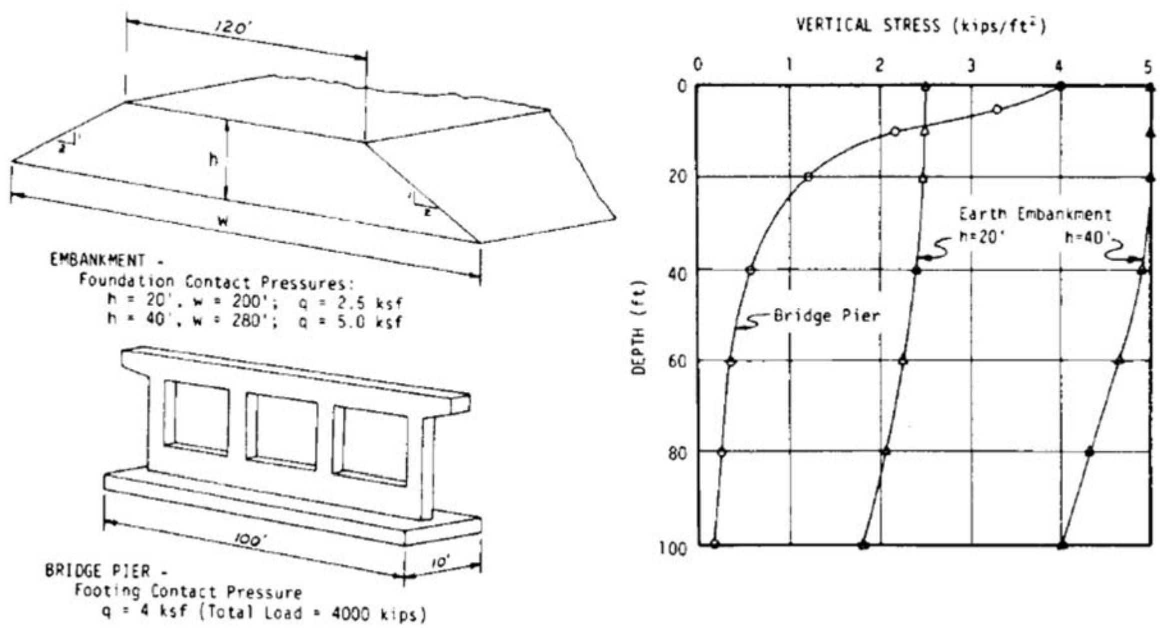


Figure C.3-1. Figure. Computation of vertical stresses beneath center lines of bridge piers and earth embankments (NCHRP, 1971).

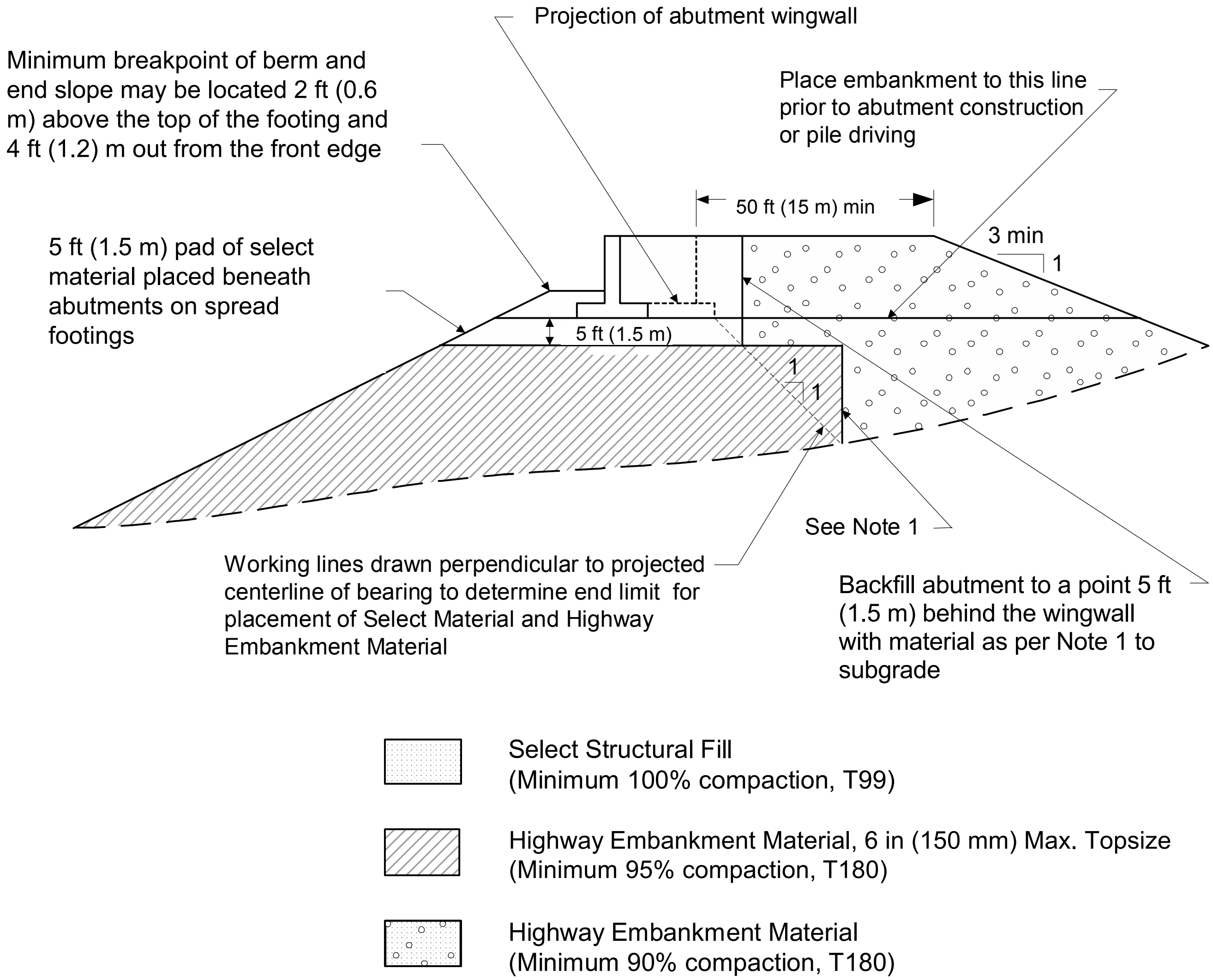
Compacted structural fill materials used for supporting spread footings should be a select and specified material that includes sand- and gravel-sized particles. Furthermore, the fill material should be compacted to a minimum relative compaction of 95% based on Modified Proctor compaction energy. The fill material should extend through the entire embankment below the footing. Suggested approach embankment details for spread footing applications are shown in Figure C.3-2. The soil zoning shown in Figure C.3-2 also controls the internal embankment deformations (FHWA, 2006). FHWA (2002c) notes that the Washington State Department of Transportation (WSDOT) successfully used the gradation listed in Table 4.1 to design spread footings for the I-5 Kalama Interchange. WSDOT limited the maximum bearing pressures to 6 ksf. The measured settlements were found to be less than 1.5-inches within the fill.

In addition to WSDOT, the Nevada Department of Transportation (NDOT) commonly uses spread foundations founded within compacted structural embankment fills (FHWA, 2006). A successful case history of the use of an abutment spread footing on a 130-ft high welded wire reinforced rockfill embankment is also presented in Anderson and LaFronz (2007).

Direct shear testing of materials such as those described in Table C.3-1 is not practical on a project-by-project basis since such materials require large specialized test equipment. Therefore the design of spread footings on compacted sand and gravel is based on a combination of experience and the results of infrequent large-scale laboratory testing on specified gradations of select fill materials. Material specifications are then developed based on the specified gradations to ensure good quality control during construction. This procedure helps ensure that the conclusions from the laboratory tests are valid for the construction practices used to place the fills.

Table C.3-1. Typical specification for compacted structural fill material used by WSDOT (FHWA, 2006).

Sieve Size	Percent Passing
4” (100 mm)	100
2” (50 mm)	75 – 100
No. 4 (4.75 mm)	50 – 80
No. 40 (0.425 mm)	30 max
No. 200 (0.075 mm)	7 max
Sand Equivalent (ASTM D2419)	42 min



Note 1: Highway embankment material and select material shall be placed simultaneously of the vertical payment line

Figure C.3-2. Schematic. Suggested approach embankment details for spread footing application (FHWA, 2006).

C.4 HORIZONTAL DEFORMATIONS

Chapters 2 to 5 of the report concentrated on the issue of vertical deformations or what is commonly called settlement. Although settlements are important, horizontal deformations can often cause more severe and widespread damage than an equal magnitude of vertical movements (FHWA, 1985). For spread footings, horizontal deformations can occur because of the following two mechanisms:

- lateral loads
- lateral squeeze of the foundation soil

C.4.1 Horizontal Deformations Due To Lateral Loads

Assuming that adequate drainage features are in-place and functioning satisfactorily, the primary source of lateral loads at abutments is earth fill and any surcharges behind the abutment. If appropriate drainage is not provided, then additional lateral loads can occur due to the build-up of hydrostatic pressures and frost action. Assuming that the abutment walls are free to displace laterally and the foundation soils are competent, the minimum movement that can be anticipated for design is the movement required to mobilize the active earth pressure. Such lateral movements can occur by sliding at the base of the spread footing and/or by rotation of the abutment stem wall. In the later case, the spread footing is subjected to rotation (tilting). In either case, the primary concern is the horizontal movement at the superstructure level.

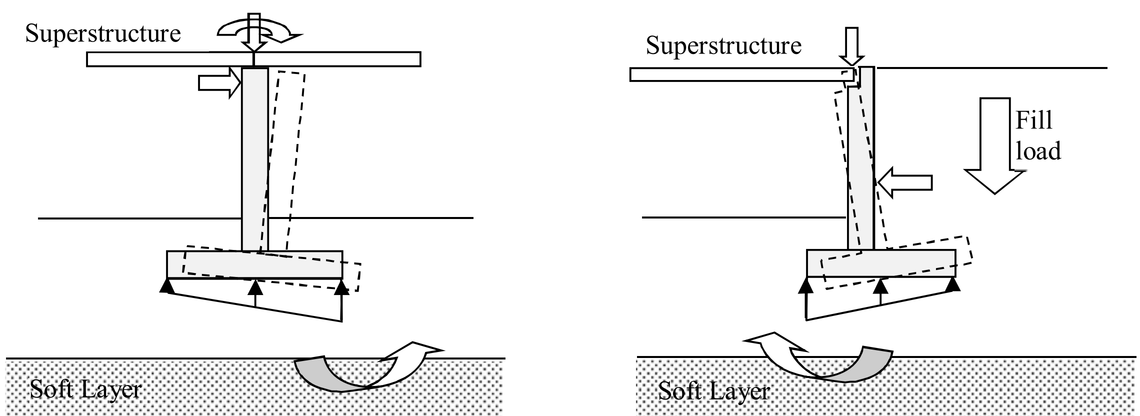
Generally granular fills are used at abutment locations. For these types of materials, the typical horizontal movements that can be anticipated are in the range of 0.001 to 0.004 times the height of the abutment wall. Thus, for example, if the abutment is 20-ft tall, horizontal movements in the range of ¼- to 1-inch may be anticipated. In a general construction sequence, the earth fill behind the abutment is substantially complete prior to placement of the superstructure. In this case, the horizontal movement at the superstructure level is virtually complete and should not be of concern assuming that the vertical joint between the end of the superstructure and the abutment back-wall was designed properly to accommodate the movement. However, the lateral movements caused by lateral loads due to surcharges, such as live loads and thermal effects, experienced by the abutment after the placement of the superstructure should be considered in the design of the bridge structure.

At pier locations, the primary source of lateral loading is from thermal effects, braking forces and forces due to unequal spans if any exist on either side of the pier. Assuming that the pier substructure has sufficient structural resistance, these lateral loads are primarily resisted by sliding resistance at the base of the spread footing. Where the foundation soils are weak in shear strength, e.g., fine-grained clayey soils, the interface shear strength may be small which increases the potential for sliding. Once the interface shear strength is overcome by the horizontal forces, large sliding movements can occur that can cause significant damage to the superstructure. Where such conditions exist, consideration should be given to excavating the weaker soils and replacing them with select granular fill materials. Consideration should also be given to incorporating a shear key at the base of the spread footing.

C.4.2 Horizontal Deformations Due To Lateral Squeeze

When the width of the footing is larger than the thickness of the compressible layer beneath it or when there is a finite soft layer within the DOSI below the loaded area, significant lateral stresses and associated lateral deformations can occur within the soil. These deformations are transmitted to the superstructure. Figure C.4-1 shows schematics of such movements at pier and abutment locations. The lateral squeeze phenomenon is due to an unbalanced load at the surface of the soft soil. The lateral squeeze behavior may be: (a) short-term undrained deformation that results in horizontal deformation from a local bearing resistance type of failure, or (b) long-term drained, creep-type deformation. Creep refers to the slow deformation of soils under sustained loads over extended periods of time and can occur at stresses well below the shear strength of the soil. The bridge abutment may tilt forward or backward depending on a number of factors including the relative configuration of the fill and the abutment, the relative stiffness of the footing and the soft deposit, the strength and thickness of the soft layer, and the rate of construction of the fill.

In all cases, these types of movements will likely occur only if inadequate subsurface investigations were performed that missed the presence of the softer layers within the DOSI. Where such softer layers exist within the DOSI, spread footings should not be used without some ground improvement.



(a) **(b)**

Figure C.4-1. Schematic. Horizontal deformations at the superstructure level due to settlement and rotation of spread footings for the case of soft layers within DOSI. (a) at piers, (b) at abutments.

APPENDIX D – CONSTRUCTION INSPECTION, MONITORING AND INSTRUMENTATION

Construction inspection requirements for spread footings are similar to those for other concrete structures. In some cases, agencies may have inspector checklists for construction of shallow foundations. Table D-1 provides a summary of construction inspection check points for spread footings on soils. Throughout construction, the inspector should check submittals for completeness before transmitting them to the engineer.

D.1 STRUCTURAL FILL MATERIALS

Fill requirements should be strictly adhered to because the fill must perform within expected limits with respect to strength and, more importantly, within tolerance for differential settlement. Sometimes the area for construction of the fill is small, such as behind abutments and wingwalls. In such situations, the use of hand compactors or smaller compaction equipment may be necessary.

When the construction of structural fills that will support shallow foundations is being monitored, particular attention should be paid to the following items:

- The material should be tested for gradation and durability at sufficient frequency to ensure that the material being placed meets the specification.

- The specified level of compaction must be obtained in the fill. Testing, if applicable, should be performed in accordance with standard procedures and at the recommended intervals or number of tests per lift.

If a surcharge fill is required for pre-loading, it should be verified that the unit weight of the surcharge fill meets the value assumed in the design.

D.2 MONITORING

The elevations of constructed foundations should be checked before and after the structural load is applied. The measurements made at those times will serve as a baseline for the long-term monitoring of the bridge. Subsequently, additional survey measurements should be made to confirm satisfactory performance or to identify whether potentially harmful settlements are occurring. If the fill was constructed over soft compressible soils, it may be important to check that the fill settlements are complete before foundation construction begins. Instruments such as settlement plates, horizontal inclinometers, or other types of instrumentation are typically installed in such cases. In some instances, the lateral displacement potential can be greater than the vertical movements; therefore, if conditions warrant, monitoring may also include complete survey coordinates and possibly more accurate instrumentation. Instrumentation is discussed in Section D.3.

Table D-1. Inspector responsibilities for construction of spread footings (FHWA, 2006).

CONTRACTOR SET UP
• Review plans and specifications. • Review contractor's schedule. • Review test results and certifications for pre-approved materials, e.g., cement, coarse and fine aggregate. • Confirm that the contractor's stockpile and staging area are consistent with locations shown on plans. • Discuss anticipated ground conditions and potential problems with the contractor. • Review the contractor's survey results against the plans.
EXCAVATION
• Verify that excavation slopes and/or structural excavation support is consistent with the plans. • Confirm that limits of any required excavations are within right-of-way limits shown on the plans. • Confirm that all unsuitable materials, e.g., sod, snow, frost, topsoil, soft/muddy soils, are removed to the limits and depths shown on the plans and the excavation is backfilled with properly compacted granular material. **The in-place bearing stratum of soil or rock should be checked to verify the in-situ condition and the degree of improvement achieved by the contractor's preparation approach. Some soil types can become remolded and weakened from disturbance. If the conditions deviate from those anticipated in the geotechnical report and/or the plans and specifications, the geotechnical specialist should be consulted to determine if additional measures are necessary.** • Confirm that leveling and proof-rolling of the foundation area is consistent with the requirements of the specifications. Probing is recommended for verification of subgrade. • Confirm that contractor's excavation operations do not result in significant water ponding. • Confirm that existing drainage features, utilities, and other features are protected. • Identify areas not shown on the plans where unsuitable material exists and notify engineer.
SPREAD FOOTING
• Approve footing foundation condition before concrete is poured. • Confirm reinforcement strength, size, and type consistent with the specifications. • Confirm consistency of the contractor's outline of the footing (footing size and bottom of footing depth) with the plans. • Confirm location and spacing of reinforcing steel consistent with the plans. • Confirm water/cement ratio and concrete mix design consistent with the specifications. • Record concrete volumes poured for the footing. • Confirm appropriate concrete curing times and methods as provided in the specifications. • Confirm that concrete is not placed on ice, snow, or unsuitable ground. • Confirm that concrete is being placed in continuous horizontal layers and that the time between successive layers is consistent with the specifications.
POST INSTALLATION
• Verify pay quantities.

D.3 INSTRUMENTATION

The three primary parameters of interest with respect to performance monitoring of a spread footing are as follows:

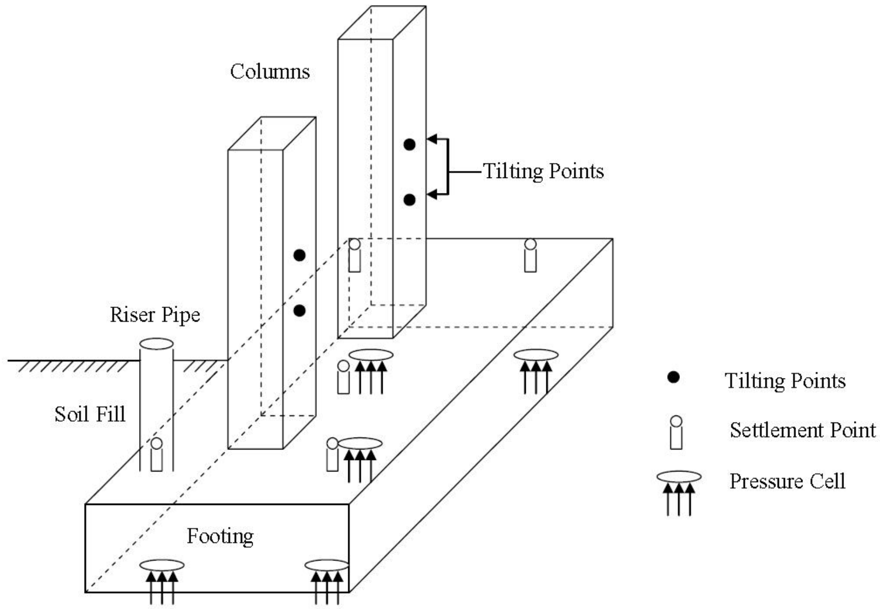
- Settlement
- Bearing pressure
- Rotation (or tilting)

A typical layout of instrumentation is shown in Figure D-1(a). The instrumentation needed to achieve the measurements of the three parameters listed above is as follows (after Sargand and Masada, 2006 and FHWA 1987):

- Settlement – Settlement monitoring points are placed over the top surface of the footing to allow vertical displacement measurements with respect to a stationary benchmark located nearby. A minimum of 5 monitoring points are recommended with one point located at or near the center of the footing and the remaining four points located symmetrically near the footing corners. The monitoring points can be in the form of stainless steel eyebolts anchored into the footing and encased in a 4-inch diameter PVC riser pipe. Survey methods for leveling can be used to detect the vertical displacement at each point with respect to the stationary benchmark.

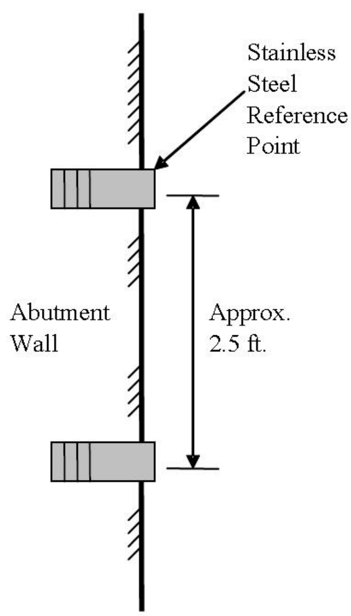
- Bearing pressure – Pressure cells are positioned at the base of the footing to measure the magnitude and distribution of the contact pressure. A minimum of five pressure cells are used and their position is similar to that of the settlement points except that the cells are located at the base of the footing. If the footing supports more than one column then at least one pressure cell should be located directly under the center of each column.

  The pressure cell should be of the vibrating-wire type with a minimum diameter of 9-inches. Each pressure cell is typically precast in a concrete block which is 12” wide x 24” long x 2” thick. This precasting is necessary to prevent the cell from being disturbed during footing construction. In the field, each precast pressure cell block is placed carefully at the predetermined location with the sensing disk pressed against a 2” thick compacted sand layer. A nonwoven geotextile is inserted between the sand layer and the bearing soil to keep the two dissimilar materials separate. The pressure cell in each block should be carefully calibrated before the concrete for the footing is placed. The range of each pressure cell should be from 0 to 100 psi with a sensitivity of 0.1 psi. Provisions should be made for extending the pressure cell wires out of the footing to a convenient and safe location where a readout unit can be attached to obtain the measured values.

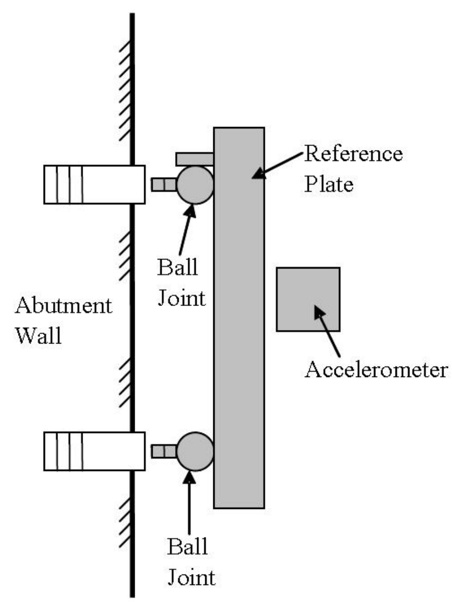


(a) General layout of instrumentation for monitoring settlement, bearing pressure and tilting

(b)



(b) Grouted reference points



(c) Positioning of reference plate and sensor

Figure D-1. Schematic. Typical instrumentation setup for a spread footing foundation. (Sargand and Masada, 2006)

- Rotation (or tilting) – Rotation of the spread footing is inferred from the tilt of the substructure unit (pier column or abutment) above the footing. Tilting is measured by accelerometers attached to the substructure unit. Tilt stations are established on the substructure unit as shown in Figure D-1(a). At each monitoring station, two stainless steel reference points are grouted at least 2-inches deep into the substructure unit approximately 2.5-ft apart vertically as shown in Figure D-1(b). For taking tilt measurements, a stainless steel ball joint is screwed into each reference point and a reference plate is set to rest freely against the ball joints. An accelerometer is then attached to the side of the reference plate as shown in Figure D-1(c). Data is collected from a readout box that is connected to the accelerometer. The sensor should have a range of ±30° and a sensitivity of 0.003°.

A full suite of readings for the three parameters should be obtained at every significant construction-point as noted in Chapter 4. Where deformations are anticipated to occur after the end of construction, a suitable frequency of measurements should be established on a project-specific basis.

APPENDIX E – LRFD GUIDANCE FOR SPREAD FOOTINGS

The Load and Resistance Factor Design (LRFD) methodology is currently being implemented across the United States, particularly for federally funded transportation facilities. The American Association of State Highway and Transportation Officials (AASHTO) recently released the 4th Edition of the LRFD Bridge Design Specifications in 2007. Since October 1, 2007, the AASHTO-LRFD approach must be fully implemented by states seeking federal funding for new transportation projects. It is important that the structural and geotechnical specialists involved in the design of such transportation facilities properly understand the basics of the LRFD approach as included in AASHTO's specifications (AASHTO, 2007 with 2009 Interims).

The purpose of Appendix E is to present detailed guidance for the analysis of spread footings in the LRFD framework. To achieve this purpose the appendix is organized as follows:

- Limit states and spread footings (Section E.1) – this section presents a summary of the primary limit states in the LRFD framework and the categories of the various failure modes for spread footings with respect to the LRFD-based limit states.
- Spread footing design based on geotechnical considerations (Section E.2) – this section introduces and discusses the concept of the bearing resistance chart to evaluate the strength and service limit states for spread footings.
- Spread footing design using LRFD methodology (Section E.3) – this section presents and discusses a detailed design process flowchart for the design of spread footing using LRFD methodology.
- Numerical example for spread footing analysis using LRFD methodology (Section E.4) – this section presents the calculations for a comprehensive numerical example using the various concepts introduced in the report and described in the previous sections of Appendix E.

E.1 LIMIT STATES AND SPREAD FOOTINGS

In the AASHTO-LRFD framework, there are four distinct limit states: (a) strength (or ultimate) limit states, (b) service limit states, (c) extreme event limit states and (d) fatigue limit states. For most routine cases, the design of a bridge or a component is generally governed by either the strength or the service limit states. These two commonly analyzed limit states are briefly described below:

- Strength (or ultimate) limit states are limit states that pertain to structural safety and the loss of load-carrying capacity. These limit states may be reached through either geotechnical or structural failure. Evaluation of strength limit states is based on inelastic behavior of the structure, which is accomplished by using increased or factored loads, and on modification of soil behavior, which is accomplished by using reduced or factored strengths. From a geotechnical viewpoint, strength limit states are reached when they involve the partial or total collapse of the structure due to sliding, bearing failure, etc. Figure E.1-1 shows the three strength limit states for spread footings. For well-designed structures strength limit states have an extremely low probability of occurrence.

- Service limit states are the limiting conditions affecting the function of the structure under expected service conditions. Thus, service limit states address serviceability and include conditions short of complete collapse that may restrict the intended use of the structure, e.g., excessive total or differential settlements, cracking, local damage, poor ride quality, etc. Evaluation of service limit states is usually performed by using expected service loads, nominal strengths and elastic analyses. Compared to strength limit states, the service limit states have a higher probability of occurrence, but, if exceeded, involve less likelihood of loss of life. Figure E.1-2 shows the two service limit states for spread footings. An associated service limit state that involves horizontal movement and rotation (due to coupling with vertical and horizontal movements) is also possible.

Due to the rather large size of spread footings for highway bridges, deformations at service limit states are generally the controlling factors in footing design. Finally, extreme event limit states are also evaluated by considering each of the failure modes shown in Figure E.1-1 for strength limit state. Settlements are not a consideration at extreme event limit state since the expectation is to preserve life and not necessarily the serviceability of the structure.

(a) (b) (c)

Figure E.1-1. Schematic. Strength limit states for spread footings. (a) Sliding, (b) Limiting eccentricity, (c) Bearing resistance.

(a) (b)

Figure E.1-2. Schematic. Service limit states for spread footings. (a) Settlement, (b) Overall stability.

E.2 SPREAD FOOTING DESIGN BASED ON GEOTECHNICAL CONSIDERATIONS

The design of a spread footing based on geotechnical considerations is a two part process. First the factored soil bearing resistance must be established to ensure stability of the foundation and determine if the proposed structural loads can be supported on a reasonably sized foundation. Second, the amount of settlement due to the actual structural loads must be calculated and the time of occurrence estimated. Experience has shown that settlement is usually the controlling factor in the decision to use spread footings for highway bridges. This is not surprising since structural considerations usually limit tolerable settlements to values that can be achieved only on competent soils not prone to a bearing resistance failure. Thus, the **factored net bearing resistance** of a spread footing is defined as the lesser of:

- The applied stress that is equal to the net nominal bearing resistance (q_{nn}) multiplied by a suitable resistance factor (ϕ_b). This criterion is based on a **strength limit state** as discussed in Section E.1.

  or

- The applied stress that results in a specified amount of settlement (S_{tol}). S_{tol} is generally dictated by structural considerations and/or institutional culture. This criterion is based on a **service limit state** as discussed in Section E.1.

Both of these criteria are a function of the least lateral dimension of the footing, typically called the effective footing width and designated as B'_f. The effective footing width is defined as the total footing width, B_f, minus two times the eccentricity in the direction of the width of the footing, e_B, (Meyerhof, 1953). Thus, $B'_f = B_f - 2e_B$ and, if there is no load eccentricity, $B'_f = B_f$.

The influence of effective footing width on factored net bearing resistance and settlement is shown conceptually in Figure E.2-1. The factored net bearing resistance of a footing is usually limited by soil shear-failure considerations for narrow footing widths as shown in Zone A in Figure E.2-1. As the footing width increases, the factored net bearing resistance is generally controlled by the value of (S_{tol}) with respect to the settlement potential of the soils supporting the footing, as expressed by the contours of net bearing resistance for given settlements shown in Zone B in Figure E.2-1. The contour of net bearing resistance for a given settlement varies with effective footing width because the DOSI is a function of footing width as discussed in Chapter 3.1 and Section C.1 of Appendix C. Stated another way, for a given applied load, as the footing width increases the applied stress decreases; therefore, the stress increase experienced by the soil below the footing may also decrease. However, the effect of the applied stress will extend deeper below the footing base, i.e., the DOSI increases. Therefore, a stress increase will be experienced by soils that had not been influenced previously so that settlements may actually increase depending on the types of soils within the deeper DOSI. The result of this interrelationship is shown schematically in Zone B in Figure E.2-1.

The concept of decreasing factored bearing resistance with increasing footing width for the settlement-controlled cases is an important concept to understand. In such cases, the factored net bearing resistance is the value of the applied stress at the footing base that will result in a given settlement. Since the DOSI increases with increasing footing width, the only way to limit the settlements to a certain desired value (S_{tol}) is by reducing the applied stress. The more stringent the settlement criterion (S_{tol}) the less the stress that can be applied to the footing, i.e., the factored net bearing resistance is correspondingly less. This concept is illustrated in Figure E.2-1, which shows that decreasing the settlement, i.e., going from 3S to 2S to S decreases the factored bearing resistance at a given footing width. More details regarding the process for development of charts similar to that shown in Figure E.2-1 can be found in FHWA (2006). A numerical example of the use of the bearing resistance chart is presented in Section E.4 of this appendix.

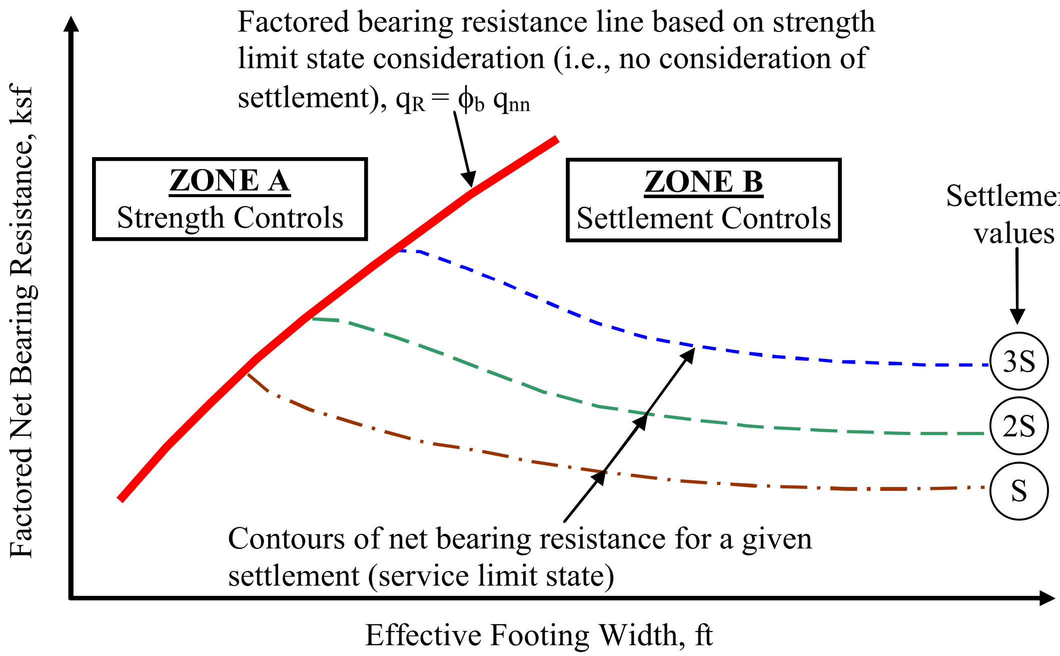


Figure E.2-1. Schematic. Bearing resistance chart showing strength limit state and service limit state criteria as a function of effective footing width.

While the format of the bearing resistance chart shown in Figure E.2-1 is convenient for the purpose of establishing the design footing width as demonstrated in Section E.4 of this appendix, the chart can be easily re-organized in the format of load settlement curves for a range of footing widths to permit a rational evaluation of deformations at the service limit state in relation to the strength limit state and extreme event limit state. For a given footing width, Figure E.2-2 shows the load-settlement (Q-S) curve that was first introduced in Figure 2-1 in Chapter 2. In Figure E.2-2, two additional points on the Q-S curve are introduced; these are points S and F. Point S corresponds to the service load (Q_S) at the appropriate service limit state while Point F corresponds to the factored load (Q_F) at the appropriate strength limit state. By definition, the following conditions are necessary:

- Condition 1: $Q_F > Q_S$.
- Condition 2: Point F corresponds to the factored bearing resistance (the steeply ascending solid line in Figure E.2-1) because otherwise stability at the strength limit state will not be satisfied.
- Condition 3: Point N corresponds to the condition of a bearing resistance failure. Thus, it represents the nominal bearing resistance for a footing of effective width B'_f.
- Condition 4: Based on Conditions 2 and 3, for stability of the footing, $Q_F = \phi_b(Q_N)$ where ϕ_b is the bearing resistance factor from Section 10 in AASHTO (2007 with 2009 Interims) and Q_F and Q_N are as defined before.
- Condition 5: Since Point N corresponds to the nominal bearing resistance, the deformation at this point can be expected to occur at the extreme event limit state since ϕ_b= 1 and nominal resistance is used at this limit state.

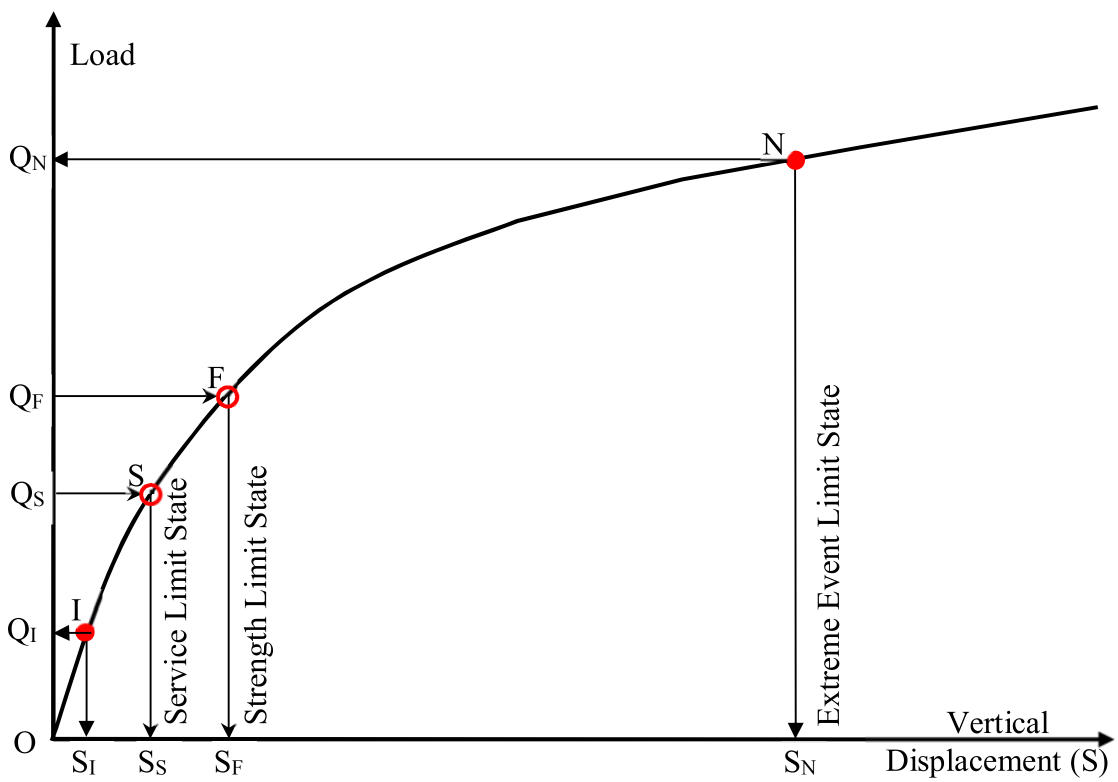


Figure E.2-2. Schematic. Axial load versus vertical displacement curve for a spread footing having a given effective width B'_f showing significant points of interest with respect to the LRFD approach..

By using the concepts introduced in Figure E.2-2, the data in Figure E.2-1 can be re-plotted as a series of stress-settlement curves for a range of footing widths as shown in Figure E.2-3. Basically, the plots in Figure E.2-3 represent vertical sections at discrete footing widths through the various curves in Figure E.2-1. This construction merely takes conventional computations one step further and does not require any advanced computational skills. Section E.4 of this appendix shows the development of this format of the bearing resistance chart by using a numerical example. The advantage of this format is that it readily allows an evaluation of settlements that will be experienced at various stages of bridge construction. Since a range of effective footing widths is used in Figure E.2-3, the Y-axis is in terms of stress (resistance) units. If the chart were to be developed for a given effective footing width then the Y-axis can be expressed in terms of stress units or force units. In the latter case, the Q-S curve up to Point F shown in Figure E.2-2 will occur. The key point is that by manipulating the same load-settlement data, a number of different design aspects can be explored in a rational manner. Before the use of curves such as those shown in Figure E.2-3 is demonstrated, the design process for spread footings using the LRFD approach will be discussed so that the need for close collaboration between the geotechnical and structural specialists can be better understood.

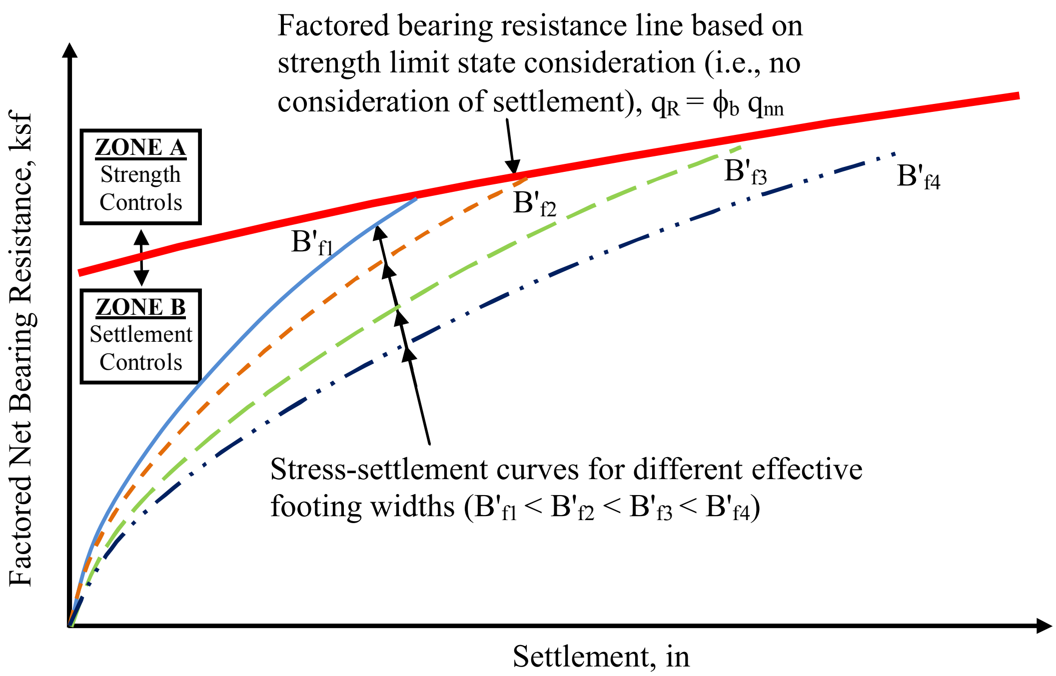


Figure E.2-3. Schematic. Alternative bearing resistance chart in the format of stress-settlement curves for a range of effective footing widths and settlements.

E.3 SPREAD FOOTING DESIGN USING LRFD METHODOLOGY

The design of a spread footing in the LRFD approach is done in a systematic manner that accounts for all of the limit states shown in Figures E.1-1 and E.1-2 while considering all applicable load combinations. The design process for spread footings using the LRFD approach is discussed in this section and a design process flow chart is presented and explained.

Design of a spread footing must provide adequate resistance against geotechnical and structural limit states, i.e., "failure" modes. The geotechnical limit states are identified in Section E.1 of this appendix and include the following:

- Strength limit state
    - Bearing resistance
    - Limiting eccentricity
    - Sliding
- Service limit state
    - Settlement
    - Global stability
- Extreme Event limit state
    - Bearing resistance
    - Limiting eccentricity
    - Sliding

The structural design includes consideration of limit states for the following:

- Flexural resistance (strength limit)
- Shear resistance (strength limit)
- Crack control (service limit)

All the above limit states are analyzed in a systematic manner as shown by the flow chart in Figure E.3-1. The first 8 steps establish the necessary geotechnical and structural information to evaluate the various limit states. As noted in sections E.1 and E.2, the size of the footings for highway bridges is large and is almost always governed by tolerable settlement, S_{tol}. Therefore, the first limit state to be evaluated is the service limit state as shown in Step 9 in the flow chart. After the plan dimensions of the footing are established based on settlement (service limit state) considerations in “Decision 1,” they are verified with respect to the various failure modes in strength limit states as shown in Steps 11 through 14 in the flow chart. If the footing is found to be acceptable from the viewpoint of strength limit state, then the dimensions are verified for the extreme event limit state as shown in Step 15 in the flow chart. After the evaluation of the extreme event limit state, the design process moves on to structural design, which establishes the thickness of the footing and the size and placement of the reinforcement within the footing.

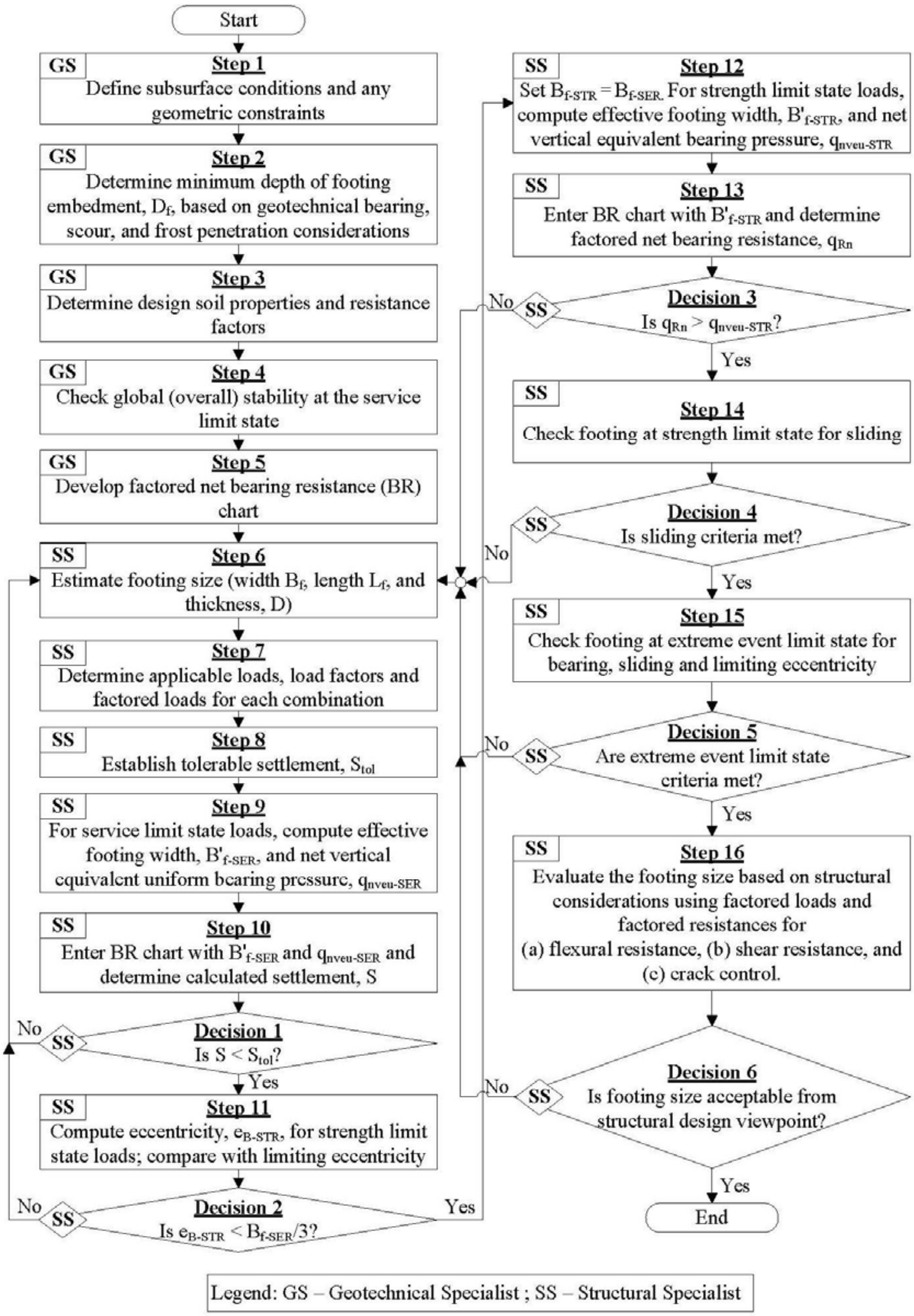


Figure E.3-1. Flowchart. Design process for spread footings using LRFD approach.

In terms of responsibilities, the geotechnical specialist is responsible for performing the tasks in Steps 1 to 5. The remaining steps are best performed by a structural specialist since s/he has the knowledge of the various loads and load combinations. It is strongly recommended that the structural specialist interact with the geotechnical specialist as necessary to obtain more information rather than making assumptions.

The geotechnical and structural aspects of the design of spread footings are included in Section 10 (Foundations) and Section 5 (Concrete Structures) of AASHTO (2007 with 2009 Interims).

E.4 NUMERICAL EXAMPLE WITH DETAILED CALCULATIONS

The bearing resistance chart developed by the geotechnical specialist provides the structural specialist with a powerful tool for studying the interrelationships among effective footing widths, uniform bearing pressures (or resistances) and settlements. The procedure for developing a bearing resistance chart is found in FHWA (2006). This section of Appendix E presents a step-by-step procedure that the structural specialist should follow when using such a chart and illustrates the procedure through an example problem based on an actual bridge in Arizona. The presentation herein assumes that the reader is familiar with AASHTO (2007 with 2009 Interims) and the general procedures for spread footing analysis. Table E.4-1 presents the terminology and notation used in the example problem that follows.

<u>Example Problem Statement:</u>

For a 2-span post-tensioned box girder viaduct bridge located in a non-seismic zone in Arizona, the abutments will be founded on spread footings similar to the configuration shown in Figure C.2-1(a). The span lengths are 105-ft and 123-ft. The length, L_f of the abutment footings is 150-ft. The minimum embedment depth of embedment, D_f, of the footing base is 6-ft. Using the procedures in FHWA (2006), the geotechnical specialist (NCS, 2007) developed a bearing resistance chart based on site-specific subsurface data. The chart is shown in Figure E.4-1. The pertinent parameters for the example abutment footing are included in Tables E.4-2 and E.4-3.

Table E.4-1. Terminology and notation used in example problem.

Parameter	Notation
Footing length	L_f
Footing width	B_f
Footing depth of embedment	D_f
Vertical component of resultant load	V
Moment	M
Eccentricity	$e = M/V$
Eccentricity in B_f direction	e_B
Eccentricity in L_f direction	e_L
Effective width	$L'_f = L_f - 2e_L$
Effective length	$B'_f = B_f - 2e_B$
Effective footing area	$A'_f = B'_f L'_f = (B_f - 2e_B)(L_f - 2e_L)$
Equivalent total uniform bearing stress	$q_{tveu} = V/A'_f$
Equivalent net uniform bearing stress	$q_{nveu} = q_{tveu} - \gamma_p (\gamma_s D_f)$
Factored net bearing resistance	q_{Rn}
Resistance factor of bearing resistance	ϕ_b
Load factor for permanent earth load	γ_p
Unit weight of soil within D_f	γ_s
<u>Note</u>: γ_p is the load factor for permanent vertical earth ("EV" designation) pressure load and can be obtained from Tables 3.4.1-1 and 3.4.1-2 of AASHTO (2007 with 2009 Interims).	

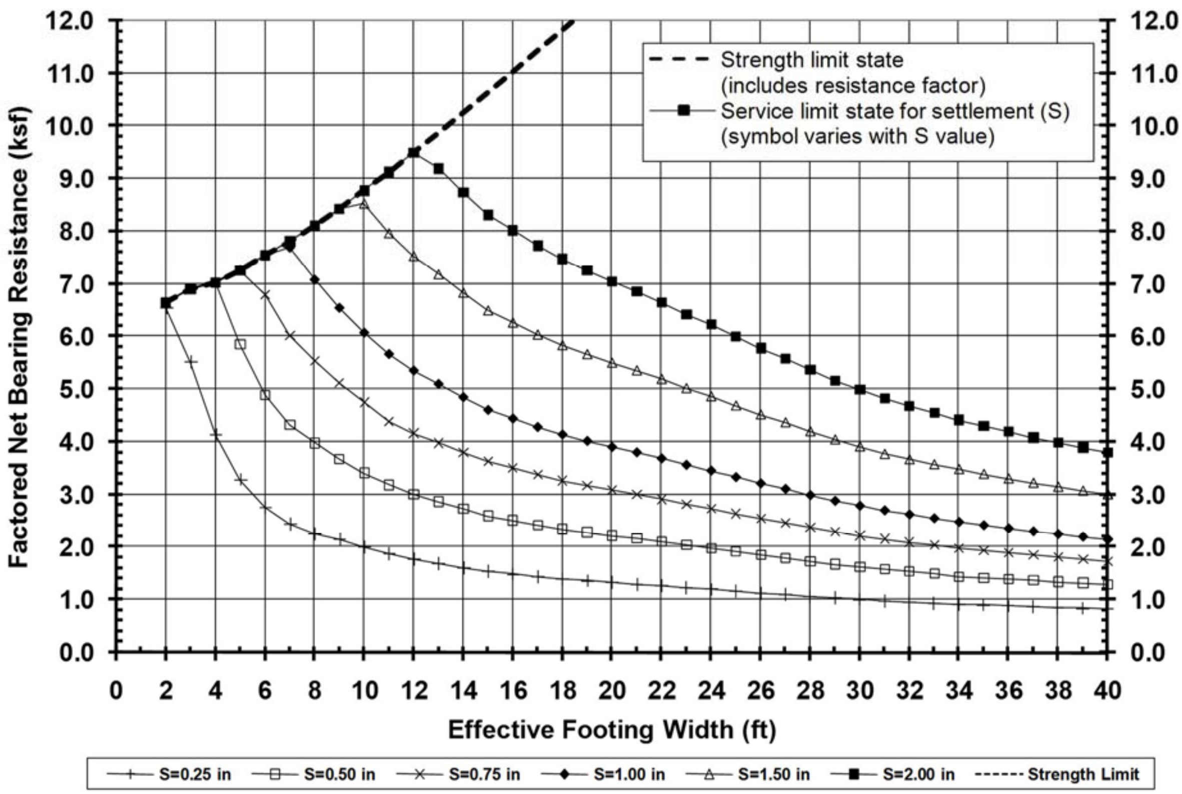


Figure E.4-1: Graph. Example of a factored bearing resistance chart for a footing of length, $L'_f = L_f = 150$-ft (no eccentricity) and depth of embedment, $D_f = 6$-ft with base elevation of 994-ft. The resistance factor of $\phi_b = 0.45$ is included in the strength limit state curve. For the service limit state $\phi_b = 1.00$ and it is included in the service limit state curves. Therefore all curves are for factored resistance. "S" in the legend refers to immediate settlement.

Table E.4-2. Pertinent parameters for example abutment footing (L'_f=150-ft).

Parameter	Limit State (Notes 1 and 2)	
	Service I Limit State	Strength I (maximum)
Vertical component of the resultant load	V_{SER} = 9,080 kips	V_{STR} = 12,028 kips
Moment	M_{SER} = 22,720 k-ft	M_{STR} = 35,290 k-ft
Eccentricity in the B_f-direction (Note 3)	$e_{B\text{-}SER}$ = 2.50-ft	$e_{B\text{-}STR}$ = 2.93-ft

Notes:

(1) Only one service limit state and strength limit state are used herein for illustration purposes. In actual design all applicable service and strength limit states must be considered.

(2) The notations are appended by subscripted text to denote the appropriate limit state being considered. Thus, V_{STR} denotes the vertical component of the resultant load for the strength limit state while $e_{B\text{-}SER}$ denotes the eccentricity in the B_f-direction using service limit state loads.

(3) The B-direction is the direction of the least lateral dimension of the footing. The eccentricity in the L-direction for an abutment footing is commonly negligible and is assumed to be zero for this example case, i.e., $L'_f = L_f$. For cases where the footing has eccentricity in both directions, the eccentricity in the L-direction should also be evaluated. In the case of the eccentricity in both directions, the least lateral dimension is the smaller dimension of the footing after adjustment for the eccentricities.

Table E.4-3. Pertinent parameters for Service I Limit State related to significant construction points for example abutment footing (L_f=150-ft).

Quantity	Units	Construction-point			
		1	2	3	4
		End of construction of footing	End of construction of stem, backwall and wingwalls	Completion of earth fill behind abutment	Placement of Superstructure and open to traffic
V	k	1,310	3,310	6,446	9,080
M	k-ft	0	400	6,215	22,720
e_B = M/V	ft	0.00	0.12	1.93	2.50

Notes:

1. As indicated in Table E.4-1, V is the vertical load, M is the moment due to the vertical and horizontal load, and e_B is the eccentricity in the B-direction.
2. Only Service I limit state is addressed since settlements will be evaluated using the construction-point concept.

NOTE: This example problem uses only Service I and Strength I (max) limit states to illustrate the process of determining the plan size of a footing. In actual designs all applicable limit states must be evaluated using the procedures described in this appendix.

E.4.1 STEP-BY-STEP PROCEDURE FOR SIZING A SPREAD FOOTING AT SERVICE AND STRENGTH LIMIT STATES

1. Assume a total footing width, $B_{f\text{-}SER}$. Calculate effective footing width $B'_{f\text{-}SER} = B_{f\text{-}SER} - 2e_{B\text{-}SER}$. Calculate $q_{nveu\text{-}SER}$. Enter the bearing resistance chart with $q_{nveu\text{-}SER}$ and effective footing width, $B'_{f\text{-}SER}$ and determine the settlement, S. Compare S with a target tolerable total settlement value, S_{tol}. If necessary iterate the footing width until $S \approx S_{tol}$.

   Example: Assume S_{tol} = 0.90-in.[1] Assume $B_{f\text{-}SER}$ = 15-ft
   Since $e_{B\text{-}SER}$ = 2.50-ft, $B'_{f\text{-}SER} = B_{f\text{-}SER} - 2e_{B\text{-}SER}$ = 15-ft – 2(2.5-ft) = 10-ft
   For L'_f = 150-ft and $B'_{f\text{-}SER}$ = 10-ft, $A'_{f\text{-}SER}$ = (150-ft) (10-ft) = 1,500 ft^2
   $q_{tveu\text{-}SER} = V_{SER}/A'_{f\text{-}SER}$ = 9,080 kips / 1,500 ft^2 = 6.05 ksf

   From Table 3.4.1-1 of AASHTO (2007 with 2009 Interims) the load factor γ_p for vertical earth pressure corresponding to Service I limit state is 1.0. Using the values provided in Note 12 of Table 1, the factored overburden stress at footing base level = $\gamma_p(\gamma_s D_f)$ = (1.0)(0.120 kcf) (6-ft) = 0.72 ksf

   $q_{nveu\text{-}SER} = q_{tveu\text{-}SER} - \gamma_p(\gamma_s D_f)$ = 6.05 ksf – 0.72 ksf = 5.33 ksf

   Enter Figure E.4-1 with $B'_{f\text{-}SER}$ = 10-ft from X-axis and $q_{veu\text{-}SER}$= 5.33 ksf from the Y-axis and find the point of intersection on the chart which represents the estimated settlement for this particular set of $B'_{f\text{-}SER}$ and $q_{veu\text{-}SER}$ values. From Figure E.4-1, the estimated settlement, S, is 0.87-in which is slightly less than 0.90-in. Since $S \approx S_{tol}$ the assumed footing width is correct. Otherwise, repeat the process with another assumed footing width until $S \approx S_{tol}$ is achieved.

2. Check if $e_{B\text{-}STR} < B_{f\text{-}SER}/3$. If yes, then denote the total footing width after this step as $B_{f\text{-}STR}$ since it is based on comparison with strength limit state criterion for eccentricity.

   Example: For $B_{f\text{-}SER}$ = 15-ft, $B_{f\text{-}SER}/3$ = 5.00-ft
   From Table E.4-2, $e_{B\text{-}STR}$ = 2.93-ft.
   Since $e_{B\text{-}STR} < B_{f\text{-}SER}/3$, a footing width of 15-ft is acceptable based on eccentricity consideration.
   Denote the footing width for strength limit state design as B_{STR} = 15-ft. This is the footing width that is also used for structural design and detailing.

3. For strength limit state, determine the effective width of the footing $B'_{f\text{-}STR} = B_{f\text{-}STR} - 2e_{B\text{-}STR}$ and $q_{nveu\text{-}STR}$

[1] The value of 0.90-in is used for illustration purposes and does not represent a standard or fixed value. In actual design, the value shall be based on the tolerable total settlement determined by the structural specialist.

Example: For $B_{f\text{-}STR}$ = 15-ft and $e_{B\text{-}STR}$ = 2.93-ft.
$B'_{f\text{-}STR} = B_{f\text{-}STR} - 2e_{B\text{-}STR}$ = 15-ft – 2(2.93-ft) = 9.14-ft.
For L'_f = 150-ft and $B'_{f\text{-}STR}$ = 9.14-ft, $A'_{f\text{-}STR}$ = (150-ft) (9.14-ft) = 1,371 ft^2
$q_{tveu\text{-}STR} = V_{STR}/A'_{f\text{-}STR}$ = 12,028 kips / 1,371 ft^2 = 8.77 ksf

From Table 3.4.1-2 of AASHTO (2007), the load factor γ_p for permanent vertical earth pressure corresponding to Strength I (maximum) limit state is 1.35 for "Retaining Walls and Abutments." Using γ_s= 120 pcf, the factored overburden stress at footing base level = $\gamma_p(\gamma_s D_f)$ = (1.35)(0.120 kcf) (6-ft) ≈ 0.97 ksf

$q_{nveu\text{-}STR} = q_{tveu\text{-}STR} - \gamma_p(\gamma_s D_f)$ = 8.77 ksf – 0.97 ksf = 7.80 ksf

4. For $B'_{f\text{-}STR}$ determine the factored net bearing resistance, q_{Rn}, from the steeply rising curve based on shear strength considerations.

Example: Enter Figure E.4-1 with $B'_{f\text{-}STR}$ = 9.14-ft from X-axis and find the point of intersection with the steeply rising curve above the settlement curves. This point of intersection represents the factored net bearing resistance, q_{Rn}, for $B'_{f\text{-}STR}$.
From Figure E.4-1, for $B'_{f\text{-}STR}$ = 9.14-ft, q_{Rn} ≈ 8.4 ksf.

5. If $q_{Rn} > q_{nveu\text{-}STR}$ then the footing width B_{STR} is adequate.

Example: From Step 3, $q_{nveu\text{-}STR}$ = 7.80 ksf
From Step 4, q_{Rn} ≈ 8.4 ksf
Since $q_{Rn} > q_{nveu\text{-}STR}$, the footing width $B_{f\text{-}STR}$ is adequate.

Repeat the above steps for all applicable strength and service limit states and determine the governing spread footing size, i.e., total width (B_f) and total length (L_f). For every limit state, the spread footing size should also be checked for sliding as per the requirements of Article 10.6.3.4 of AASHTO (2007 with 2009 Interims).

There are several other ways to use factored bearing resistance charts. For example, one can conceivably establish a preferred footing size based on project space constraints and then enter the chart from the X-axis to design the footing. Alternatively, one can select a tolerable settlement contour curve and evaluate several alternative combinations of factored bearing resistance and effective footing width in an attempt to balance the settlements across several discrete footings at a given substructure element.

Regardless of the way the data in a factored bearing resistance chart are evaluated, the structural specialist can perform parametric analyses to optimize the size of footings. It is anticipated that some level of iterative analysis will be required to determine a footing configuration that meets the requirements of the various limit states. Commonly, the service limit state is evaluated first to establish the size of the footing and then the footing is checked with respect to the strength limit state.

Finally, it should be remembered that the structural design of the footing should be performed by using the total governing footing width (B_f) and length (L_f) with the appropriate bearing stress distribution as per Article 10.6.5 of AASHTO (2007 with 2009 Interims) - uniform if no eccentricity, trapezoidal or triangular if there is eccentricity.

E.4.2 CONSTRUCTION-POINT ANALYSIS

A factored bearing resistance chart can be used to develop a plot of net factored bearing resistance versus settlement for one or more effective footing widths of interest. Such a plot is useful for performing a construction-point analysis. To develop this plot the structural specialist enters the X-axis of a bearing capacity chart, such as that shown in Figure E.4-1, with the effective footing width(s) of interest and reads the values of net bearing resistance corresponding to the intersection points of the effective width line with the curves representing the various magnitudes of settlement. The results can then be plotted as shown in Figure E.4-2, which was developed by following the above procedure for selected footing widths with reference to the X-axis in Figure E.4-1.

Figure E.4-2 can then be used to perform a construction-point analysis for a footing having a given effective footing width by entering the chart on the Y-axis with values of the effective net bearing resistance corresponding to the construction points of interest and reading the corresponding settlements on the X-axis. For the example problem of an abutment footing, the significant construction-points are as noted in Table E.4-4.

Table E.4-4: Common significant construction points for an abutment footing.

Construction-point	Description
1	End-of-construction of the spread footing
2	End-of-construction of the abutment stem-wall, back-wall, and structurally connected wing-walls
3	End-of-placement of earth fill behind the abutment
4	After placement of superstructure and opening to traffic

Table E.4-5 summarizes the computations for the various construction points using the data provided in Table E.4-3 and Figure E.4-2 developed from Figure E.4-1. The settlements that are particularly relevant to the performance of the bridge superstructure are those that occur at construction points 3 and 4. Based on the data in Table E.4-5, the difference in settlement between construction point 4 (S_4) and construction-point 3 (S_3) is 0.36 inches (= 0.87-in–0.51-in). Thus, the relevant settlement of 0.36 inches is only 41% of the total settlement of 0.87 inches as calculated in Step 1 of the procedure above. The value of 41% is consistent with the observations and discussions in Chapter 4. Furthermore, for a footing width of 15-ft, the S/B_f value is 0.48% based on the total settlement (0.87 inches) and 0.20% based on the construction-point settlement (0.36 inches). These values of S/B_f are within the ranges of the documented data discussed in Chapter 2 and Appendix B.

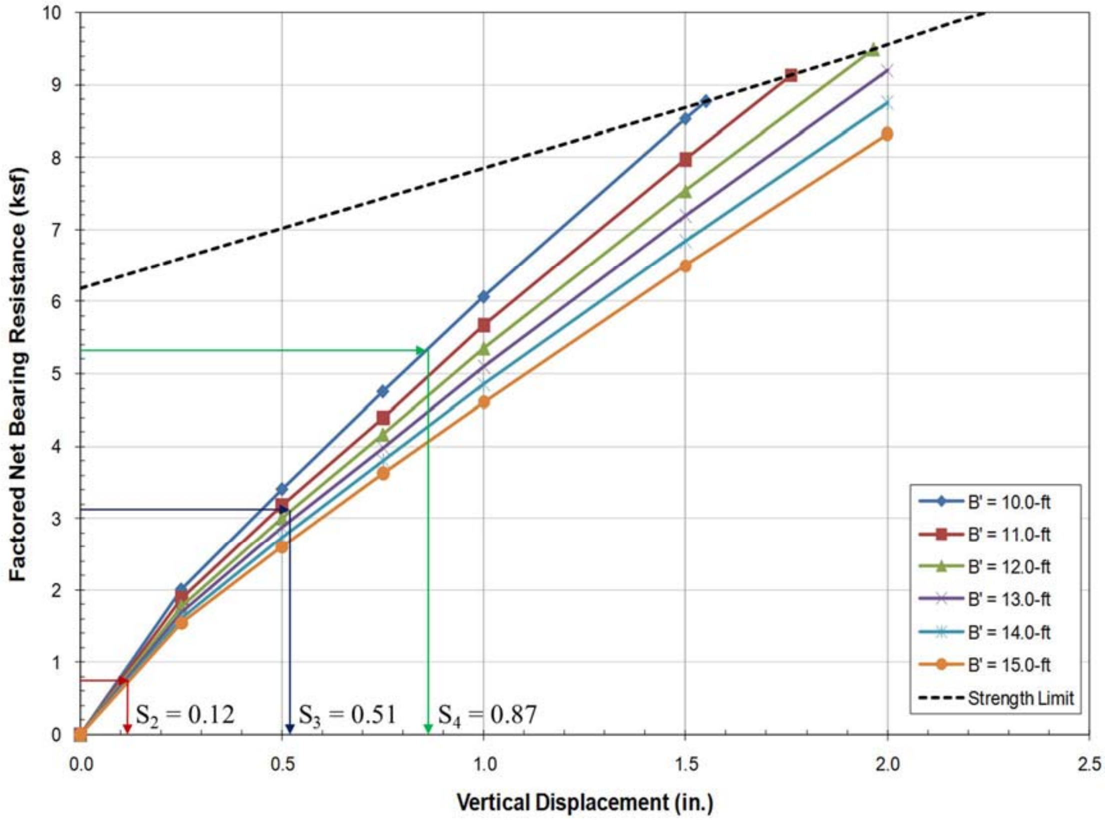


Figure E.4-2. Graph. Example of a factored bearing resistance chart in terms of stress-settlement curves for a range of effective footing widths.

Table E.4-5: Summary of computations of settlements at significant construction points for the example abutment footing.

Quantity	Units	Construction-point 1	2	3	4
		End of construction of footing	End of construction of stem, backwall and wingwalls	Completion of earth fill behind abutment	Placement of Superstructure and open to traffic
V	k	1,310	3,310	6,446	9,078
M	k-ft	0	400	6,215	22,720
$L'_f = L_f$	ft	150.00	150.00	150.00	150.00
B_f	ft	15.00	15.00	15.00	15.00
$e_B = M/V$	ft	0.00	0.12	1.93	2.50
$B'_f = B_f - 2e_B$	ft	15.00	14.76	11.14	10.00
$q_{tveu} = V/[(B'_f)(L'_f)]$	ksf	0.58	1.50	3.86	6.05
$\gamma_p(\gamma_s D_f)$	ksf	0.72	0.72	0.72	0.72
$q_{nveu} = q_{tveu} - \gamma_p(\gamma_s D_f)$	ksf	-0.14	0.78	3.14	5.33
S (from Figure E.4-2)	in	$S_1 = 0.00$	$S_2 = 0.12$	$S_3 = 0.51$	$S_4 = 0.87$

Conceivably, in cases where strength limit state may not be a concern, e.g., in dense soils, one could theoretically optimize the footing width by reducing it so that the theoretical settlement between construction points is closer to tolerable limits.

This example problem clearly illustrates the importance of performing construction-point analyses when spread footings are being evaluated as a technically feasible and cost effective alternative foundation system for highway bridge structures.

E.5 SPREAD FOOTING DESIGN BASED ON STRUCTURAL CONSIDERATIONS

Although the construction-point concept is very significant in determining the effects of distortion of the spread footing on the superstructure and its final alignment, the concept has little or no effect on the structural design of spread footings themselves. While geotechnical considerations and service limit states govern the plan size of a footing, the thickness of a footing and its reinforcement is based on structural considerations and strength limit states. In the strength limit-state proportioning of the footing and its reinforcement, the weightless bridge structure can be instantaneously wished into place and all the loads applied at the same time. The distortion of the footing is ignored, and at the limit, the loads are re-distributed to be resisted non-linearly by the footing sections. This section of the appendix presents a brief overview of the contact pressure distributions, use of maximum and minimum load factors to determine the critical force effects, and evaluation of the flexural and shear structural resistance.

E.5.1 Contact Pressure Distributions

The contact pressure distribution at the base of a footing is a function of the relative stiffness of the footing with respect to the soil subgrade. While uniform contact pressure distribution over the effective width of the footing is assumed for the bearing resistance and settlement evaluation, the structural design is based on the more traditional triangular or trapezoidal contact pressure distributions, which are determined from factored loads. Thus, the pressure distribution is a factored pressure distribution. The proper calculation of the factored pressure distribution is therefore a key to the structural design of the footing. In this context, it is important to understand the proper application of the maximum and minimum load factors as discussed next.

E.5.2 Maximum and Minimum Load Factors

In the LRFD approach in which uncertainty is acknowledged, loads can be greater than the nominal loads or less than the nominal loads based upon the assumed uncertainty. Thus, maximum and minimum load factors are specified for permanent loads in LRFD. Following are the two basic rules for selection and application of the load factors:

- If the particular load in question increases the load side of the LRFD equation, it must be factored up by the specified maximum load factor.
- If the particular load in question decreases the load side of the LRFD equation, it must be factored down by the specified minimum load factor.

In determining the factored moments and shears used to proportion the footing and its reinforcement, the factored pressure distributions are the reactions of the loads applied to the top of the footing used in the free-body diagrams to determine the critical force effects (moment or shear) for the structural design. To satisfy equilibrium, the loads applied to the top of the footing (including the footing's body forces) must be factored by the very same load factors, maximum or minimum, before they are used to determine the pressure distribution at the base of the footing.

E.5.3 Flexural and Shear Resistance of Footings

As noted in Section C.1 of Appendix C, the structural design of a spread footing involves an evaluation of the flexural (bending) and shear resistance. While the proportioning of the footing and its reinforcement to resist moment has not changed with the LRFD approach in terms of the determination of the flexural resistance, the determination of the shear resistance offers many new options. In Section 5 (Concrete Structures) of AASHTO (2007 with 2009 Interims) no less than four sectional-model procedures and the strut-and-tie model can be applied for the determination of the shear resistance of a footing. Since the thickness of a footing is generally governed by shear it is important that the proper method be chosen for the structural design.

The presentation of many options for the determination of shear resistance in Section 5 (Concrete Structures) of AASHTO (2007 with 2009 Interims) was in response to the complexity of the modified-compression field theory shear-resistance procedure (MCFT) of the original drafts of the first edition of the specifications. MCFT is a model that acknowledges the ability of diagonally-cracked concrete sections to resist shear. The design parameters needed to use this procedure were originally iteratively derived from tabularized values. The solution was thought to be time-consuming and non-unique. A recent interim change to the specifications has replaced this iterative procedure with a direct solution with equations for the design parameters.

The simplest and most often used procedure found in published design examples is the simplified procedure for non-prestressed sections in which constant values of the design parameters are assumed consistent with older, less accurate shear-resistance models. However, this procedure is not recommended since, as noted in AASHTO (2007 with 2009 Interims), "these traditional expressions can be seriously unconservative for large members not containing transverse reinforcement." Most spread footings are not reinforced with significant amounts of transverse reinforcement. The recommended procedure for determining the shear resistance of a spread footing is the modified-compression field theory (MCFT) because it is the most accurate and, with the addition of equations for the design parameters, it is easy to apply. The reader is referred to Section 5 (Concrete Structures) of AASHTO (2007 with 2009 Interims) for further details on the various procedures for evaluating the flexural and shear resistance of spread footings.