Incorporation of Foundation Movements in *AASHTO LRFD* Bridge Design Process Second Edition

A product of the SHRP2 solution, Service Limit State Design for Bridges

SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

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7. Author(s)
Naresh C. Samtani, PhD. PE, NCS GeoResources, LLC
John M. Kulicki, PhD, PE, Consultant

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NCS GeoResources, LLC
640 W Paseo Rio Grande
Tucson, AZ 85737

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16. Abstract

The consideration of foundation movements in the bridge design process can lead to the use of cost-effective structures with more efficient foundation systems. This report focuses on the work related to foundation movements developed as part of the SHRP2 Project R19B. Its purpose is to explain the calibration process for foundation movements and incorporation of calibrated foundation movements into the bridge design process. The proposed approach and modifications will help avoid overly conservative criteria for foundation movements that can lead to (a) foundations that are larger than needed, or (b) a choice of less economical foundation type (such as using a deep foundation at a location where a shallow foundation would be adequate). Implementation of the proposed procedures may often allow consideration of larger foundation movements. The proposed design procedures permit a rational comparison of the alternative foundations and structures and provide more uniform serviceability and safety.

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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

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Executive Summary

Foundation movements induce additional (secondary) force effects such as shear and moment in bridge structures which, in turn, can lead to detrimental consequences such as cracking. This report presents the process that was developed during Project R19B of the second Strategic Highway Research Program for calibration of the *SE* load factor related to foundation movements and its incorporation in the bridge design process.

This report first discusses the current provisions related to foundation movements in the Bridge Design Specifications of the American Association of State Highway and Transportation Officials based on the load and resistance factor design (LRFD) approach. This discussion is followed by the presentation of an approach to incorporate load-movement curves in the conventional LRFD calibration process. Practical difficulties associated with the use of Monte Carlo-based calibration processes are identified. The concept of a probability exceedance chart that permits the incorporation of uncertainty in predicted movements and user-specified deterministic tolerable movements in a unified manner is presented. The *SE* load factor is defined as the ratio of tolerable to predicted foundation movement corresponding to a target reliability index based on structural limit states such as cracking in a superstructure. The calibration process for *SE* load factor can be used for any analytical method for predicting any foundation movement (for example, vertical, lateral, etc.). The calibration process is demonstrated using the example of vertical foundation movement (that is, settlement) for five methods (Schmertmann, Hough, D'Appolonia, Peck and Bazaraa, and Burland and Burbridge) through a dataset of 20 points for predicted and measured foundation settlements based on 10 bridges in the northeast United States. Incorporation of realistic values of foundation movements through the use of a construction-point concept is discussed. Recommended values of *SE* load factors are developed for the dataset considered. Detailed example problems using two-span, four-span, and five-span bridges to evaluate the effect of a wide range of foundation settlements and proposed *SE* load factors on bridge design are presented.

The proposed approach and modifications will help avoid overly conservative criteria for foundation movements that can lead to (a) foundations that are larger than needed, or (b) a choice of less economical foundation type (such as using a deep foundation at a location where a shallow foundation would be adequate). Implementation of the proposed procedures may often allow consideration of larger foundation movements. The proposed design procedures permit a rational comparison of the alternative foundations and structures and provide more uniform serviceability and safety. This report will serve as a useful reference for researchers as well as agencies desiring to develop *SE* load factors based on local methods that are better suited to their regional geologies and subsurface investigation techniques.

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Acronyms, Abbreviations, and Symbols

AASHTO	American Association of State Highway and Transportation Officials
ADOT	Arizona Department of Transportation
ASD	allowable stress design
CDF	cumulative distribution function
COV	coefficient of variation
DL	dead load
ELP	equal load probability
FHWA	Federal Highway Administration
IAP	Implementation Assistance Program
in.	inch
LFD	load factor design
LL	live load
LRFD	load and resistance factor design
MSE	mechanically stabilized earth
PDF	probability distribution function
PEC	probability exceedance chart
R19B	SHRP2 Project R19B (*Service Limit State Design for Bridges*)
SCOBS	AASHTO Subcommittee on Bridges and Structures
SHRP2	second Strategic Highway Research Program
TRB	Transportation Research Board

A_d	angular distortion
A_{d1}, A_{d2}, A_{d3}, A_{d4}	angular distortions for a four-span bridge
A_{df}	factored angular distortion
$A_{dfi\text{-}j}$	factored angular distortion where *i* represents the span number and *j* represents the mode (1 or 2)
$A_{df1\text{-}1}$, $A_{df2\text{-}1}$, $A_{df3\text{-}1}$, $A_{df4\text{-}1}$	Mode 1 factored angular distortions for a four-span bridge
$A_{df1\text{-}2}$, $A_{df2\text{-}2}$, $A_{df3\text{-}2}$, $A_{df4\text{-}2}$	Mode 2 factored angular distortions for a four-span bridge
B_f	least lateral plan dimension (width) of spread footing
COV_g	coefficient of variation of limit state function g treated as a normal random variable
COV_{lng}	coefficient of variation of limit state function g treated as a lognormal random variable
COV_Q	coefficient of variation of the ratio of the measured to predicted values of the load
COV_X	coefficient of variation of accuracy, *X*, values
COV_λ	coefficient of variation of bias, λ, values
D_f	depth to bottom of spread footing measured from finished grade
E	elastic modulus
f	frequency
F	point on load-movement (Q-δ) curve representing strength limit state
g	limit state function
HL93	designation of live load model
J	exponent [= $\beta(\sigma_{LNA\text{-}X}) - \mu_{LNA\text{-}X}$]
K	exponent [= $\beta(\sigma_{LNA\text{-}\lambda}) - \mu_{LNA\text{-}\lambda}$]
I	moment of inertia
L_S	span length
L_{S1}, L_{S2}, L_{S3}, L_{S4}	span lengths for a four-span bridge
L_f	longer lateral plan dimension (length) of spread footing

ln (or LN)	natural logarithm
ln(X)	natural logarithm of accuracy, X, values
ln(λ)	natural logarithm (lognormal) of bias, λ, values
M_{Δ}	bending moment induced by a differential settlement, Δ_d
N	point on load-movement (Q-δ) curve representing nominal resistance level
O	origin of load-movement (Q-δ) curve
P	lateral soil reaction on a deep foundation; or probability
P_e	probability of exceedance
P_{eT}	target probability of exceedance
P_f	probability of failure
Q	load (or force effect)
Q_{mean}	mean load
Q_n	nominal load
R	resistance
R_{mean}	mean resistance
R_n	nominal resistance
S	foundation settlement (vertical movement); also refers to point on load-movement (Q-δ) curve representing service limit state
S_{A1}, S_{P1}, S_{P2}, S_{P2}, S_{A2}	support settlements for a four-span bridge
SE	force effect due to settlement
S_f	factored total relevant settlement
$S_{f\text{-}A1}$, $S_{f\text{-}P1}$, $S_{f\text{-}P2}$, $S_{f\text{-}P3}$, $S_{f\text{-}A2}$	factored support settlements for a four-span bridge
S_M	measured settlement
S_P	predicted (calculated) Settlement
SR	settlement ratio
S_t	unfactored predicted settlement
S_T	tolerable settlement
S_{tr}	unfactored total relevant settlement

S_W, S_X, S_Y, S_Z	settlements corresponding to vertical loads *W*, *X*, *Y* and *Z*
y	lateral displacement at top of piles
W	vertical load after foundation construction
X	accuracy ($X = \delta_P/\delta_T$ or $X = S_P/S_M$); or vertical load after substructure construction
Y	vertical load after superstructure construction
z	standard normal variable (variate)
Z	vertical load after wearing surface construction
%	percent
β	reliability index
β_T	target Reliability Index
γ	load factor
γ_{SE}	load factor for *SE* load; movement load factor
δ	movement
δ_f	factored movement
δ_F	movement at factored force effect, $Q_F = \gamma(Q_n)$
δ_M	measured movement (resistance)
δ_N	movement at load corresponding to nominal resistance, R_n
δ_P	predicted movement (force effect)
δ_S	movement at nominal force effect, Q_n
δ_T	tolerable movement (resistance)
δ_{T1}, δ_{T2}, δ_{T3}	various tolerable movements
Δ_d	differential settlement
Δ_{d1}, Δ_{d2}, Δ_{d3}, Δ_{d4}	differential settlements for a four-span bridge
$\Delta_{d100'}$	differential settlement over 100 ft with pier or abutments and differential settlement between piers
Δ_f	factored differential settlement
$\Delta_{fi\text{-}j}$	factored differential settlement where *i* represents the span number and *j* represents the mode (1 or 2)

$\Delta_{f1\text{-}1}$, $\Delta_{f2\text{-}1}$, $\Delta_{f3\text{-}1}$, $\Delta_{f4\text{-}1}$	Mode 1 factored differential settlements for a four-span bridge
$\Delta_{f2\text{-}1}$, $\Delta_{f2\text{-}2}$, $\Delta_{f3\text{-}2}$, $\Delta_{f4\text{-}2}$	Mode 2 factored differential settlements for a four-span bridge
λ	Bias (= $1/X = S_M/S_P$)
λ_Q	Bias factor for load
λ_R	Bias factor for resistance
λ_δ	Bias factor for movement (= $1/X = \delta_T/\delta_P$)
μ	arithmetic mean
μ_X	arithmetic mean of Accuracy, X, values
μ_g	arithmetic mean of limit state function g treated as a normal random variable
μ_{lng}	arithmetic mean of limit state function g treated as a lognormal random variable
μ_λ	arithmetic mean of Bias, λ, values
$\mu_{LNA\text{-}X}$	arithmetic mean of ln(X) values
$\mu_{LNA\text{-}\lambda}$	arithmetic mean of ln(λ) values
$\mu_{LNC\text{-}X}$	correlated mean of ln(X) values
$\mu_{LNC\text{-}\lambda}$	correlated mean of ln(λ) values
σ	arithmetic standard deviation
σ_g	arithmetic standard deviation of limit state function g treated as a normal random variable
σ_{lng}	standard deviation of limit state function g treated as a lognormal random variable
σ_X	standard deviation of Accuracy, X, values
σ_λ	standard deviation of Bias, λ, values
$\sigma_{LNA\text{-}X}$	arithmetic standard deviation of ln(X) values
$\sigma_{LNA\text{-}\lambda}$	arithmetic standard deviation of ln(λ) values
$\sigma_{LNA\text{-}X}$	correlated standard deviation of ln(X) values
$\sigma_{LNA\text{-}\lambda}$	correlated standard deviation of ln(λ) values
ϕ	resistance factor

Φ	standard normal distribution
Φ^{-1}	inverse of standard normal distribution
η	a factor that depends on desired target reliability index

Chapter 1. Introduction

Selected elements of the second Strategic Highway Research Program (SHRP2) are advanced into practice primarily through the Implementation Assistance Program (IAP) sponsored by the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO). The IAP provides technical and financial support to transportation agencies to encourage widespread adoption and use of research initially conducted through the Transportation Research Board (TRB).

Service Limit State Design for Bridges (R19B), often referred to as Project R19B, is a SHRP2 Solution whose objectives include the development of design and detailing guidance, calibration of service limit states to provide 100-year bridge life, and a framework for further development of calibrated service limit states. The Project R19B team developed a set of possible service limit states on the basis of a survey of owners and a literature review that included other national and international bridge design specifications. Those service limit states were reviewed to determine what could be calibrated using reliability theory. Calibrated, reliability-based load factors or resistance factors, or both, were developed for:

- Foundation movements
- Cracking of reinforced concrete components
- Live-load deflections
- Permanent deformations
- Cracking of prestressed concrete components
- Fatigue of steel and reinforced concrete components

The details of these topics are provided in Kulicki et al. (2015). Portions of the work were presented at several meetings of the AASHTO Subcommittee on Bridges and Structures (SCOBS).

A rational consideration of foundation movements in the bridge design process can lead to the use of cost-effective structures with more efficient foundation systems. The proposed approach and modifications will help avoid overly conservative criteria that can lead to (a) foundations that are larger than needed, or (b) a choice of less economical foundation type (such as using a deep foundation at a location where a shallow foundation would be adequate). The Project R19B work pertaining to foundation movements was presented by either one or both authors of this report at the following forums:

- Annual AASHTO SCOBS meetings in New Orleans, Louisiana (2012); Columbus, Ohio (2014); Saratoga Springs, New York (2015); and Minneapolis, Minnesota (2016)
- Joint T-5 and T-15 SCOBS mid-year meeting in Chicago, Illinois (2015)

- Southwest Geotechnical Engineers Conference in Phoenix, Arizona (2017)
- 1.5-day training course at Central Federal Lands Highway in Denver, Colorado (2017)
- 1.5-day training course at Eastern Federal Lands Highway in Sterling, Virginia (2017)
- 1.5-day training course at Western Federal Lands Highway in Vancouver, Washington (2017)
- T-15 mid-year meeting in Raleigh, North Carolina (2017)

The presentations and training courses were attended by personnel from several agencies (federal, state, local, and forest) as well as consultants from many design firms. These presentations and training courses generated considerable discussion and valuable comments.

This report was developed as part of the technical assistance provided through the IAP and concentrates on the work related to foundation movements developed as part of the SHRP2 Project R19B. The goal of this report is to explain the development and implementation of calibrations for foundation movements into the bridge design process. This report does not address foundation movements due to extreme events; for example, movements related to liquefaction, lateral spreading, vessel or vehicle impact, etc.

The scope of this report is to identify and consolidate the relevant content of the Project R19B report (Kulicki et al., 2015) and additional materials developed since the issuance of the Project R19B report (for example, flowcharts and examples based on the comments received as part of the various presentations previously noted). This information will provide background and rationale that can be used to support decisions regarding changes to the *AASHTO LRFD* B*ridge Design Specifications*.

Documents from various sources such as AASHTO, FHWA, and SHRP2 are referenced in this report. Each reference document has its own style and organization, which often creates confusion during cross-referencing of documents. Appendix A provides the conventions used in this report vis-à-vis conventions in other publications.

Chapter 2. Bridge Foundation Types and Movements

Figure 2-1 illustrates major components of a common bridge structure. In broad terms, bearings and all components above the bearing level are part of the bridge superstructure, while all components below the bearing level are part of a bridge substructure. The foundation is defined as the component of the substructure that is below the ground level. On Figure 2-1, the foundation is shown as footing supported by piles.

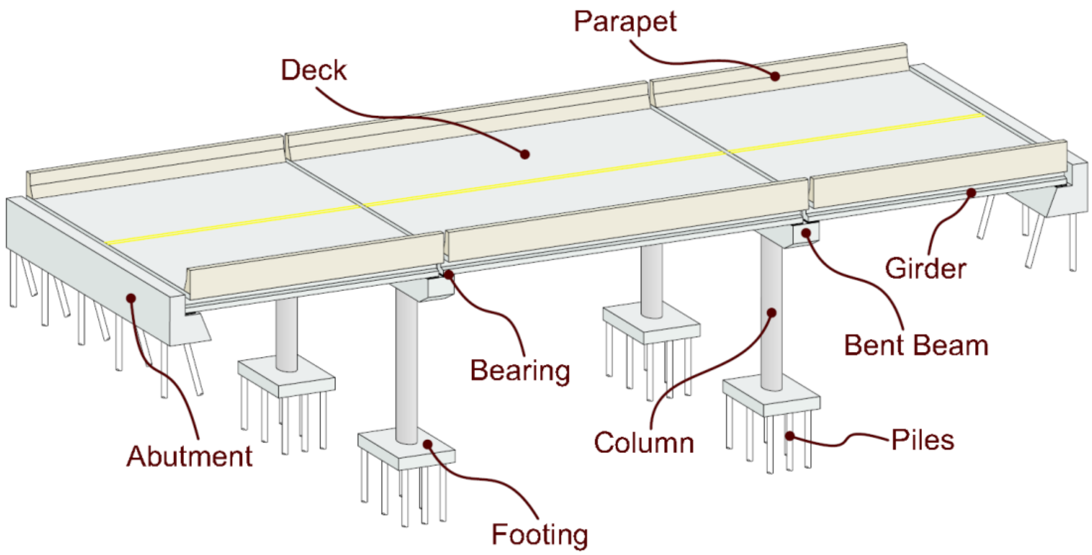


Figure 2-1. Major Components of a Bridge Structure (Nielson, 2005)

The two major alternate foundation types are "shallow" and "deep." The geometry of a typical shallow foundation or spread footing is shown on Figure 2-2. Shallow foundations are those wherein the depth to the bottom of the footing, D_f, is small compared to the cross-sectional size (width, B_f, or length, L_f). This is in contrast to deep foundations, such as driven piles and drilled shafts, whose depth of embedment is considerably larger than the cross-section dimension (diameter) as shown on Figure 2-3.

Foundation design and construction involves assessing factors related to engineering and economics. The selection of a feasible foundation system requires consideration of both shallow and deep foundation types in relation to the characteristics and constraints of the project and site conditions. The presence of unsuitable soil layers in the subsurface profile, adverse hydraulic conditions, or relatively small tolerable movements of the structure generally dictates the use of deep foundations because they can be designed to transfer load through less suitable subsurface layers to more suitable bearing strata.

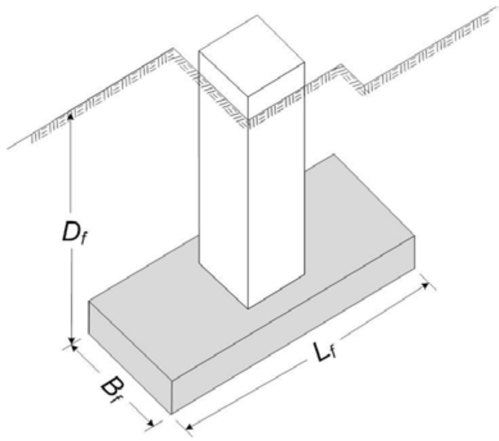
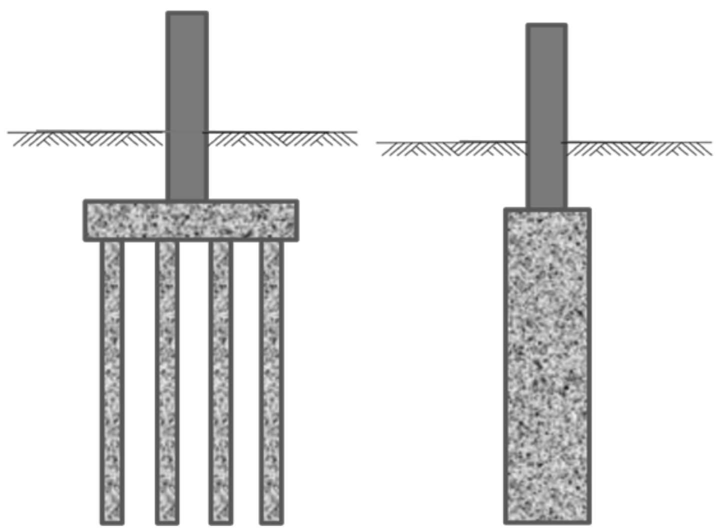


Figure 2-2. Geometry of a Typical Shallow Foundation



(a) (b)

Figure 2-3. Common Configurations of Deep Foundations, (a) Group Configuration, (b) Single Element Configuration

Regardless of the type of foundation, the key point of interest is the effect of the estimated foundation movements on the various elements of the bridge substructure and superstructure components above the foundations. The foundation movements can have multiple degrees of freedom, but for the purpose of analysis, the foundation movements can be broadly categorized as vertical (settlement) and lateral. Rotational movements can be manifested due to the combined effects of vertical and lateral movements. Torsional movements may also be possible under certain specific loading conditions (for example, dynamic). Bridge foundations and other geotechnical features, such as approach embankments, should be designed so that their movements (settlements and/or lateral movements) will not cause damage to the bridge structure.

Chapter 3. Consideration of Foundation Movements in AASHTO Bridge Design Specifications

3.1 *AASHTO LRFD*

Figures 3-1, 3-2, 3-3 and 3-4 present Tables 3.4.1-1, 3.4.1-2. 3.4.1-3, and 3.4.1-4, respectively, from the *AASHTO LRFD*. These tables present load factors for various loads used to develop design load combinations. Two-letter abbreviations are used for load designations on Figures 3-1, 3-2, 3-3, and 3-4. Figure 3-5 provides definitions for the two-letter abbreviations for load designations in accordance with Article 3.3.2 of the *AASHTO LRFD*.

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I— *LL, IM & CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL, IM & CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

Figure 3-1. *AASHTO LRFD* Table 3.4.1-1 - Load Combinations and Load Factors

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

Figure 3-2. *AASHTO LRFD* Table 3.4.1-2 - Load Factors for Permanent Load, γ_P

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effective}$	1.0	1.0
Steel Substructures	1.0	1.0

Figure 3-3. *AASHTO LRFD* Table 3.4.1-3 - Load Factors for Permanent Loads Due to Superimposed Deformations, γ_P

Component	γ_{LL}
Prestressed concrete components designed using the refined estimates of time-dependent losses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.0
All other prestressed concrete components	0.8

Figure 3-4. *AASHTO LRFD* Table 3.4.1-4 - Load Factors for Live Load for Service III Load Combination, γ_{LL}

Permanent Loads			Transient Loads		
CR	=	force effects due to creep	*BL*	=	blast loading
DD	=	downdrag force	*BR*	=	vehicular braking force
DC	=	dead load of structural components and nonstructural attachments	*CE*	=	vehicular centrifugal force
DW	=	dead load of wearing surfaces and utilities	*CT*	=	vehicular collision force
EH	=	horizontal earth pressure load	*CV*	=	vessel collision force
EL	=	miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction	*EQ*	=	earthquake load
ES	=	earth surcharge load	*FR*	=	friction load
EV	=	vertical pressure from dead load of earth fill	*IC*	=	ice load
PS	=	secondary forces from post-tensioning for strength limit states; total prestress forces for service limit states	*IM*	=	vehicular dynamic load allowance
SH	=	force effects due to shrinkage	*LL*	=	vehicular live load
			LS	=	live load surcharge
			PL	=	pedestrian live load
			SE	=	force effect due to settlement
			TG	=	force effect due to temperature gradient
			TU	=	force effect due to uniform temperature
			WA	=	water load and stream pressure
			WL	=	wind on live load
			WS	=	wind load on structure

Figure 3-5. Key to *AASHTO LRFD* Loads and Load Designations

Article 3.4.1 of the *AASHTO LRFD* states the following:

> "All relevant subsets of the load combinations shall be investigated. For each load combination, every load that is indicated to be taken into account and that is germane to the component being designed, including all significant effects due to distortion, shall be multiplied by the appropriate load factor….
>
> The factors shall be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes shall be investigated.
>
> In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. For permanent force effects, the load factor that produces the more critical combination shall be selected from Table 3.4.1-2. Where the permanent load increases the stability or load-carrying capacity

> of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated."

As per Article 3.3.2 of the *AASHTO LRFD*, the *SE* load type is categorized as transient and represents "force effect due to settlement." The force effects can be manifested in a variety of forms, such as additional (secondary) moments and change in roadway grades. Thus, even though *SE* load is considered as a transient load, the force effects caused by *SE* load type may induce irreversible (permanent) effects in the bridge superstructure unless the induced force effects are made reversible through intervention with respect to the bridge superstructure. As per Article 3.12 of the *AASHTO LRFD*, the *SE* load type is considered similar to load types *TU*, *TG*, *SH*, *CR*, and *PS*, in that it generates force effects because of superimposed deformations and which are in self-equilibrium; that is, there are no other gravity or traction (for example, braking, wind, etc.) loads in equilibrium with these forces.

While the *AASHTO LRFD* uses the word "settlement," the broader meaning of *SE* load type applies to foundation movements, whether it is settlement (vertical movement) or lateral movement or rotation. Article 3.12.1 of the *AASHTO LRFD* uses the words "support movements" as follows:

> "Force effects resulting from resisting component deformation, displacement of points of load application, and support movements shall be included in the analysis."

Thus, any reference to *SE* load type should, in general, be considered a reference to foundation movement, whether it is vertical movement (settlement) or lateral movement or rotation. Based on these discussions, it is clear that the *AASHTO LRFD* incorporates the force effects of foundation movements in the bridge design process through the concept of force effects generated by superimposed movements.[1] Further, by including the *SE* load factor, γ_{SE}, for foundation movements in both the strength and service limit states, the *AASHTO LRFD* is clearly acknowledging that foundation movements can affect the long-term load-carrying capacity and functionality of the bridge structure. Note that this load factor is shown in four out of the five strength limit states and three out of the four service limit states with an explicit value of 1.0 for the Service IV limit state. The other superimposed deformation load factors for *CR*, *SH*, *PS*, *TU* and *TG* are defined in the *AASHTO LRFD*, but *SE* does not have a value of load factor clearly defined except for Service Limit IV, for which a value of 1.0 is provided. Article 3.4.1 of the *AASHTO LRFD* states the following for selection of a value of γ_{SE}:

---

[1] Conceptually, the treatment of *SE* load type is similar to that of the *DD* load type that represents downdrag force (or drag load) due to a settlement based mechanism. Drag load is categorized as a permanent load type, and in the *AASHTO LRFD* framework, a geotechnical phenomenon of settlement is considered in terms of additional permanent load that is induced. The *DD* load type is considered in both strength and service limit state evaluations.

> "The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{SE}, may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement."

This specification provision indicates that γ_{SE} can take a value of 1.0 when settlement is considered and a value of 0.0 when settlement is not considered. Use of a load factor of 1.0 implies that the loads are taken at nominal value. For foundation movement, the nominal value of induced force effect, such as moments, is directly proportional to the value of the foundation movement (for example, settlement). When a value of γ_{SE} = 1.0 is used, the implication is that that computed value of foundation movement has no uncertainty. However, the provision does state that, "In lieu of project-specific information to the contrary…." which means that other values of γ_{SE} may be used, but no recommendations are provided for the selection of an appropriate value.

Article 3.12.6 of the *AASHTO LRFD* further indicates the following regarding *SE* load type:

> "Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered."

The commentary portion (Article C3.12.6 of the *AASHTO LRFD*) states the following:

> "Force effects due to settlement may be reduced by considering creep. Analysis for the load combinations in Tables 3.4.1-1 and 3.1.4-2 which include settlement should be repeated for settlement of each possible substructure unit settling individually, as well as combinations of substructure units settling, that could create critical force effects in the structure."

Based on these discussions, it is clear that the *AASHTO LRFD* makes explicit consideration of foundation movements in the bridge design process.

3.2 AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002)

AASHTO (2002) represented the 17th and last edition of the *Standard Specifications for Highway Bridges* that was based on the allowable stress design (ASD) (also referred to as service load design) and load factor design (LFD) platform. It is worth noting that settlement is handled more explicitly in *AASHTO LRFD* Table 3.4.1-1 than it was in corresponding Table 3.22.1A (AASHTO, 2002) wherein the settlement was not included. It may appear that the *AASHTO LRFD*-based specifications are a departure from past practice as exemplified by AASHTO (2002), in that settlement does not appear in the load combinations in AASHTO (2002), but this is not the case. Settlement is mentioned in Article 3.22.1 of AASHTO (2002), which states, "If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement." The parent article is 3.3 DEAD LOAD, implying that settlement effects should be considered wherever dead load appears in the ASD or LFD load combinations. The consideration of foundation movements in the bridge design process has been mandated by AASHTO in the past and is not a new requirement in the *AASHTO LRFD* specifications. The *AASHTO LRFD* simply clarifies previous requirements.

3.3 General Observations

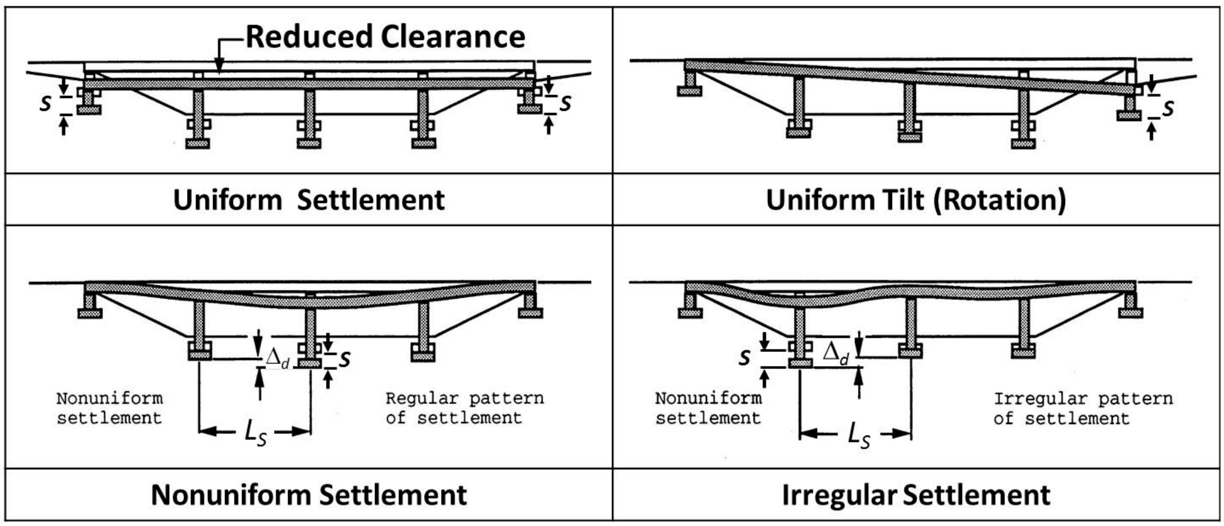
Based on the discussions in Section 3.2, the following general observations are made:

- Although the *AASHTO LRFD* refers to settlement, it should be considered in the broader context of foundation movements because a foundation can have multiple degrees of freedom.
- Evaluation of differential foundation movement has been mandated by the *AASHTO LRFD* Bridge Design Specification regardless of design platform (ASD, LFD, or LRFD). It is not a new requirement.
- In the *AASHTO LRFD* platform, foundation movements are included in the category of superimposed deformation, and the *SE* load factor, γ_{SE}, appears in both strength and service limit state load combinations.
- The choice of γ_{SE} = 1.0 might seem to imply that there is no uncertainty in the estimated value of foundation movements. However, this value was calibrated by TRB's SHRP2 Project R19B (Kulicki et al., 2015) to incorporate uncertainty based on the type of method used to estimate the foundation movements.
- Although the issue of foundation movements may appear to belong to the *AASHTO LRFD*, Section 10 (Foundations), it is the induced force effects of foundation movements that need to be incorporated in the bridge structure design. Therefore, the effect of foundation movements has been included in *SE* load type in the *AASHTO LRFD*, Section 3 (Loads and Load Factors), Table 3.4.1-1 (Load Combination and Load Factors).

Chapter 4. Effect of Foundation Movements on Bridge Structures and Uncertainty

The bridge superstructure and substructure movements can be caused by a variety of reasons, including foundation movements. The foundation movements need to be evaluated in the context of span lengths and various construction steps to understand their effect on the bridge superstructures.

Figure 4-1 presents idealized vertical movement (settlement) patterns that serve to illustrate the effect of a bridge structure within the framework of the *AASHTO LRFD Bridge Design Specifications*.



Sources: Barker et al., 1991 and Samtani and Nowatzki, 2006

S = Total Settlement; Δ_d = Differential Settlement; L_S = Span Length

Figure 4-1. Idealized Vertical Movement (Settlement) Patterns and Terminology

Vertical movement (settlement) can be subdivided into the following three components, which are illustrated on Figure 4-1:

1. **Uniform settlement**

In this case, all bridge support elements settle equally. Although the bridge support elements settle equally, they can cause differential settlement with respect to the approach embankment and associated features such as approach slabs and utilities that are commonly located in or across the end-spans of bridges. Such differential settlement can create problems. For example, it can reduce the clearance of the overpass; create a bump at the end of the bridge; or change grades at the end of the bridge causing drainage problems, misaligned joints, and distorted underground utilities at the interfaces of the bridge and approaches.

Although uniform settlements may be computed theoretically, from a practical viewpoint it is not possible for the bridge structure to experience truly uniform settlement because of a combination of many factors including the variability of loads and soil properties.

2. **Tilt or rotation**

Tilt or rotation occurs mostly in single-span bridges with stiff superstructures. Tilt or rotation may not cause distortion of the superstructure and associated damage, but because of its differential movement with respect to the facilities associated with approach embankments, tilt or rotation can create problems similar to those of uniform settlement, discussed previously. Examples include a bump at the end of the bridge, drainage problems, and damage to underground utilities.

3. **Differential settlement, Δ_d**

Differential settlement, Δ_d, defined as the difference in settlement between adjacent supports, directly results in movement of the bridge superstructure. As shown on Figure 4-1, two different patterns of differential settlement can occur:

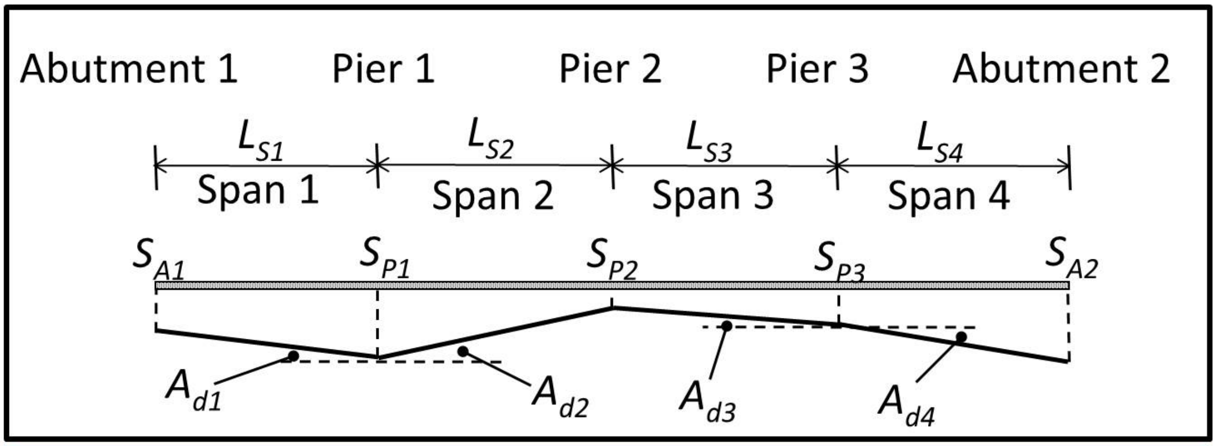
- Regular (nonuniform) pattern: The settlement increases progressively from the abutments toward the center of the bridge.
- Irregular (uneven) pattern: The settlement at each support location varies along the length of the bridge.

Both of these patterns of settlement lead to angular distortion, A_d, which is defined as the ratio of the difference in settlement between two points divided by the distance between the two points. For bridge structures, the two points to evaluate the differential settlement, Δ_d, are commonly selected as the distance between adjacent support elements, L_S, as shown on Figure 4-1. Thus, angular distortion $A_d = \Delta_d/L_S$. Stated another way, angular distortion is a normalized measure of differential settlement that includes the distance over which the differential settlement occurs. A number of studies (for example, Skempton and MacDonald [1956] and Grant et al. [1974]), have determined that the severity of differential settlement on structures is roughly proportional to the angular distortion.

Because of the inherent variability of geomaterials, the vertical movements at the support elements of a given bridge (that is, piers and abutments) will generally be different. This is true regardless of whether deep foundations or spread footings are used. Therefore, differential settlement and associated angular distortion is the most common situation and is treated herein.

Figure 4-2 shows the hypothetical case of a four-span bridge structure with five support elements (two abutments and three piers), wherein the calculated settlement, S, at each support is different. The settlements at Abutment 1, Pier 1, Pier 2, Pier 3, and Abutment 2 are

S_{A1}, S_{P1}, S_{P2}, S_{P3} and S_{A2}, respectively. In this hypothetical case, it is assumed that the substructure units between foundations and bridge superstructure are rigid (that is, all the movements experienced by the superstructure are equal to the foundation movements). Differential settlements, Δ_d, are defined as noted in the second column of the table on Figure 4-2. The angular distortion, A_d, term for each span is shown in the third column of the table on Figure 4-2. Angular distortion is a dimensionless quantity that is expressed as an angle in radians. Theoretically, the ratio Δ_d/L_S represents the tangent of the angle of distortion, but for small values of the tangent, the angles are also very small. Therefore, the tangents (that is, angular distortion, A_d) are shown as angles on Figure 4-2.



Span	Differential Settlement, Δ_d	Angular Distortion, $A_d = \Delta_d/L_S$
1	$\Delta_{d1} = \lvert S_{A1} - S_{P1}\rvert$	$A_{d1} = (\lvert S_{A1} - S_{P1}\rvert)/L_{S1}$
2	$\Delta_{d2} = \lvert S_{P1} - S_{P2}\rvert$	$A_{d2} = (\lvert S_{P1} - S_{P2}\rvert)/L_{S2}$
3	$\Delta_{d3} = \lvert S_{P2} - S_{P3}\rvert$	$A_{d3} = (\lvert S_{P2} - S_{P3}\rvert)/L_{S3}$
4	$\Delta_{d4} = \lvert S_{P3} - S_{A2}\rvert$	$A_{d4} = (\lvert S_{P3} - S_{A2}\rvert)/L_{S4}$

Figure 4-2. Concept of Total Settlement, *S*, Differential Settlement, Δ_d, and Angular Distortion, A_d, in Bridges

When spans are continuous over supports, differential settlements induce bending moments and shear in the bridge superstructure and potentially cause structural damage. For a continuous-span beam, the fixed-end bending moment, M_Δ, induced by a differential settlement, Δ_d, can be computed by using Equation 4-1.

$$M_\Delta = \frac{6EI\Delta_d}{L_S^2} \tag{4-1}$$

where, E is the elastic modulus and I is moment of inertia of a prismatic beam with a span length, L_S. Equation 4-1 can be re-written as follows:

$$M_{\Delta} = 6\left(\frac{EI}{L_S}\right)\frac{\Delta_d}{L_S} \tag{4-2}$$

Most equations for moments in beams can be re-arranged in a format similar to that shown in Equation 4-2. Equation 4-3 shows a generalized format of a moment equation for beams.

$$M_{\Delta} = func\left(\frac{EI}{L_S}, \frac{\Delta_d}{L_S}\right) \tag{4-3}$$

The term EI/L_S is a representation of the stiffness of the superstructure over the span length L_S, while the term Δ_d/L_S is the angular distortion as discussed earlier. Clearly, the induced moment is not only a direct function of the differential settlement but also the stiffness. Similar considerations also apply to induced shear due to differential settlement.

Depending on factors such as the type of superstructure, the connections between the superstructure and substructure units, and the span lengths and widths, the magnitudes of differential settlement that can cause damage to the bridge structure can vary significantly. For example, the damage to the bridge structure because of a differential settlement of 2 inches (in.) over a 50-foot span is likely to be more severe than the same amount of differential settlement over a 150-foot span. Because the induced force effect (for example, moment) is a direct function of EI/L_S for all bridges, stiffness should be appropriate to the considered limit state. Similarly, the effects of continuity with the substructure should be considered. In assessing the structural implications of foundation movements of concrete bridges, the determination of the stiffness of the bridge components should consider the effects of cracking, creep, and other inelastic responses. To a lesser extent, differential settlements can also cause damage to a simple-span bridge. However, the major concern with simple-span bridges is the quality of the riding surface (rideability), adverse deck drainage, and aesthetics. Because of 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 that can also potentially affect the safety of the traveling public.

Chapter 5. Tolerable Foundation Movement Criteria

5.1 Tolerable Vertical Movement Criteria

As discussed in Chapter 4, uneven displacements of bridge abutments and pier foundations can affect the ride quality (rideability), functioning of deck drainage, and the safety of the traveling public as well as the structural integrity and aesthetics of the bridge. Such movements often lead to costly maintenance and repair measures. In contrast, overly conservative criteria can be wasteful by leading to (a) foundations that are larger than needed, or (b) choice of a less economical foundation type (such as using a deep foundation at a location where a shallow foundation would be adequate). To determine the optimum solution for movement criteria, collaboration between the geotechnical engineer and the structural engineer is needed.

Within the context of foundation movement, the geotechnical limit states can be broadly categorized into vertical and horizontal movements for any foundation type (for example, spread footings, driven piles, drilled shafts, and micropiles). Agencies often limit the movement to values of 1 in. or less without any rational basis. The literature survey performed as part of the TRB's SHRP2 Project R19B (Kulicki et al., 2015) revealed that the only definitive rational guidance related to the effect of foundation movements on bridge structures is based on a report by Moulton et al. (1985) (Moulton). From an evaluation of 314 bridges nationwide, the report offered 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."

Another study (Wahls, 1983), states 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 in. (50 to 100 millimeters [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. Finally, horizontal movements in excess of 2 in. (50 millimeters) appear likely to cause structural distress. The potential for horizontal movements of abutments and piers should be considered more carefully than is done in current practice."

Based on the data from these two studies, Article 10.5.2.2 of the *AASHTO LRFD* included the guidance summarized in Table 5-1 for the evaluation of tolerable vertical movements in terms of angular distortions. AASHTO (2002) includes the same criteria, which means these criteria are not new in LRFD specifications but can be traced back to AASHTO (2002), based on ASD and LFD platform and to the work by Moulton.

Table 5-1. Tolerable Movement Criteria for Highway Bridges (*AASHTO LRFD*)

Limiting Angular Distortion, Δ_d/L_S (radians)	Type of Bridge
0.004	Multiple-span (continuous span) bridges
0.008	Simple-span bridges

The criteria in Table 5-1 suggest that for a 100-foot span, a differential settlement of 4.8 in. is acceptable for a continuous span, and 9.6 in. is acceptable for a simple span. These relatively large values of differential settlement concern structural designers, who often arbitrarily limit tolerable movements to one-half to one-quarter or one less order of magnitude (for example, 0.0004 instead of 0.004) of the values listed in Table 5-1 or develop guidance as shown in Table 5-2.

Table 5-2. Tolerable Movement Criteria for Highway Bridges (after Washington State Department of Transportation, 2015)

Total Settlement at Pier or Abutment	Differential Settlement over 100 feet within Pier or Abutments and Differential Settlement Between Piers (Implied Limiting Angular Distortion, radians)	Action
$S \leq 1"$	$\Delta_{d100'} \leq 0.75"$ (0.000625)	Design and construct
$1" < S \leq 4"$	$0.75" < \Delta_{d100'} \leq 3"$ (0.000625-0.0025)	Ensure structure can tolerate settlement
$S > 4"$	$\Delta_{d100'} > 3"$ (> 0.0025)	Need Department approval

Notes:
S = settlement (vertical movement)
< = less than
> = greater than
≤ = less than or equal to
' = feet
" = inches

Another example of the use of more stringent criteria is from the Arizona Department of Transportation (ADOT) Bridge Design Guidelines, Chapter 10 (ADOT, 2017), which states the following:

> "The bridge designer should limit the settlement of a foundation per 100 ft span to 0.75 in. Linear interpolation should be used for other span lengths. Higher settlements may be used when the superstructure is adequately designed for such settlements. Any settlement that is in excess of 4.0 in, including stage construction settlements if applicable, must be approved by the ADOT Bridge Group. The designer shall also check other factors, which may be adversely affected by foundation settlements, such as rideability, vertical clearance, and aesthetics."

The ADOT guidelines provide additional guidance in terms of the *S*-0 and construction-point concepts that are discussed later in this report. ADOT also provides guidance on consideration of creep as part of the evaluation of the effect of foundation movements on bridge structures.

While from the structural integrity viewpoint there are no technical reasons for structural designers to set arbitrary additional limits to the criteria listed in Table 5-1, there are often practical reasons based on the tolerable limits of movement of other structures associated with a bridge (for example, approach slabs, wingwalls, pavement structures, drainage grades, utilities on the bridge, and movements that adversely affect ride quality). The relatively large differential settlements based on Table 5-1 should be considered in conjunction with functional or performance criteria not only for the bridge structure itself but also for all associated facilities. Samtani and Nowatzki (2006) suggest the following steps:

Step 1: Identify all possible facilities associated with the bridge structure and the movement tolerance of those facilities. An example of a facility on a bridge is a utility (such as gas, power, and water). The facility owners can identify the movement tolerance of their facility. Alternatively, the facility owners should design their facilities for the movement anticipated for the bridge structure.

Step 2: Because of the inherent uncertainty associated with estimated values of settlement, determine the differential settlement by using conservative assumptions for geomaterial properties and prediction methods. It is important that the estimation of angular distortion be based on a realistic evaluation of the construction sequence and the magnitude of loads at each stage of the construction sequence.

Step 3: Compare the angular distortion from Step 2 with the various tolerances identified in Step 1 and in Table 5-1. Based on this comparison, identify the critical component of the facility. Review this critical component to check whether it can be relocated or whether it can be redesigned to more relaxed tolerances. Repeat this process as necessary for other facilities. In some cases, a simple re-sequencing of the facility construction based on the construction sequence of the bridge structure may help mitigate the issues associated with intolerable movements.

This three-step approach can be used to develop project-specific limiting angular distortion criteria that may differ from the general guidelines listed in Table 5-1. For example, if a compressed gas line is fixed to a simple-span bridge deck and the gas line can tolerate an angular distortion of only 0.002, then the utility will limit the angular distortion value for the bridge structure, not the criterion listed in Table 5-1. However, this problem is typically avoided by providing flexible joints along the utility such that it does not control the bridge design.

5.2 Tolerable Horizontal Movement Criteria

Horizontal movements cause more severe and widespread problems for highway bridge structures than equal magnitudes of vertical movement. Tolerance of the superstructure to horizontal (lateral) movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects. Moulton found that horizontal movements less than 1 in. were almost always reported as being tolerable, while horizontal movements greater than 2 in. were typically considered to be intolerable. Based on this observation, Moulton recommended that horizontal movements be limited to 1.5 in. The data presented by Moulton show that horizontal movements resulted in more damage when accompanied by settlement than when occurring alone.

5.3 Perspective on Tolerable Movements

The AASHTO criteria are based on work done by Moulton that was based on the following:

- 12th Edition (1977) of AASHTO *Standard Specifications for Highway Bridges*. This version of AASHTO specifications used the ASD platform and HS20-44 truck loading or its equivalent lane loading and a tandem axle load for live loads.
- The use of the following tolerable movements definition that is in accordance with TRB Committee A2K03 (Foundations of Bridges and Other Structures, currently Committee AFS30) in mid 1970s:

  "Movement is <u>not</u> tolerable if damage requires costly maintenance and/or repairs <u>and</u> a more expensive construction to avoid this would have been preferable."

The base definition of tolerable movements that was used is subjective, and the work is dated based on an old edition of AASHTO *Standard Specifications for Highway Bridges*, which was not calibrated based on reliability concepts such as the current LRFD specifications. Additionally, Moulton indicates that attempts to establish tolerable movements from the effects of differential settlement analyses on the stresses in bridges significantly underestimated the criteria established from field observations. One reason Moulton attributed the discrepancy between analytical studies and field observations is that the analytical studies often do not account for the construction time of a structure and that components of the foundation movement estimated based on analytical studies have already occurred before the structure completion. Structure portions (for example, the bridge superstructure) that are constructed last do not have damage consistent with the level predicted by analytical studies that assume all loads are applied instantaneously. Another reason supporting Moulton's observations is that building materials such as concrete (especially while it is curing) are able to undergo a considerable amount of stress relaxation when subjected to movements. Under conditions of very slowly imposed movements, the effective value of the Young's modulus of concrete is considerably lower [due to creep] than the value for rapid loading (Barker et al., 1991).

All of the previously described considerations were recognized by Moulton. Since the 1990s, valuable data have been collected that help quantify the amount of movements that occurs as bridge structures are constructed. These data have led to the formulation of the construction-point concept in FHWA documents (for example, Samtani and Nowatzki, 2006) and is also discussed in Chapter 6. At a minimum, adoption of the construction-point concept in the bridge design process will be a significant step in the right direction toward comparing estimated foundation movements with AASHTO criteria for tolerable movements.

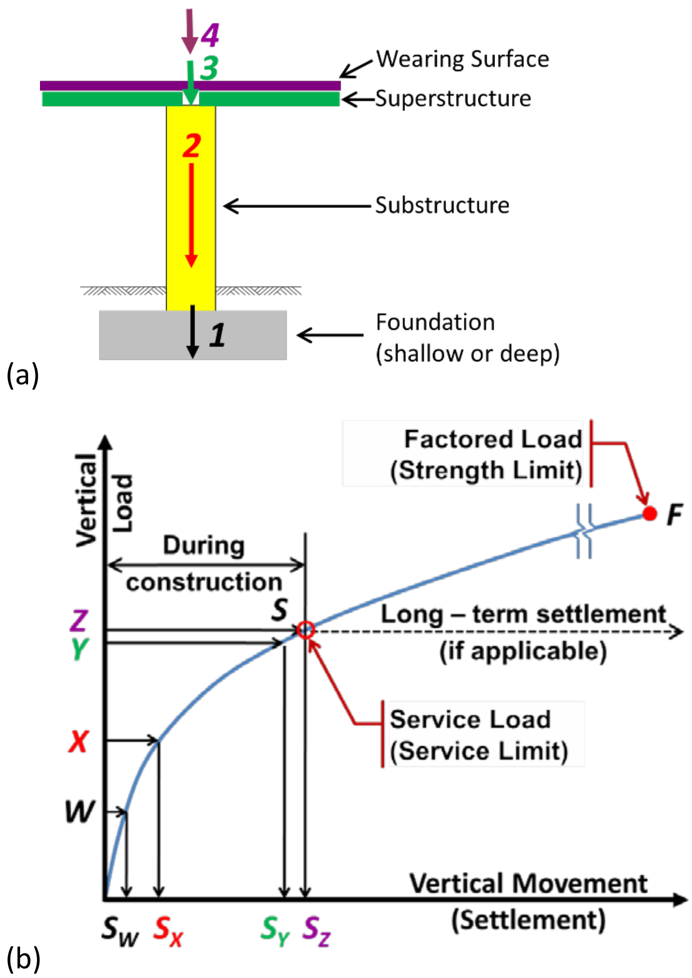
Chapter 6. Construction-Point Concept

6.1 Vertical Movement (Settlement)

Most designers analyze foundation movements as if a weightless bridge structure were instantaneously set into place and all the loads were applied at the same time. In reality, loads are applied gradually as construction proceeds, and settlements also occur gradually as construction proceeds. Several critical construction points or stages during construction should be evaluated separately by the designer. Figure 6-1 shows the critical construction stages and their associated load-movement behavior. Formulation of settlements as shown on Figure 6-1 would permit an assessment of settlements up to that point that can affect the bridge superstructure. For example, 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 *X* and *Z* are the most relevant.

Studies such as Sargand et al. (1999) and Sargand and Masada (2006) have documented data that indicate that the percentage of settlement between placement of beams and end-of-construction is generally between 25 and 75 percent of the total settlement, depending on the superstructure type and the construction sequence. This is a significant observation; therefore, it is recommended that the limit state of vertical movements (that is, settlements), and its implications should be evaluated using the construction-point concept. This observation applies to all other movements (for example, lateral and rotation).

While using the construction-point concept, it is important that various quantities are being measured at discrete construction points and that 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 foundations are founded on saturated clay deposits or if a layer of saturated clay falls within the zone of stress influence below the foundation, even though the foundation 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 on Figure 6-1. Continued settlements during the service life of the structure will tend to reduce the vertical clearance under the bridge with associated problems of over-height vehicles impacting the bridge superstructure. 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. A key point in evaluating settlements at critical construction points is that the approach requires close coordination between the structural and geotechnical specialists. More detailed discussion is included in Samtani et al. (2010).

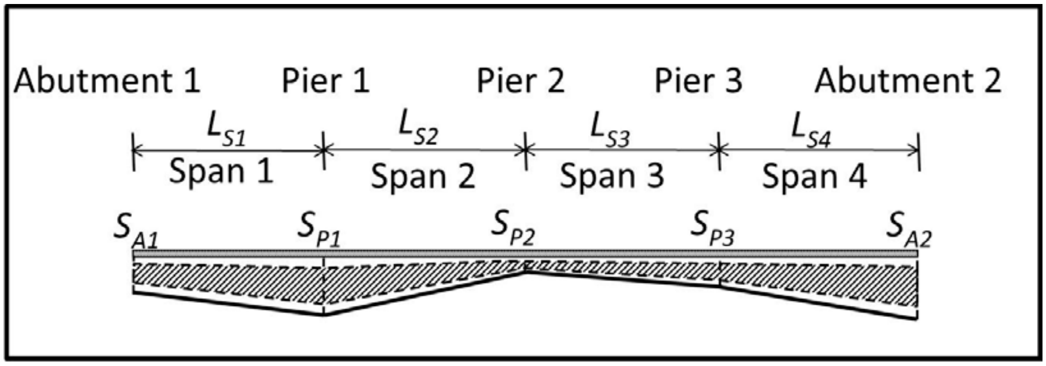


Legend:

W	Load after foundation construction (= Load *1*)
X	Load after pier column/wall construction (= Load *1* + Load *2*)
Y	Load after superstructure construction (= Load *1* + Load *2* + Load *3*)
Z	Load after wearing surface construction (= Load *1* + Load *2* + Load *3* + Load *4*)
S	Service load (service limit)
F	Factored load (strength limit)
S_W	Settlement under load *W*
S_X	Settlement under load *X*
S_Y	Settlement under load *Y*
S_Z	Settlement under load *Z*

Figure 6-1. Construction-point Concept for a Bridge Pier. (a) Identification of Critical Construction Points, (b) Conceptual load-Displacement Pattern for a Given Foundation

With respect to the example of the four-span bridge shown on Figure 4-2, the use of the construction-point concept would result in smaller settlement to be considered in the structural design. Figure 6-2 shows a comparison of the profiles of the calculated settlements (solid lines) and the actual relevant settlements (hatched pattern zones) based on the construction-point concept. The range of the hatched pattern zone can be from 25 to 75 percent of the total settlement values (solid line), as previously discussed. For a given project and site-specific conditions, the actual relevant settlement profile will be within the hatched pattern zone.



Legend:

Calculated total settlement profile (refer to Figure 4-2)

Range of relevant settlement profile using construction-point concept

Figure 6-2. Relevant Angular Distortion in Bridges Based on Construction-point Concept

6.2 Horizontal Movements

Horizontal movements generally occur because of sliding and/or rotation of the foundation. Moulton indicates that horizontal movements cause more severe and widespread problems than do equal magnitudes of vertical movement. The most common location of horizontal movements is at the abutments, which are subject to lateral earth pressure. Horizontal movements can also occur at the piers because of lateral loads and moments at the top of the substructure unit. The estimation of the magnitudes of horizontal movements should take into account the movements associated with lateral squeeze as discussed in Samtani and Nowatzki (2006) and Samtani et al. (2010). Lateral movements from lateral squeeze can be estimated by geotechnical specialists, while lateral movements from sliding or lateral movements of deep foundations can be estimated by structural specialists based on input from geotechnical specialists. The limiting horizontal movements are strongly dependent on the type of superstructure and the connection with that substructure; therefore, the tolerable horizontal movements are project-specific.

Chapter 7. Reliability of Predicted Foundation Movements

All analytical methods (models) for predicting foundation movements have some degree of uncertainty. The reliability of predicted foundation movements varies as a function of the chosen analytical method. Because the induced force effects (for example, moments) are a direct function of foundation movements, the values of the induced force effects are only as reliable as the estimates of the foundation movements. It is important to quantify the uncertainty in foundation movements by calibrating the analytical method used to predict the foundation movements using stochastic procedures. In the LRFD framework, the uncertainty is calibrated through use of load and/or resistance factors. As discussed in Chapter 2, the *AASHTO LRFD* considers uncertainty of foundation movements in terms of the induced effects through the use of *SE* load factor, γ_{SE}. The calibration procedure of *SE* load factor is discussed Chapter 8.

Chapter 8. Calibration Procedures

This chapter describes procedures that can be used for the calibration of limit states to evaluate the effect of vertical or horizontal movements of all structural foundation types such as footings, drilled shafts, and driven piles. The effect of foundation movements on the bridge superstructures is discussed in the context of construction-point concept. The implementation of the calibration procedure is demonstrated in Chapter 9, by using the case of immediate settlements of spread footings.

8.1 Relevant *AASHTO LRFD* Articles for Foundation Movements

Within the context of foundation movement, the geotechnical limit states can be broadly categorized into vertical and horizontal movements for any foundation type (for example, spread footings, driven piles, drilled shafts, and micropiles). Table 8-1 summarizes the various relevant articles in the *AASHTO LRFD* that address vertical (settlement) and horizontal movements for various types of structural foundations.

Table 8-1. Summary of *AASHTO LRFD* Articles for Estimation of Vertical and Horizontal Movements of Structural Foundations

***AASHTO LRFD* Article**	**Comment**
10.6.2.4: Settlement Analyses for Spread Footings	Article 10.6.2.4 presents methods to estimate the settlement of spread footings. Settlement analysis is based on the elastic and semi-empirical Hough (1959) (Hough) method for immediate settlement and the 1-D consolidation method for long-term settlement.
10.7.2.3: Settlement (related to driven pile groups) 10.8.2.2: Settlement (related to drilled shaft groups) 10.9.2.3: Settlement (related to micropile groups)	The procedures in these Articles (10.7.2.3, 10.8.2.2 and 10.9.2.3) refer to the settlement analysis for an equivalent spread footing (see *AASHTO LRFD*, Figure 10.7.2.3.1-1).
10.7.2.4: Horizontal Pile Foundation Movement 10.8.2.4: Horizontal Movement of Shaft and Shaft Groups 10.9.2.4: Horizontal Micropile Foundation Movement	Lateral analysis based on the *P-y* method is included in the *AASHTO LRFD* for estimating horizontal (lateral) movements of deep foundations. Use of the strain wedge method is allowed per 10.7.2.4.

Note: Section 11 (Abutments, Piers and Walls), Article 11.6.2 of the *AASHTO LRFD* refers to the various Articles noted in the left column of this table; therefore, the Articles shown in this table also apply to fill retaining walls and their foundations.

8.2 Overarching Characteristics to be Considered

For limit states that deal with movements, there are some overarching characteristics in terms of cause (load) and consequences to the bridge structure if limit states are exceeded that must be addressed. Additionally, the reliability of foundation movements must be consistent with the level of reliability that is considered in the structural service limit states.

8.2.1 Consequences of Exceeding Movement-related Limit States and the Effect on Target Reliability Indices

For strength (or ultimate) limit states, reliability index values in the range of 3.0 to 3.5 are used. Strength limit states pertain to structural safety and the loss of load-carrying capacity. In contrast, service limit states are user-defined limiting conditions that affect the function of the structure under expected service conditions. Violation of service limit states occurs at loads much smaller than those for strength limit states. Because there is no danger of collapse if a service limit state is violated, a smaller value of target reliability index may be used for service limit states. In the case of foundation movement such as settlement, the structural force effect is manifested in increased moments and shears and potential cracking. The force effect due to the settlement would generally be small relative to the force effect; this is due to dead and live loads because the *SE* load factor that represents the uncertainty in estimated settlement is only one of many load factors in all the limit state load combinations. The primary moments due to the sum of dead and live loads are usually much larger than the additional (secondary) moments because of settlement.

Based on these considerations and consideration of reversible and irreversible service limit states (as discussed in Section 8.2.3) for bridge superstructures, a target reliability index, $\beta_{T,}$ in the range of 0.50 to 1.00 for calibration of *SE* load factor for foundation movement in the Service I limit state was used in the SHRP2 Project R19B.

The following factors should be considered to differentiate among various service limit states according to the consequences of exceedance:

- Whether the limit state is reversible or irreversible as defined in Section 8.2.3: Irreversible limit states may have higher target reliability than reversible limit states because the consequences may be more critical. Reversible-irreversible limit states may have target reliability similar to reversible limit states.
- Relative cost of repairs: Limit states that have the potential to cause damage that is costly to repair may have higher target reliability than limit states that have the potential of causing only minor damage.

8.2.2 Load-driven versus Non-load-driven Limit States

The difference between load-driven and non-load-driven limit states is in the degree of involvement of externally applied load components in the formulation of the limit state function. In the load-driven limit states, the damage occurs because of applications of external loads, usually live load (trucks). Examples of load-driven limit states include decompression and cracking of prestressed concrete and vibrations or deflection. In contrast, in non-load-driven limit states, the damage occurs because of deterioration or degradation over time and aggressive environment or as inherent behavior from certain material properties. Examples of non-load-driven limit states include penetration of chlorides leading to corrosion of reinforcement, leaking joints leading to corrosion under the joints, shrinkage cracking of concrete components, and corrosion and degradation of reinforcements in reinforced soil structures (such as mechanically stabilized earth [MSE] walls). In these examples, the external load occurrence plays a secondary role. In case of foundation movements, the computations are usually performed with consideration of live load (load-driven) for short-term movements but without consideration of live load for long-term or time-dependent movements.

8.2.3 Reversible versus Irreversible Limit States

The damage caused by exceeding limit states may be reversible or irreversible; therefore, the cost of repair may vary significantly. Hence, limit states may be categorized as reversible and irreversible. Reversible limit states are those for which no consequences remain once a load is removed from a structure. Irreversible limit states are those for which consequences remain.

An extended concept is that of reversible-irreversible limit states, where the effect of an irreversible limit state may be reversed by intervention. An example of this concept is foundation settlement, which is an irreversible limit state with respect to the foundation elements but may be reversible in terms of its effect on the bridge superstructure through intervention (for example, through shimming or jacking).

Because of their reduced service implications, irreversible limit states, which do not concern the safety of the traveling public, are calibrated to a higher probability of failure and a corresponding lower reliability index than the strength limit states. Reversible limit states are calibrated to an even lower reliability index.

8.3 Calculation Models

While considering limit states from movements, the load-movement characteristics of the structure or its member are important to understand because the resistance must now be quantified as a function of the movement. This section discusses the extension of the *AASHTO LRFD* framework to incorporate the load-movement behavior. This section also presents a calibration framework for foundation movements. The proposed step-by-step procedure for calibration is described in Section 8.3.5, which leads to a load factor for movements based on the target reliability index that was discussed in Section 8.2.1. This procedure is demonstrated by an example for immediate settlements of spread footings using various analytical methods in Chapter 9.

8.3.1 Incorporation of Load-movement (Q-δ) Characteristics in the *AASHTO LRFD* Framework

The basic *AASHTO LRFD* framework in terms of distributions of loads and resistances is shown on Figure 8-1, where

Q	=	load
Q_{mean}	=	mean load
Q_n	=	nominal load
λ_Q	=	Bias factor for load
γ	=	load factor
R	=	resistance
R_{mean}	=	mean resistance
R_n	=	nominal resistance
λ_R	=	Bias factor for resistance
ϕ	=	resistance factor
f	=	frequency

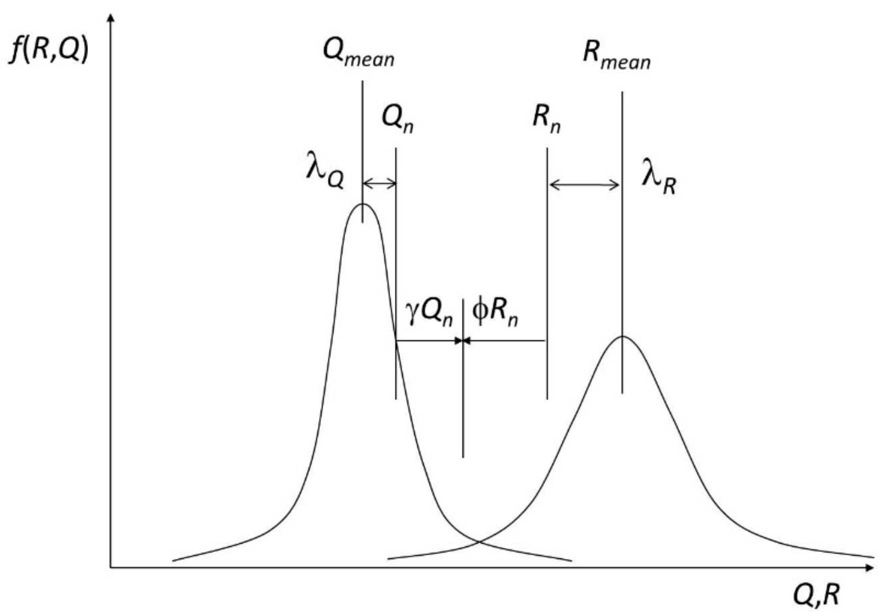


Figure 8-1. Basic *AASHTO LRFD* Framework for Loads and Resistances

Details of the *AASHTO LRFD* framework can be found in Nowak and Collins (2000). Appendix B provides an overview of the processes used to establish the *AASHTO LRFD* framework. Strength limit states were evaluated by using this framework. Determination of movement is a necessary part of the evaluation of serviceability. Therefore, for the evaluation of movements, the basic *AASHTO LRFD* framework shown on Figure 8-1 needs to be modified to include load-movement or Q-δ behavior. The Q-δ behavior can be considered to be another dimension of the basic *AASHTO LRFD* framework as shown on Figure 8-2, where

δ = movement

δ_S = movement at nominal load, Q_n

δ_F = movement at factored load, $Q_F = \gamma(Q_n)$

δ_N = movement at load corresponding to nominal resistance, R_n

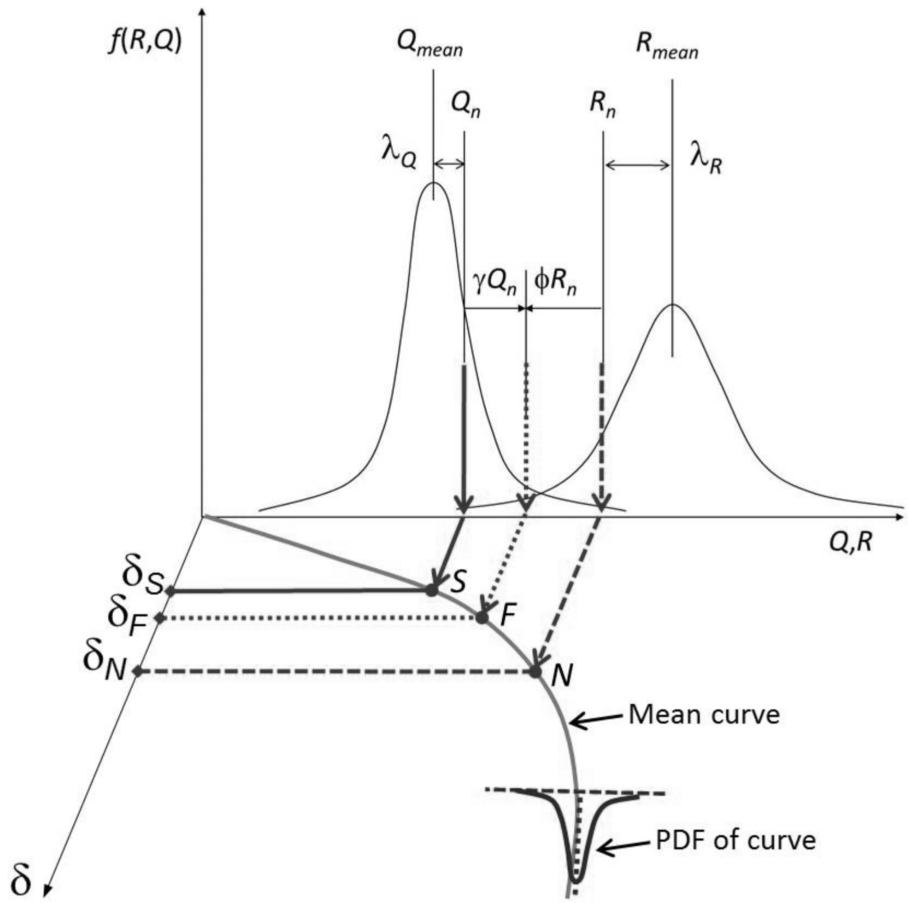


Figure 8-2. Incorporation of *Q*-δ Mechanism into the Basic *AASHTO LRFD* Framework

Although *Q*-δ curves can have many different shapes, for illustration purposes, a strain hardening curve is shown on Figure 8-2. For discussion purposes, the mean *Q*-δ curve is shown, and the spread of the *Q*-δ data about the mean curve is represented schematically by a probability distribution function (PDF) that is discussed later in this report. The various relevant load and movement quantities shown in the *Q*-δ space on Figure 8-2 are shown in the regular first quadrant of the two-dimensional plot on Figure 8-3. Note that nominal resistance is compared to nominal load to assess safety.

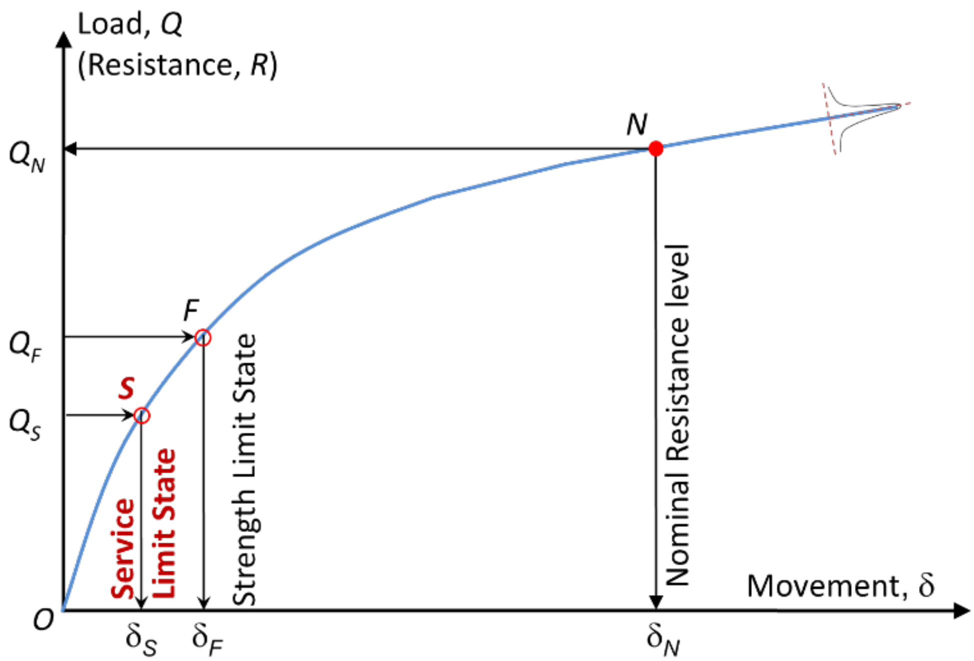


Figure 8-3. Significant Points of Interest on the Mean *Q*-δ Curve

Figure 8-2 combines a number of different aspects of material behavior that covers both loads and resistances. It is important to understand the inter-relationships among the various parameters displayed on the curves. The following points are made:

- The load-movement (*Q*-δ) curves shown on Figure 8-2 and Figure 8-3 represent the measured mean curves based on field measurements.
- Field measurements have upper and lower bounds with respect to the mean of the measured data. These bounds are shown schematically on Figure 8-4 and also on Figure 8-2 and Figure 8-3 through a PDF. Although PDFs for normal distributions are shown, the spread of the data along the mean may be represented by normal or nonnormal distributions, as appropriate. The spread of the data around the mean curve generally increases with increasing movements.
- Many theoretical methods are used to predict the load-movement behavior. The theoretical models may predict a stiffer or softer material response compared to the actual response. An example of "softer" material behavior is shown on Figure 8-5.
- Because Bias is defined as the ratio of measured to predicted values, the Bias for movements will vary over the full range of the *Q*-δ curve. In other words, the predicted values may be larger or smaller than the measured values and vice versa.

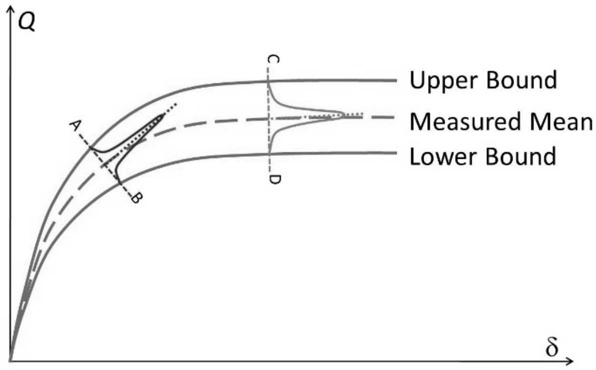


Figure 8-4. Range and Distribution along a *Q*-δ Curve

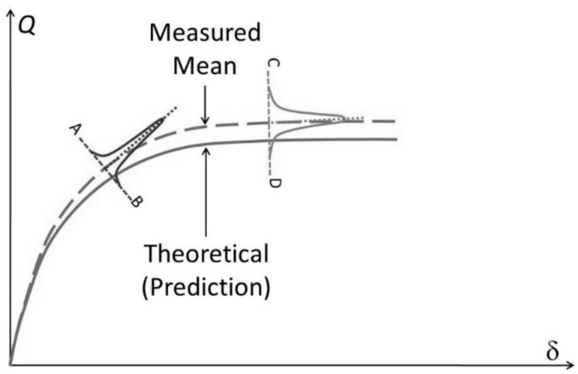


Figure 8-5. Example Relationship of Measured Mean with Theoretical Prediction

8.3.2 Consideration of Bias in Calibration of Movements

A varying Bias along the *Q*-δ curve, although a reality, can be difficult to handle in the calibration process. However, the problem is made easier by realizing that for calibration of movement, the force effects between Points *O* and *S* as shown on Figure 8-3 are of primary interest. Point *S* represents the service force effects, and the movements corresponding to this point are of primary interest. Because the Bias will generally increase with increasing movements, the value of the Bias at Point *S* will be the maximum between Point *O* and *S*, and the use of the Bias at Point *S* will be conservative. In this context, the Bias at Point *S* is most relevant and, at a minimum, field data under full service loads are of importance in movement calibrations. The data needed for movement evaluations are the full range of incremental loads and movements measured on in-service structures from the beginning of construction of the first element (for example, the foundation) to the completion of the roadway and beyond. These data will help in evaluating the variability in predicted movements for structural, as well as geotechnical, features. Currently, these types of data are not routinely available; however, programs such as the FHWA Long-Term Bridge Performance Program may offer a good avenue to collect such data.

8.3.3 Application of *Q*-δ Curves in the *AASHTO LRFD* Framework

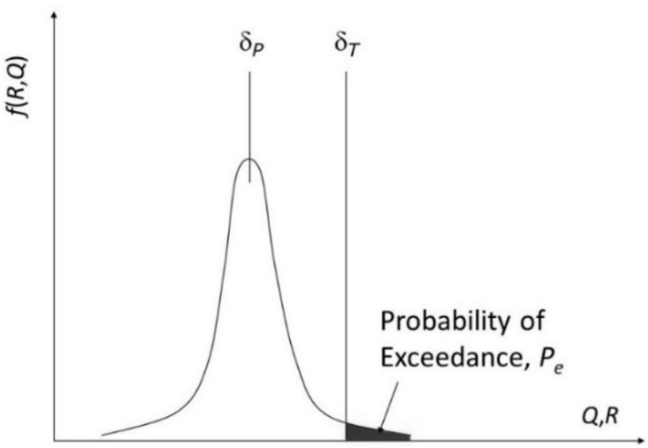
The calibration of the strength limit state in the *AASHTO LRFD* was performed by using the general concepts on Figure 8-1 and as summarized in Appendix B. This approach presumes that statistical data are available to quantify the spread of the force effects and resistances. In the context of movements, tolerable movements (δ_T) can be considered as resistances, while the predicted movements (δ_P) can be considered as loads. Thus, a limit state function (g) can be written as follows:

$$g = \delta_T - \delta_P \tag{8-1}$$

Once the movements are expressed in the form of a limit state, probabilistic calibration processes similar to those used for the strength limit state can be used. For strength limit states, the Monte Carlo analysis is often used for calibrations. One of the assumptions of the Monte Carlo procedure is that PDFs for both the load (Q) and resistance (R) are available. However, for movement calibration, there are practical limitations to this approach. Although the statistical data for modeling the uncertainty in predicted movements, δ_P, are available, the same is not true for tolerable movements, δ_T. Some attempts have been made (Zhang and Ng, 2005) to evaluate the distribution of tolerable movements, but from a geotechnical viewpoint, it may not be possible to obtain a PDF for tolerable movement that is applicable to the various structural service limit states mentioned in Chapter 1. This is largely because it is virtually impossible to identify a consistent tolerable movement across all elements of a structure. Many variables such as type of bridge structure (for example, simple span or continuous span) and bearing types as well as the consequence (for example, cracking) under consideration can affect the value of tolerable movement for a given element.

To bypass the difficulties associated with choice of a tolerable movement, a single deterministic value of tolerable movement, δ_T, is often used for comparison against the potential spread of data for predicted movements, δ_P. In practical terms, a bridge engineer often assumes a deterministic tolerable movement that would serve to limit the amount of superstructure movement that would be considered. In this case, the Monte Carlo calibration process becomes difficult because there would be a PDF for load (Q) but a deterministic value for resistance (R). To use Monte Carlo in this situation, an arbitrarily small value of standard deviation, or coefficient of variation (COV), would have to be used. Although theoretically possible, this process could lead to spurious results. In the future, if adequate high-quality data are available from the Long-Term Bridge Performance Program for defining the distribution statistics of tolerable movements (δ_T), then the use of Monte Carlo analysis may be possible. In the meantime, an alternative formulation for calibration of movements is necessary.

When a deterministic value for δ_T is used, then using Figure 8-1 as the basis, the resistance PDF is reduced to a single value while the load PDF can be used to represent the predicted movements. This modified treatment for movements is shown on Figure 8-6. In this approach, the probability of exceedance (P_e) for the predicted movements to exceed the tolerable movement is given by the area of the overlap of the two curves (the shaded zone shown on Figure 8-6). Because the goal is to prevent movement-related problems, P_e can be selected based on the acceptable value of target reliability index (β_T). The ratio δ_T/δ_P can be thought of as a load factor, γ_{SE}, for movements for a given probability of exceedance, P_e, corresponding to a target reliability index, β_T.



Q = force effect
R = resistance
δ_P = predicted movements (force effect)
δ_T = deterministic value of tolerable movement (resistance)

Figure 8-6. Relationship of Deterministic Value of tolerable Movement, δ_T, and a Probability Distribution Function for Predicted Movement, δ_P

By looking at the representation of load (δ_P) and resistance (δ_T) it is tempting to think that the load factor for movements could be determined using the equal load probability (ELP) formulation summarized in Appendix B that was used for developing load factors for strength limit state. However, for the reasons explained in Appendix B.3, the ELP formulation may not be applicable for the case of foundation movements. One of the key considerations in calibration of foundation movements is the recognition that the horizontal distance between δ_T and the mean value of δ_P as shown on Figure 8-6 is not fixed but variable due to the following reasons:

- The values of predicted movement (δ_P) are a function of the analytical model of prediction and local geology (nature). Thus, the uncertainty in the predicted movement (δ_P) is a function of nature and the chosen analytical model for prediction of movements. In other words, for the same load and same foundation configuration, the chosen analytical model will have different levels of uncertainty in different geologic formations; for example, the Hough method of predicting immediate settlement of a spread footing may predict values closer to actual (measured) values in one part of the United States compared to another.

- The values of the tolerable movement (δ_T) are a function of the bridge type and its components such as girders, bearings, and foundations. In reality, the chosen value for the tolerable movement (δ_T) is often based on intangibles such as the use of arbitrary criteria (for example, the settlement shall not exceed 1 in.) established by an owner, the judgment of the designer based on past experience, and/or simply some sort of comfort level based on basic human instinct for conservativeness.

This means that on Figure 8-6 the relative positions of the PDF for predicted movement (δ_P) and the line representing a deterministic value of tolerable movement (δ_T) varies; that is, two moving targets need to be accounted for in the reliability analysis to achieve a chosen target reliability, β_T. Thus, the *SE* load factor for movements, γ_{SE}, needs to account for a relationship between three independent variables: δ_P, δ_T, and β_T. Such a three-way relationship makes the determination of the *SE* load factor for movements difficult. The following steps can be used to reduce the number of variables and resolve this difficulty.

1. Establish a target reliability index, β_T, based on structural service limit states. This reduces the number of variables from three to two.
2. Express δ_P in terms of δ_T, by using a ratio, δ_P/δ_T or δ_T/δ_P. This ratio reduces the remaining two variables to a single random variable expressed by the ratio that can then be modeled with an appropriate PDF.
3. Express a limit state, g, in terms of the ratio from Step 2; for example, $g = \delta_P/\delta_T < 1$ or $g = \delta_T/\delta_P > 1$ to ensure that the value of δ_T selected by the bridge designer is not exceeded consistent with the target reliability index, β_T, in Step 1.

Thus, the key to resolving the three-way relationship is to first establish a target reliability index, β_T. Table 8-2 shows the target reliability index, β_T, values for various structural limit states based on the work done as part of the SHRP2 Project R19B and the discussions as part of the proposed ballot items at AASHTO SCOBS meetings. Table 8-2 shows reliability index values much smaller than the typical reliability index values of 3.0 to 3.5 for the strength limit state, which is consistent with earlier discussions in Section 8.2.1. Kulicki et al. (2015) recommend a β_T value of 1.0 for the calibration of foundation movements based on the consideration of irreversible limit states. If the owner commits to reversing the effects of irreversible foundation movements through intervention mechanisms such as shimming or jacking, then consideration could be given to the reduced consequences discussed earlier for reversible-irreversible limit states. For such cases, Kulicki et al. (2015) suggest a lower β_T value of 0.50.

Table 8-2. Target Reliability Index, γ_{SE} for Various Structural Limit States (Kulicki et al., 2015)

Limit State	Target Reliability Index, β_T	Approx P_e (Note 1)
Fatigue I and Fatigue II limit states for steel components	1.0	16%
Fatigue I for compression in concrete and tension in reinforcement	0.9 (Compression) 1.1 (Tension)	18% 14%
Tension in prestressed concrete components	1.0 (Normal environment) 1.2 (Severe environment)	16% 11%
Crack control in decks (Note 2)	1.6 (Class 1) 1.0 (Class 2)	5% 16%
Service II limit state for yielding of steel and for bolt slip (Note 2)	1.8	4%

Note 1: P_e is based on "Normal" distribution.

Note 2: Although smaller values of the reliability index can be used as per the SHRP2 Project R19B, the subcommittees have expressed a desire not to change the values implied by the current standard.

Now that the values of target reliability indices, β_T, are established, Step 2 above requires determination of the ratio of δ_P in terms of δ_T. The value of δ_T is a deterministic criterion that is established by a bridge designer. Thus, the only variable that still needs definition is the PDF for δ_P as shown on Figure 8-6. The PDF for δ_P is developed from the data at Point *S* shown on Figure 8-2 and Figure 8-3. This is where the concept of *Q*-δ curve fits into the formulation for calibration based on movements. Thus, any model that can predict a *Q*-δ curve can be used in the conventional *AASHTO LRFD* framework as long as the data at Point *S* corresponding to service limit state force effects are available through field measurements. The effect of material brittleness (or ductility) and deterioration aspects (see Section 8.3.5) can now be introduced in the *AASHTO LRFD* framework through the use of an appropriate *Q*-δ model. Examples of *Q*-δ models are stress-strain curves, vertical load-settlement curves for foundations, *P-y* (lateral load-lateral displacement) curves for laterally loaded piles, shear force-shear strain curves, and moment-curvature curves. The proposed formulation can incorporate any *Q*-δ model and is therefore a general formulation that is applicable to structural or geotechnical aspects.

In summary, the above three-step process helps achieve the goal stated in Section 8.2 that the reliability of foundation movements must be consistent with the level of reliability that is considered in the structural service limit states. The result of achieving this goal is the mitigation of bridge serviceability (movement-related) problems due to the foundation movements being incorporated into the design process consistent with the target reliability indices in Table 8-2. This goal is also consistent with the goal of the ELP formulation to achieve a uniform level of reliability or, in other words, uniform levels of serviceability and safety.

8.3.4 Formulation for Determination of *SE* Load Factor for Foundation Movements

Based on the general three-step process discussed in Section 8.3.3, Figure 8-7 shows a specific formulation to determine *SE* load factor for foundation movements, γ_{SE}. This formulation is developed as follows:

1. Obtain data for predicted (δ_P) and measured (δ_M) movements for the movement mode of interest (for example, immediate settlement of spread footings, lateral movement of a deep foundation, lateral deflection at top of MSE wall, etc.). Recognize that the value of δ_M can be considered as resistance and equivalent to the tolerable settlement (δ_T).
2. Express the data in Step 1 in terms of ratio $X = \delta_P/\delta_T$. In geotechnical literature (for example, Tan and Duncan [1991] and Sivakugan and Johnson [2002, 2004]), *X* is often referred to as "Accuracy" or settlement ratio, *SR*. *X* is a random variable that can now be modeled by an appropriate (for example, normal, lognormal, etc.) PDF. Express the PDF in terms of corresponding cumulative distribution function, CDF.
3. As shown on Figure 8-7, plot a family of CDF curves for a range of values of tolerable movements (for example, $\delta_{T1} < \delta_{T2} < \delta_{T3}$). The CDFs are generated by multiplying the CDF for Accuracy ($X = \delta_P/\delta_T$) by selected values of tolerable movements (δ_{T1}, δ_{T2}, δ_{T3}). The plot shown on Figure 8-7 is referred to as a probability exceedance chart (PEC). For a given predicted movement, δ_P, the PEC permits the determination of values of the probability of exceedance (P_e) for a range of values of tolerable movements, δ_T.
4. Select the value of probability of exceedance (P_{eT}) corresponding to the target reliability index (β_T), and determine the value of δ_T for a given value of δ_P, as shown on Figure 8-7.
5. Compute the value of the *SE* load factor for movement, $\gamma_{SE} = \delta_T/\delta_P$, as shown on Figure 8-7.

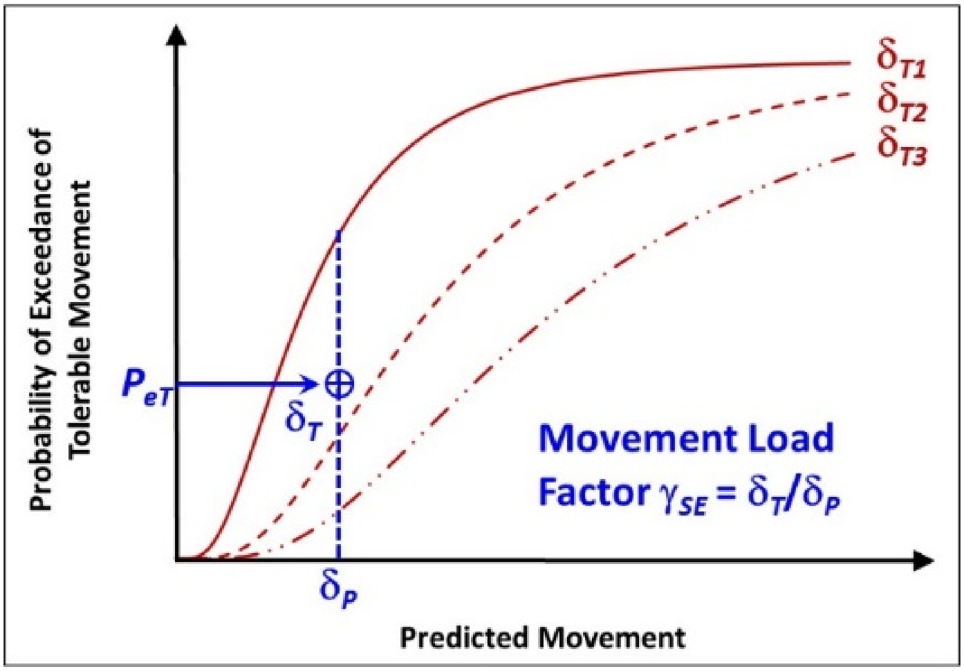


Figure 8-7. Probability Exceedance Chart for Evaluation of Load Factor for a Target Probability of Exceedance (P_{eT}) at the Applicable Service Limit State Combination

The benefit of the calibration formulation as shown on Figure 8-7 is that the *X*-axis is in terms of predicted movement, δ_P, a value that is developed by the designer based on a chosen analytical prediction model, and the various curves for tolerable movement provide flexibility to the designer to select an appropriate value of δ_T consistent with the target reliability index, β_T. Once the designer computes (predicts) a movement for any given movement mechanism, then the designer simply multiplies the predicted movement, δ_P, by the load factor for movement, γ_{SE}, and uses the factored value for evaluation at the applicable service and strength load combinations. Alternatively, the force effects such as moment and shear due to predicted movement, δ_P, can be multiplied by the *SE* load factor for movement, γ_{SE}. This concept is valid whether structural or geotechnical movement mechanisms are evaluated. This formulation is demonstrated by an example in Chapter 9.

The PEC, which forms the basis of the calibration formulation, is essentially a representation of CDF of Accuracy, or *X*. Similar charts are referred to as probabilistic design charts by Das and Sivakugan (2007) and Sivakugan and Johnson (2002, 2004) and artificial neural network charts by Shahin et al. (2002) and Musso and Provenzano (2003). Although not specifically in chart format, similar concepts are also presented in Tan and Duncan (1991) and Duncan (2000). As used here, the PEC permits incorporation of uncertainty in predicted movements and user-specified deterministic tolerable movements in a unified manner. The specific format of PEC that is developed and used here is amenable to correlation to the *AASHTO LRFD*-based concept of target reliability index.

8.3.5 Deterioration of Foundations and Wall Elements

Most, if not all, foundation elements are buried in geomaterials. This is also true for most earth-retaining structures. Therefore, the long-term performance of the foundation and wall elements can be affected by the corrosion and degradation potential of the geomaterials. In such cases, non-load driven serviceability limit states as discussed in Section 8.2.2 may be realized. Hence, consideration of the effect of deterioration of foundations and wall elements on serviceability is important.

The term “corrosion” applies to metal components, while “degradation” applies to non-metal components such as polymeric soil reinforcements in MSE walls. If the geomaterials have significant corrosion or degradation potential, then the sectional properties of the foundation and wall elements will deteriorate by reduction in the section or loss of strength, or both, which results in more movements that can cause serviceability problems. The *AASHTO LRFD* specifications recognize this mode of deterioration and provide definitive guidelines. For example, *AASHTO LRFD* Articles 10.7.5 and 10.9.5 of Section 10 (Foundations) provide guidelines to evaluate the corrosion and deterioration of driven piles and micropiles, respectively. Similarly, *AASHTO LRFD* Section 11 (Abutments, Piers and Walls) provides guidance in Article 11.8.7 for non-gravity cantilevered walls, Article 11.9.7 for anchored walls, and Articles 11.10.2.3.3 and 11.10.6.4 for MSE walls. Supplementary guidance can be found in Elias et al. (2009) and Fishman and Withiam (2011). The *AASHTO LRFD*, Elias et al., and Fishman and Withiam documents cross-reference a number of publications that discuss the corrosion or degradation potential of geomaterials.

In general, the various AASHTO articles and other documents cited provide guidance for testing frequencies and protocols to evaluate the corrosion or degradation potential of various geomaterials. It is assumed that the foundation and wall designer will perform the necessary tests and, as appropriate, implement the necessary mitigation measures to minimize the inevitable effects of corrosion or degradation. The most common approach is to estimate the rate of corrosion or degradation over the design life of the structure and provide additional sectional or strength properties (or both) that will permit the structure to perform within its strength and serviceability requirements. For example, metal elements are often provided additional section based on the anticipated loss of metal over the design life of the structure. Concrete deterioration from sulfate attack is often mitigated by the use of an appropriate type of cement.

The end result of the above described deterioration mechanisms is a detrimental effect that leads to reduction in resistance and associated increased movements and serviceability concerns. As indicated in Section 8.3.3, these deterioration aspects can now be introduced into the *AASHTO LRFD* framework through the use of an appropriate Q-δ model that considers deterioration.

Chapter 9. Calibration Implementation

9.1 General

The *AASHTO LRFD* explicitly includes specific analytical methods to evaluate the movements of various foundation types. For example, for immediate settlement of spread footings, it includes a method by Hough (1959) (Hough). However, some other methods, such as Schmertmann et al. (1978) (Schmertmann), that are recommended by FHWA or another local method may be preferred by an owner based on local regional geologic conditions. Based on the calibration approach included in Section 8.3.3 and Section 8.3.4, this chapter illustrates the calibration implementation to serve as an aid for an owner to perform a calibration of the γ_{SE} load factor for geotechnical features by using an analytical method to predict (estimate) movement based on local geologic conditions. A step-by-step format is provided with the intention that end users can simply substitute the appropriate data for the method and the mode of foundation movement that they are trying to calibrate. The vertical and lateral movements for all structural foundation types, such as footings, drilled shafts, and driven piles can generally be calibrated by using the process described herein. The concept can also be applied to other geotechnical features such as retaining walls (for example, calibration of face movements of MSE walls with inextensible or extensible reinforcements). To demonstrate the calibration process, the immediate vertical settlement of bridge spread footings is used herein.

For convenience, reference is made to the widely used commercial software Microsoft Excel (references to Microsoft Excel herein are applicable to its 2007, 2010, and later versions). This has been done to help simplify the calibration process without complicating the process with esoteric probabilistic principles, which in the end lead to the same result. All figures in this section have been generated using Microsoft Excel.

Table 9-1 summarizes the framework for calibration. Sections 9.2.1 to 9.2.6 demonstrate the application of each step in Table 9-1.

In Table 9-2, the numbers are to the second or third significant digit. However, in tables such as Table 9-3 and Table 9-4, wherein computed data are presented, the numbers are extended to the fifth or sixth significant figure. It is not the intent to imply that the level of accuracy of five to six significant figures is required for statistics. The only reason for this level of reporting is to help researchers verify the final results for load factors with their computational programs (for example, spreadsheets). The final load factors are reported to three significant figures and then further rounded as discussed later in this chapter.

Table 9-1. Basic Framework for Calibration of Movements

Step	Comment
1. Formulate the limit state function and identify basic variables.	Identify the load and resistance parameters and formulate the limit state function. For each considered limit state, establish the acceptability criteria.
2. Identify and select representative structural types and design cases.	Select the representative components and structures to be considered; for example, structural type could be spread footing and the design case may be immediate settlement.
3. Determine load and resistance parameters for the selected design cases.	Identify the design parameters on the basis of typical foundation types and movements. For each considered foundation type and movement, the parameters to be calibrated must be determined; for example, immediate settlement of a spread footing based on the Hough method, lateral deflection of a driven pile group at groundline based on the *P*-*y* method.
4. Develop statistical models for load and resistance.	Gather statistical information about the performance of the considered movement types and prediction models. Resistance is often based on deterministic approach and its value will vary as a function of the considered structural limit state. Determine the Accuracy (*X*) factor and statistics for loads based on prediction models. Choose an appropriate PDF for *X*.
5. Apply the reliability analysis procedure.	Use the PEC method to calculate reliability. In some cases, depending on the type of PDF, a closed form solution may be possible.
6. Review the results and develop the *SE* load factors for target reliability indices.	Develop the *SE* load factor for all applicable structural limits states and their corresponding target reliability indices and consider reversible and irreversible limit states.
7. Select the *SE* load factor.	Select an appropriate *SE* load factor based on owner criteria; for example, reversible-irreversible condition.

9.2 Steps for Calibration

9.2.1 Step 1: Formulate the Limit State Functions and Identify Basic Variables

In the context of movements, tolerable movements (δ_T) can be considered as resistances while the predicted movements (δ_P) can be considered as loads. Thus, a limit state function (g) can be given by Equation 9-1 (first introduced as Equation 8-1):

$$g = \delta_T - \delta_P \tag{9-1}$$

For calibration of movements, the limit state *g* expressed as a ratio is more appropriate, as given by Equation 9-2:

$$g = \delta_P / \delta_T \tag{9-2}$$

9.2.2 Step 2: Identify and Select Representative Structural Types and Design Cases

To demonstrate the calibration process, immediate settlement of spread footings is used as a design case. As noted earlier, the vertical and lateral movements for all structural foundation types (for example, footings, drilled shafts, and driven piles) and retaining walls can generally be calibrated using the process described in this example.

9.2.3 Step 3: Determine Load and Resistance Parameters for the Selected Design Cases

The load and resistance parameters for the selected design case of immediate vertical settlement of spread footings are as follows: Load is predicted (or calculated) immediate vertical settlement (δ_P), and resistance is tolerable (or limiting or measured) immediate settlement (δ_T).

The *AASHTO LRFD* uses the symbol "*S*" for foundation settlement (vertical movement). Therefore, for further discussions, the symbol *S* will be used instead of δ. Similarly, while calibrating other movement modes, an appropriate symbol may be used that defines that particular movement mode; for example, the symbol "*y*" is used for lateral displacement at the top of piles using the *P-y* method of analysis. For this example problem, load is predicted (or calculated) immediate vertical settlement (S_P), and resistance is tolerable (or limiting or measured) immediate vertical settlement (S_T).

9.2.4 Step 4: Develop Statistical Models for Load and Resistance

Table 9-2 shows a dataset for spread footings based on vertical settlements of footings measured at 20 footings for 10 instrumented bridges in the northeast United States (Gifford et al., 1987). The bridges included five simple-span and five continuous-beam structures. Each of the site designations in Table 9-2 represents a footing supporting a single substructure unit (abutment or pier). Four of the instrumented bridges were single-span structures. Two two-span and three four-span bridges were also monitored in addition to a single five-span structure. Nine of the structures were designed to carry highway traffic, while one four-span bridge carried railroad traffic across an Interstate highway. Additional information on the subsurface conditions, instrumentation, and data collection at the 10 bridges can be found in Gifford et al. (1987).

There are similar and more extensive databases for spread footings (for example, Baus, 1992; Sargand et al., 1999; Sargand and Masada, 2006; Akbas and Kulhawy, 2009; and Samtani et al.,

2010) and other foundation types such as driven piles and drilled shafts. Similar databases are also available for lateral load behavior of deep foundations as well as movements of MSE walls. However, for the purpose of this report, the calibration concepts for foundation movements are demonstrated by the use of the limited dataset for spread footings shown in Table 9-2. All concepts discussed here are applicable to other foundation or wall types and movement patterns.

Gifford et al. (1987) compared the measured settlements against predicted settlements from several methods. Five of the methods used by Gifford et al. (1987) are as follows:

- Schmertmann: Method by Schmertmann et al. (1978)
- Hough: Method by Hough (1959)
- D'Appolonia: Method by D'Appolonia et al. (1968)
- Peck and Bazaraa: Method by Peck and Bazaraa (1969)
- Burland and Burbridge: Method by Burland and Burbridge (1984)

The predicted settlements for all five methods are shown in Table 9-2 along with the measured settlements. All the data points showed measured immediate settlement values smaller than 1.0 in. The minimum measured value was 0.23 in. and the maximum measured value was 0.94 in. Based on all five prediction methods, the predicted values ranged from 0.06 in. to 1.85 in. Figure 9-1 shows plots of the data in Table 9-2 and the spread of the data about the diagonal dashed 1:1 line, which defines the case for which the predicted and measured values are equal. Figure 9-1a shows the combined plot based on all datasets in Table 9-2, while Figures 9-1b to 9-1f show the plot for dataset for each method in Table 9-2. Such plots provide a visual frame of reference to judge the accuracy of a prediction method as follows:

- If the data points align closely with the 1:1 line, then the predictions based on the analytical method being evaluated are close to the measured values and are more accurate than the case where the data points do not align closely with the 1:1 line.
- Points corresponding to predicted values larger than measured values plot below the 1:1 line and represent the case where the predicted values are conservative. The farther below the 1:1 line the points are, the more conservative the predicted values.
- Points corresponding to predicted values smaller than the measured values plot above the 1:1 line and represent the case where the predicted values are unconservative. The farther above the 1:1 line the points are, the more unconservative the predicted values are and represent a progressively more undesirable condition.

Table 9-2. Data for Measured Settlement, S_M, and Predicted (Calculated) Settlement, S_P, Shown on Figure 9-1 Based on Gifford et al. (1987) (All Settlement Values are in Inches)

Site	Measured	Predicted Schmertmann	Predicted Hough	Predicted D'Appolonia	Predicted Peck and Bazaraa	Predicted Burland & Burbridge
#1	0.35	0.79	0.75	0.65	0.29	0.30
#2	0.67	1.85	0.94	0.39	0.16	0.12
#3	0.94	0.86	1.21	0.30	0.19	0.13
#4	0.76	0.46	1.46	0.58	0.36	0.39
#5	0.61	0.30	0.98	0.38	0.42	0.57
#6	0.42	0.52	0.61	0.50	0.17	0.34
#7	0.61	0.18	0.40	0.19	0.30	0.19
#8	0.28	0.30	0.60	0.26	0.16	0.14
#9	0.26	0.18	0.53	0.20	0.16	0.11
#10	0.29	0.29	0.40	0.23	0.16	0.09
#11	0.25	0.36	0.47	0.29	0.16	0.06
#14	0.46	0.41	1.27	0.57	0.50	0.40
#15	0.34	1.57	1.46	0.74	1.36	1.61
#16	0.23	0.26	0.74	0.39	0.17	0.17
#17	0.44	0.40	0.82	0.46	0.28	0.23
#20	0.64	1.21	1.05	0.49	0.21	0.54
#21	0.46	0.29	0.84	0.56	0.52	0.31
#22	0.66	0.54	1.39	0.61	0.34	0.64
#23	0.61	1.02	0.99	0.59	0.33	0.44
#24	0.28	0.64	0.61	0.36	0.25	0.36
Minimum	0.23	0.18	0.40	0.19	0.16	0.06
Maximum	0.94	1.85	1.46	0.74	1.36	1.61

Note: Gifford et al. (1987) note that data for footings at Sites #12, #13, and #18 were not included because construction problems at these sites resulted in disturbance of the subgrade soils, and short-term settlement was increased. Data for footing at Site #19 appears to be anomalous and have been excluded in this table and on Figure 9-1.

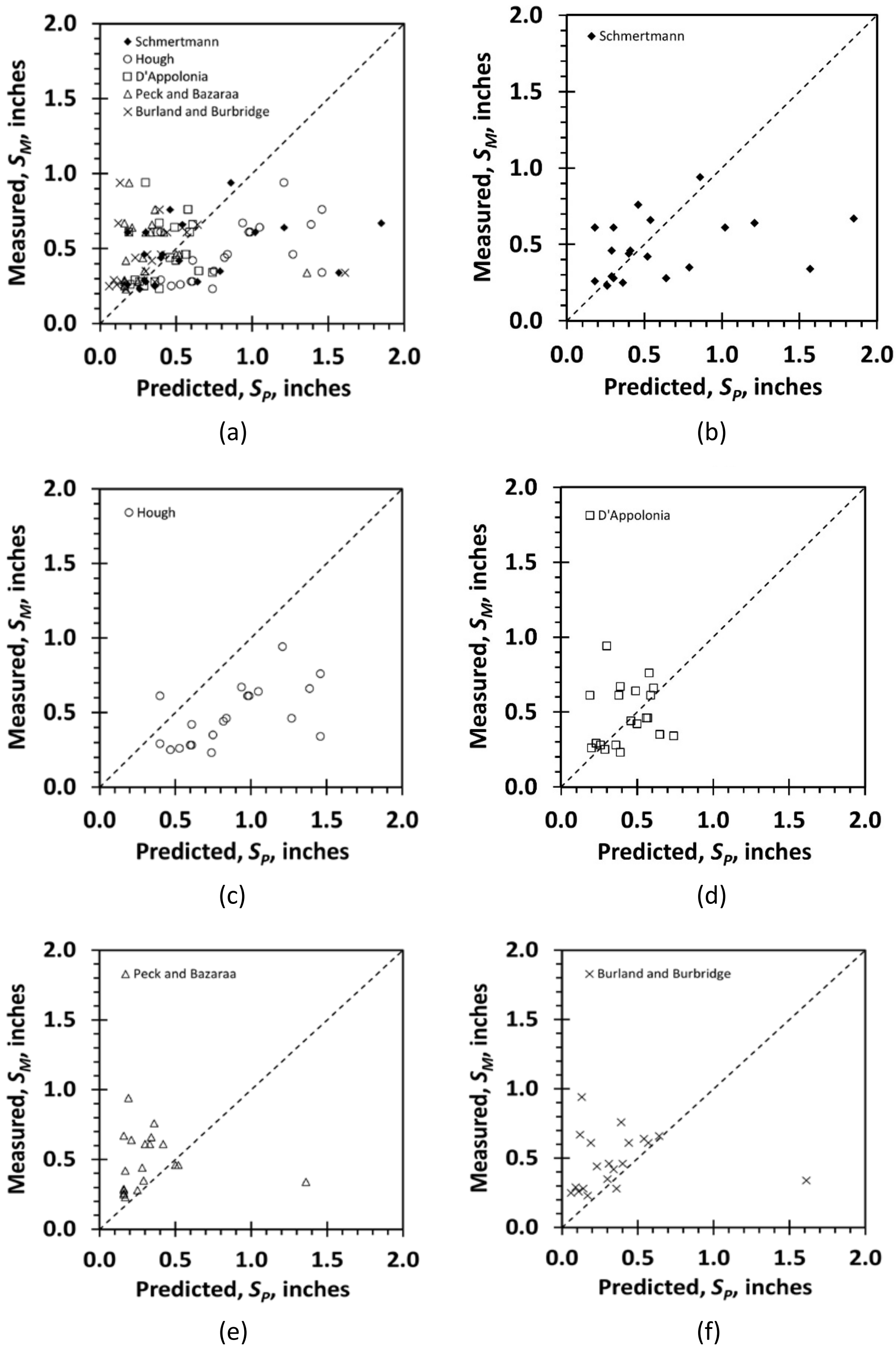


Figure 9-1. Comparison of Measured and Predicted Settlements Based on Service Load Data in Table 9-2. (a) All Methods, (b) Schmertmann Method, (c) Hough Method, (d) D'Appolonia Method, (e) Peck and Bazaraa Method, and (f) Burland and Burbridge Method

Based on the plots on Figure 9-1, following are general observations for the Gifford et al. (1987) database:

- For the Schmertmann method (Figure 9-1b) and D'Appolonia method (Figure 9-1d), the data points are distributed on both sides of the 1:1 line and therefore predictions from these methods may be conservative or unconservative. Between these two methods, the D'Appolonia method appears to have more data points clustered around the 1:1 line compared to the Schmertmann method.
- For the Hough method (Figure 9-1c), all data points, except for one data point, are below the 1:1 line thereby indicating that it is a conservative method.
- For the Peck and Bazaraa method (Figure 9-1e) and Burland and Burbridge method (Figure 9-1f), most of the data points are above the 1:1 line thereby indicating these methods are unconservative.

In the geotechnical literature (for example, Tan and Duncan, 1991), "Accuracy," (or settlement ratio, *SR*) is defined as the ratio of the predicted to the measured settlements. Table 9-3 shows the values of Accuracy (denoted by X, where $X = S_P/S_M$) for each footing based on the data in Table 9-2 for all five methods. Table 9-4 presents the arithmetic mean (μ_X) and standard deviation (σ_X) values for Accuracy, X, of various methods. The mean, standard deviation, and COV values for the five methods are consistent with the observations of the corresponding plots on Figure 9-1b to 9-1e. For example, the mean value of X for the Hough method is much larger than those for the other methods which is consistent with the earlier observation that all data points, except for one data point, plot below the 1:1 line on Figure 9-1c. The Hough method also has the smallest COV, which means that the data points are more tightly clustered about their mean value compared to other methods as can be observed from visually comparing the spread of data points for different methods on Figure 9-1b to Figure 9-1f.

The *AASHTO LRFD* recommends the use of the Hough method, which has the smallest COV for calculating immediate settlement. However, the Hough method is conservative by a factor of approximately 2 (see mean value in Table 9-4), which leads to the unnecessary use of deep foundations instead of spread footings. FHWA (Samtani and Nowatzki, 2006; Samtani et al. 2010) recommends the Schmertmann method because it considers not only the applied stress and its associated strain influence distribution with depth for various footing shapes, but also the elastic properties of the foundation soils, even if they are layered. Even though FHWA and the *AASHTO LRFD* recommend the Schmertmann and Hough methods, respectively, all the methods noted in Table 9-2 to Table 9-4 were evaluated as part of the calibration process because some agencies may use one of the remaining three methods as a result of past successful local practice.

Table 9-3. Accuracy ($X=S_P/S_M$) Values Based on Data Shown in Table 9-2

Site	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
#1	2.2571	2.1429	1.8571	0.8286	0.8571
#2	2.7612	1.4030	0.5821	0.2388	0.1791
#3	0.9149	1.2872	0.3191	0.2021	0.1383
#4	0.6053	1.9211	0.7632	0.4737	0.5132
#5	0.4918	1.6066	0.6230	0.6885	0.9344
#6	1.2381	1.4524	1.1905	0.4048	0.8095
#7	0.2951	0.6557	0.3115	0.4918	0.3115
#8	1.0714	2.1429	0.9286	0.5714	0.5000
#9	0.6923	2.0385	0.7692	0.6154	0.4231
#10	1.0000	1.3793	0.7931	0.5517	0.3103
#11	1.4400	1.8800	1.1600	0.6400	0.2400
#14	0.8913	2.7609	1.2391	1.0870	0.8696
#15	4.6176	4.2941	2.1765	4.0000	4.7353
#16	1.1304	3.2174	1.6957	0.7391	0.7391
#17	0.9091	1.8636	1.0455	0.6364	0.5227
#20	1.8906	1.6406	0.7656	0.3281	0.8438
#21	0.6304	1.8261	1.2174	1.1304	0.6739
#22	0.8182	2.1061	0.9242	0.5152	0.9697
#23	1.6721	1.6230	0.9672	0.5410	0.7213
#24	2.2857	2.1786	1.2857	0.8929	1.2857

Table 9-4. Statistics of Accuracy, *X*, Values Based on Data Shown in Table 9-3

Statistic	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
Count	20	20	20	20	20
Minimum	0.2951	0.6557	0.3115	0.2021	0.1383
Maximum	4.6176	4.2941	2.1765	4.0000	4.7353
μ_X	1.3806	1.9710	1.0307	0.7788	0.8289
σ_X	1.0064	0.7693	0.4761	0.7964	0.9678
COV_X	0.7290	0.3903	0.4619	1.0225	1.1676

Note: μ_X = Mean; σ_X = Standard Deviation; COV_X = Coefficient of Variation (=σ_X/μ_X)

9.2.4.1 Selection of an Appropriate Probability Distribution Function

The Accuracy, *X*, is a random variable that can be modeled by an appropriate PDF. To develop an appropriate PDF, an evaluation of the data spread around the mean value is needed. As a first step in selecting an appropriate PDF, the validity of a normal distribution should be evaluated. The histogram and associated PDF corresponding to a normal distribution resembles a classical "bell-shape" that is symmetric about the mean value. A simplistic evaluation of the PDF shape can be performed by evaluating the shape of the corresponding histogram. The histograms of the data for *X* taken from Columns 2 to 6 of Table 9-3 are shown on Figure 9-2. The numbers on top of each bar in a bin represents the number of data points in that bin interval.

None of the histograms on Figure 9-2 resemble a classical bell shape characteristic of normally distributed data. Further, it appears that the data distribution is positively skewed; that is, the right tail of the distribution is longer than the left tail. Comparatively, it appears that the D'Appolonia, Peck and Bazaraa, and Burland and Burbridge methods are more positively skewed than the Schmertmann and Hough methods. The Hough method, followed by the Schmertmann method, appears to be the closer to normal distribution compared to the other three methods. However, any inferences of the distributions being normally, or close to being normally, distributed must be tempered by the fact that these comparisons are based on a limited dataset of 20 points for each method due to which generalization based on a specific method is not justified. Typically, based on experience, foundation movements are nonnormally distributed. Based on the above considerations, a nonnormal distribution should be used for the Accuracy data related to any prediction method for foundation movements.

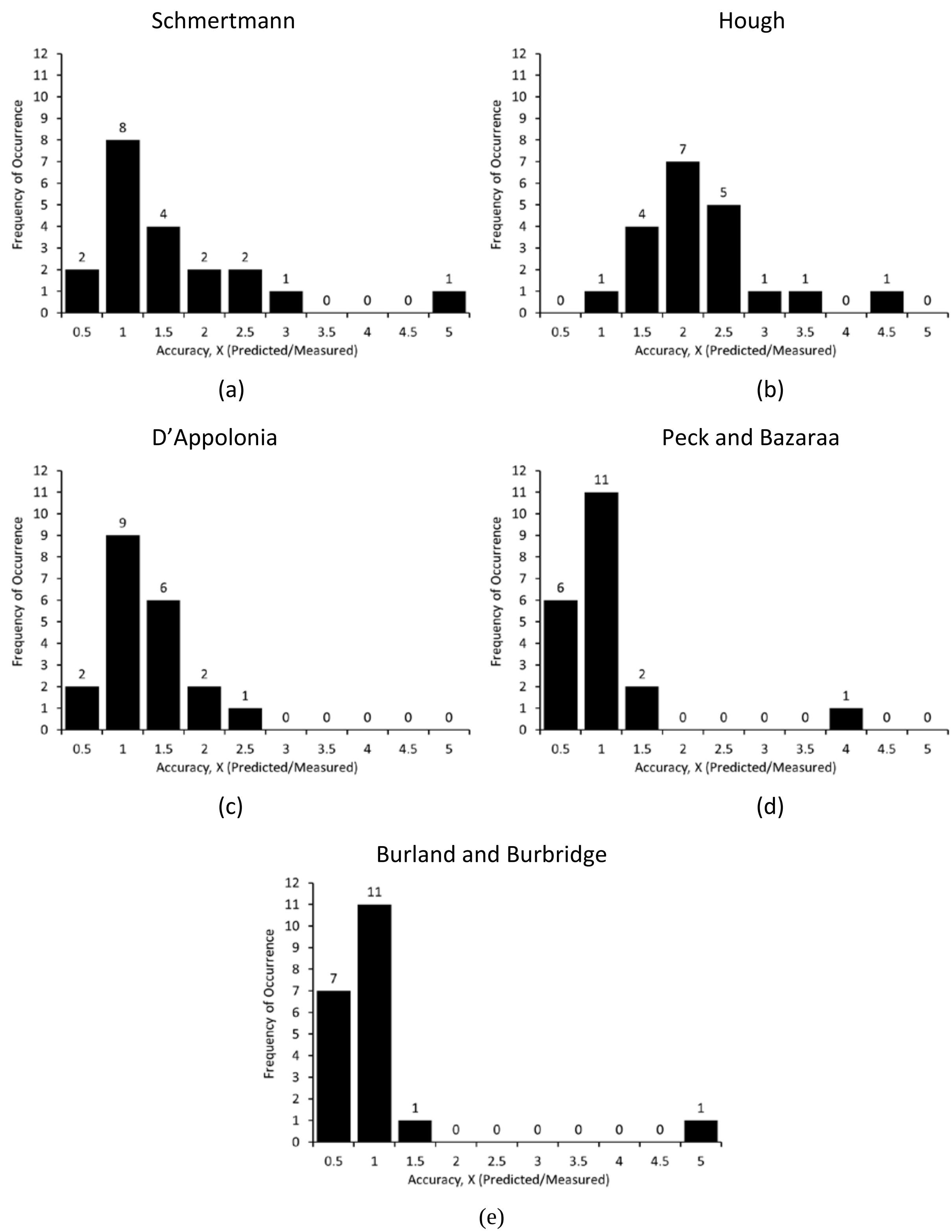


Figure 9-2. Histograms for (a) Schmertmann Method, (b) Hough Method, (c) D'Appolonia Method, (d) Peck and Bazaraa Method, and (e) Burland and Burbridge Method

The deviation of the data from a classical bell-shaped normal PDF can be evaluated by plotting the data on a normal distribution probability paper. If the data plot on a straight line on the normal probability paper, then it is reasonable to assume that the data are normally distributed. Conversely, data that are not normally distributed will deviate from a straight line on the normal probability paper. Such an evaluation can be performed using Microsoft Excel by plotting the data against the standard normal variable (z) to generate the CDFs, as shown on Figure 9-3. See Allen et al. (2005, Chapter 5) for a definition of z and procedures to develop graphs on Figure 9-3. As the graphs on Figure 9-3 show, the data points based on Table 9-3 do not plot on the straight line, which confirms the observation of nonnormal distributions made based on the histograms on Figure 9-2. The next step is to select an appropriate nonnormal distribution.

For foundation movements, a PDF with an upper bound and lower bound (beta distribution) instead of open tail(s) may be more appropriate because the conditions represented by an open-tail PDF are not physically possible when one considers foundation movements. However, even though a beta distribution may be more appropriate, a lognormal distribution is first evaluated because it has been used in the past to approximate nonnormal distributions during calibration of the strength limit state for geotechnical, as well as structural, features in the *AASHTO LRFD* framework. The lognormal distribution is valid between values of 0 and +∞ (that is, an open right tail). If a lognormal distribution is found to approximate the data reasonably well, then it may be used for further modeling; otherwise, another distribution can be considered. The random variable, X, can be considered lognormally distributed if ln(X) (that is, natural logarithm of X) is normally distributed. However, as discussed in Appendix C, unlike a normally distributed random variable that can be described using only COV (that is, one parameter), for a lognormally distributed random variable, lognormal mean and lognormal standard deviation (that is, two parameters) are needed to describe it. These values for lognormal distribution can be obtained from idealized lognormal distributions by using correlations with the mean and standard deviation values for normal distribution or calculated directly from the natural logarithm (ln) of the values of the data points. These approaches are as follows:

1. Use the correlated mean, $\mu_{LNC\text{-}X}$, and standard deviation, $\sigma_{LNC\text{-}X}$, values for idealized lognormal distribution that are calculated from the normal (arithmetic) mean, μ_X, and standard deviation, σ_X, values of the sample population, respectively, using the following correlations (Benjamin and Cornell, 1970):

   $\mu_{LNC\text{-}X} = \ln(\mu_X) - 0.50(\sigma_{LNC\text{-}X})^2$; $\sigma_{LNC\text{-}X} = [\ln\{(\sigma_X/\mu_X)^2 + 1\}]^{0.5}$

   Table 9-5 presents the values for correlated mean, $\mu_{LNC\text{-}X}$, and correlated standard deviation. $\sigma_{LNC\text{-}X}$, based on the above correlations and the sample data in Table 9-4.

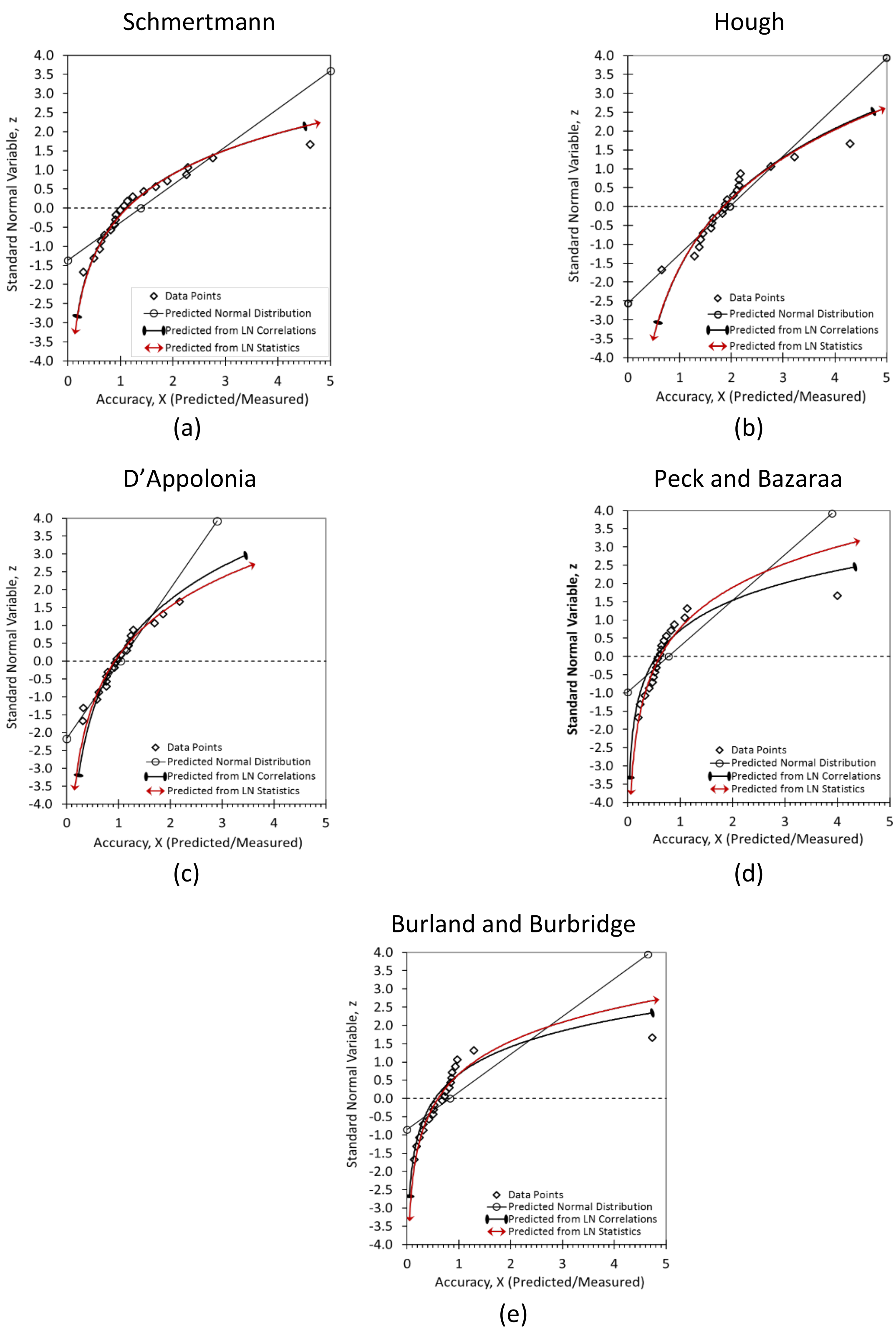


Figure 9-3. Plot of Standard Normal Variable (*z*) as a Function of *X* for (a) Schmertmann Method, (b) Hough Method, (c) D'Appolonia Method, (d) Peck and Bazaraa Method, and (e) Burland and Burbridge Method

Table 9-5. Correlated Statistics for Accuracy (*X*) for Lognormal Probability Distribution Functions

Statistic	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
$\mu_{LNC\text{-}X}$	0.1095	0.6076	-0.0665	-0.6078	-0.6177
$\sigma_{LNC\text{-}X}$	0.6528	0.3766	0.4398	0.8459	0.9274

Note: $\mu_{LNC\text{-}X}$ = correlated mean of ln(*X*) values; $\sigma_{LNC\text{-}X}$ = correlated standard deviation of ln(*X*) values

2. Use the arithmetic (normal) statistics for values of natural logarithm of *X*; that is, ln (*X*). Table 9-6 shows the natural logarithm of Accuracy values of data in Table 9-3, and Table 9-7 presents the values for arithmetic mean, $\mu_{LNA\text{-}X}$, and arithmetic standard deviation, $\sigma_{LNA\text{-}X}$, based on the ln(*X*) values in Table 9-6.

The correlated and the arithmetic values of the mean ($\mu_{LNC\text{-}X}$ and $\mu_{LNA\text{-}X}$, respectively) and standard deviation ($\sigma_{LNC\text{-}X}$ and $\sigma_{LNA\text{-}X}$, respectively) for lognormal distributions are not equal as can be observed from the values in Table 9-5 and Table 9-7. This is because the correlated values are based on derivations for an idealized lognormal distribution and not a sample distribution from actual data, which may not necessarily fit an idealized lognormal distribution. In contrast, the arithmetic values are obtained by taking the arithmetic mean and standard deviation directly from the ln(*X*) value of each data point noted in Columns 2 to 6 in Table 9-3.

Using the statistics in Table 9-5 and Table 9-7, two lognormal distribution fits are shown on Figure 9-3 for each prediction method. The first lognormal distribution fit, labeled as "Predicted from LN Correlations," is based on correlated statistics for Accuracy (*X*) for lognormal PDFs using the values in Table 9-5. The second lognormal distribution fit labeled as "Predicted from LN Statistics," is based on statistics for ln(*X*) using the values in Table 9-7. For the Schmertmann and Hough methods, the statistics in Table 9-5 and Table 9-7 are approximately the same; therefore, for these methods, the two lognormal distribution fits are virtually the same. However, for the D'Appolonia, Peck and Bazaraa, and Burland and Burbridge methods, the lognormal distribution fits are different because the statistics for these methods differ in Table 9-5 and Table 9-7. This difference between the statistics for lognormal distribution is to be expected because the distributions for the last three methods are more positively skewed compared to those for the Schmertmann and Hough methods.

It is important to use the appropriate values of mean and standard deviation based on the syntax for a lognormal distribution function used by a particular computational program. For example, if one is using the @RISK program by Palisade Corporation, then the RISKLOGNORM function in that program is based on arithmetic values (μ_X and σ_X) of the normal distribution. In contrast, the Microsoft Excel LOGNORMDIST (or LOGNORM.DIST) function uses the arithmetic mean ($\mu_{LNA\text{-}X}$) and standard deviation ($\sigma_{LNA\text{-}X}$) values of ln(*X*). Use of improper values of mean and standard deviation can lead to drastically different results. This issue is of critical importance because calibration in this report, as mentioned earlier, is based on Microsoft Excel. Thus, while use of correlations to develop lognormal distribution fits may appear expedient, for modeling foundation movements, it is recommended that when using Microsoft Excel the statistical parameters for lognormal PDF be obtained using the ln(*X*) approach rather than the correlations.

Table 9-6. Lognormal of Accuracy Values [ln(*X*)] Based on Data Shown in Table 9-3

Site	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
#1	0.8141	0.7621	0.6190	-0.1881	-0.1542
#2	1.0157	0.3386	-0.5411	-1.4321	-1.7198
#3	-0.0889	0.2525	-1.1421	-1.5989	-1.9783
#4	-0.5021	0.6529	-0.2703	-0.7472	-0.6672
#5	-0.7097	0.4741	-0.4733	-0.3732	-0.0678
#6	0.2136	0.3732	0.1744	-0.9045	-0.2113
#7	-1.2205	-0.4220	-1.1664	-0.7097	-1.1664
#8	0.0690	0.7621	-0.0741	-0.5596	-0.6931
#9	-0.3677	0.7122	-0.2624	-0.4855	-0.8602
#10	0.0000	0.3216	-0.2318	-0.5947	-1.1701
#11	0.3646	0.6313	0.1484	-0.4463	-1.4271
#14	-0.1151	1.0155	0.2144	0.0834	-0.1398
#15	1.5299	1.4572	0.7777	1.3863	1.5550
#16	0.1226	1.1686	0.5281	-0.3023	-0.3023
#17	-0.0953	0.6225	0.0445	-0.4520	-0.6487
#20	0.6369	0.4951	-0.2671	-1.1144	-0.1699
#21	-0.4613	0.6022	0.1967	0.1226	-0.3947
#22	-0.2007	0.7448	-0.0788	-0.6633	-0.0308
#23	0.5141	0.4842	-0.0333	-0.6144	-0.3267
#24	0.8267	0.7787	0.2513	-0.1133	0.2513

Table 9-7. Statistics for ln(*X*) Values Based on Data Shown in Table 9-6

Statistic	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
Count	20	20	20	20	20
Minimum	-1.2205	-0.4220	-1.1664	-1.5989	-1.9783
Maximum	1.5299	1.4572	0.7777	1.3863	1.5550
μ_{LNA-X}	0.1173	0.6114	-0.0793	-0.4854	-0.5161
σ_{LNA-X}	0.6479	0.3807	0.5029	0.6226	0.7731

Note: μ_{LNA-X} = arithmetic mean of ln(*X*) values; σ_{LNA-X} = arithmetic standard deviation of ln(*X*) values

Using the statistics (μ_{LNA-X} and σ_{LNA-X}) in Table 9-7 based on the ln(*X*) approach, Figure 9-4 shows the lognormal PDFs for each of the five settlement prediction methods using the LOGNORM.DIST function in 2010 and later versions of Microsoft Excel. The PDFs on Figure 9-4 resemble the respective histograms on Figure 9-2 quite well.

Figure 9-5 shows all the PDFs from Figure 9-4 together, and in this representation the relative positive skewness of the various distributions becomes clearer. However, to evaluate the implications on design, the PDFs need to be converted to CDFs. Corresponding to each of the PDFs on Figure 9-5, Figure 9-6 shows the CDFs for Accuracy, *X*, based on the use of the LOGNORM.DIST function in 2010 and later versions of Microsoft Excel using the μ_{LNA-X} and σ_{LNA-X} values noted in Table 9-7. These CDFs can now be used to develop the PEC discussed inSection 8.3.4 for various analytical methods.

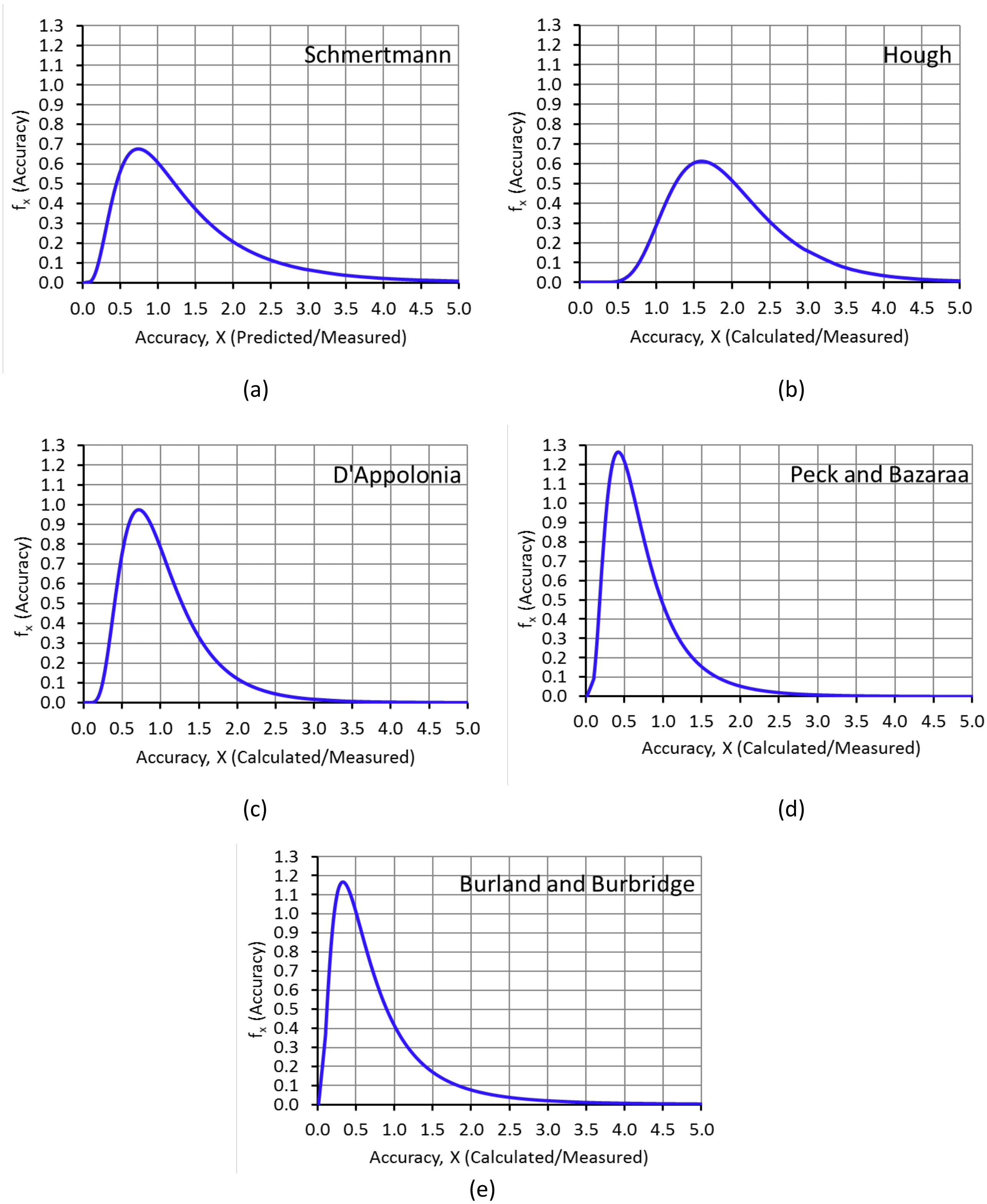


Figure 9-4. Lognormal Probability Distribution Functions for (a) Schmertmann Method, (b) Hough Method, (c) D'Appolonia Method, (d) Peck and Bazaraa Method, and (e) Burland and Burbridge Method

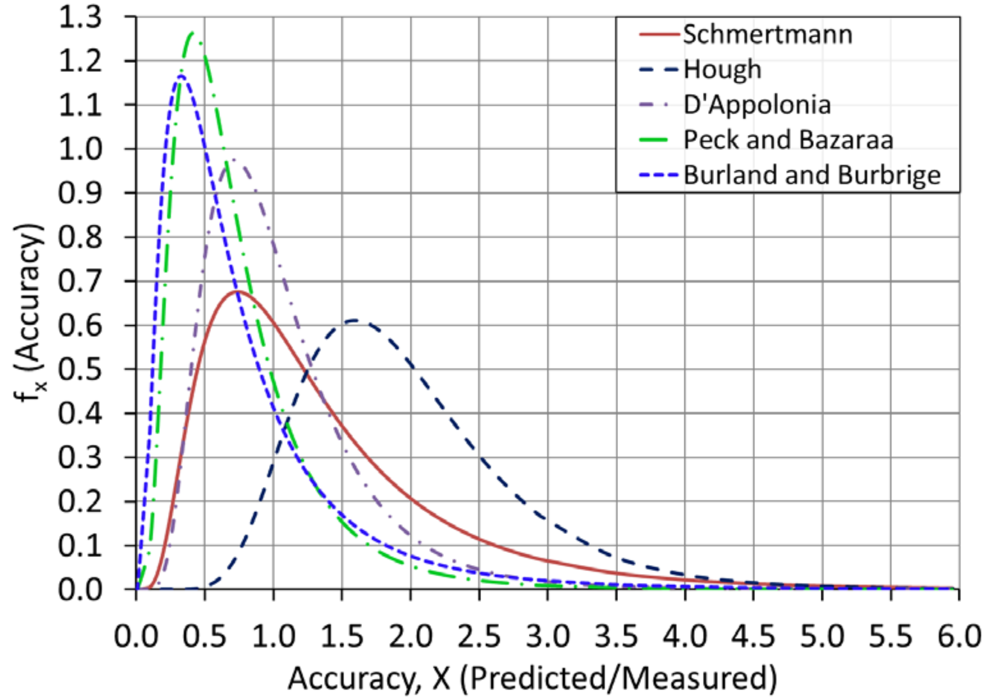


Figure 9-5. Probability Distribution Functions for Various Analytical Methods for Estimation of Immediate Settlement of Spread Footings

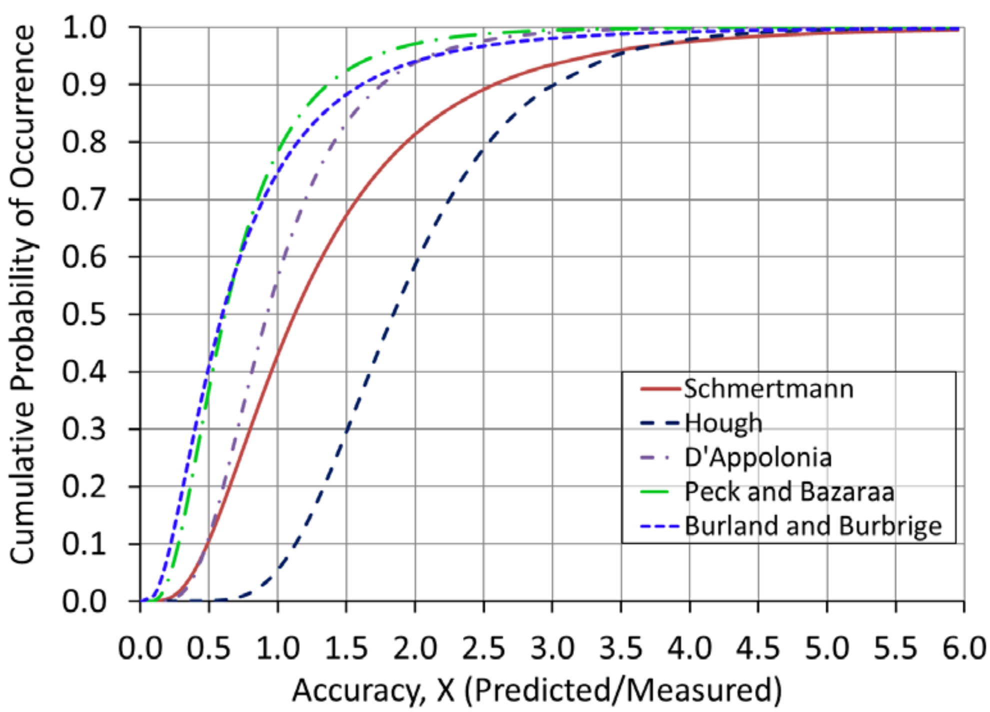


Figure 9-6. Cumulative Distribution Functions for Various Analytical Methods for Estimation of Immediate Settlement of Spread Footings

Figure 9-7 shows the PEC for method by Schmertmann et al. (1978). This figure was developed by scaling (multiplying) the Accuracy values for Schmertmann on Figure 9-6, by dimensional values of S_T; that is, S_T = 1 in., 2 in., and so on. For example, Figure 9-6 indicates a cumulative probability of about 0.8 for an Accuracy of 2.0. On Figure 9-7, if the values of Accuracy (S_P/S_T) are multiplied by S_T = 2.0 in., the result is a value of S_P = 4.0 in. at a cumulative probability of 0.8,which is now shown as a percentage called probability of exceedance of about 80 percent.

Using the above procedure, the probability of exceedance corresponding to a given predicted settlement can now be readily determined. For example, assume that the geotechnical engineer has predicted a settlement of 0.85 in. This value is shown by Point A in Figure 9-7. The probability of exceedance of 1 in. in this case is approximately 32 percent. This can be found by drawing line AB, finding the intersection of the line with the curve for 1 in., drawing line BC, and reading the value from the ordinate of the PEC on Figure 9-7. Four additional curves for settlements of 1.5, 2, 2.5, and 3 in. are shown on Figure 9-7. Using the procedure demonstrated for the example above (see dashed arrows on Figure 9-7), if the predicted (calculated) value is 0.85 in., then the probability of the measured value being greater than 1.5, 2, 2.5, and 3 in. is approximately 14 percent, 6 percent, 3 percent, and 2 percent, respectively.

A load factor for settlement, γ_{SE}, can be determined using the procedure in Section 8.3.4. For example, assume the predicted settlement is 1 in. This value is shown by Point E in Figure 9-7. To determine the value of γ_{SE} for a 25 percent target probability of exceedance (P_{eT}), draw a horizontal line from Point D on the ordinate corresponding to a value of 25 percent. Next, draw a vertical line from Point E on the abscissa corresponding to a value of 1 in. Locate the point of intersection, F, which lies between the curves for 1 in. and 1.5 in. Interpolating between the two curves leads to a value of approximately 1.35 in. Based on the definition of γ_{SE} noted above, the value of γ_{SE} is equal to 1.35 in./1.0 in., or 1.35.

PECs for other analytical methods noted on Figure 9-6 are given on Figure 9-8. Those PECs can be used in a similar manner as demonstrated for the PEC for the Schmertmann method.

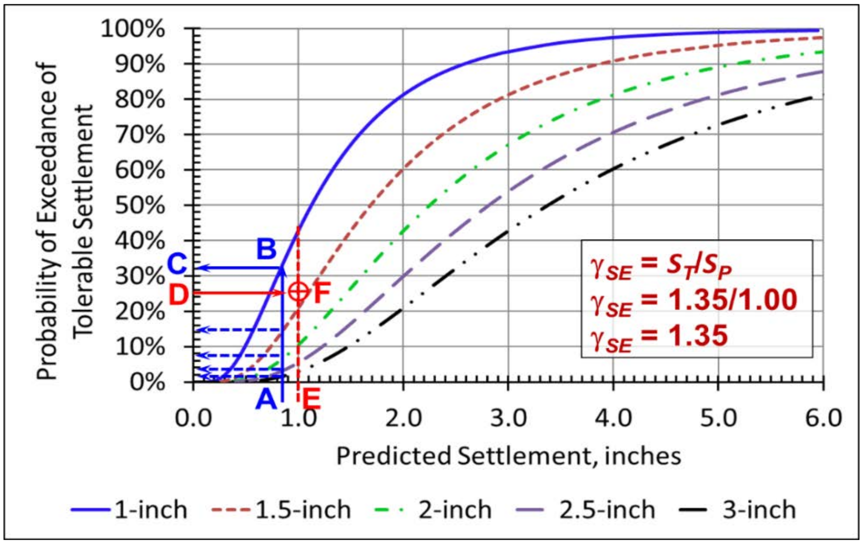


Figure 9-7. Probability Exceedance Chart for Schmertmann Method

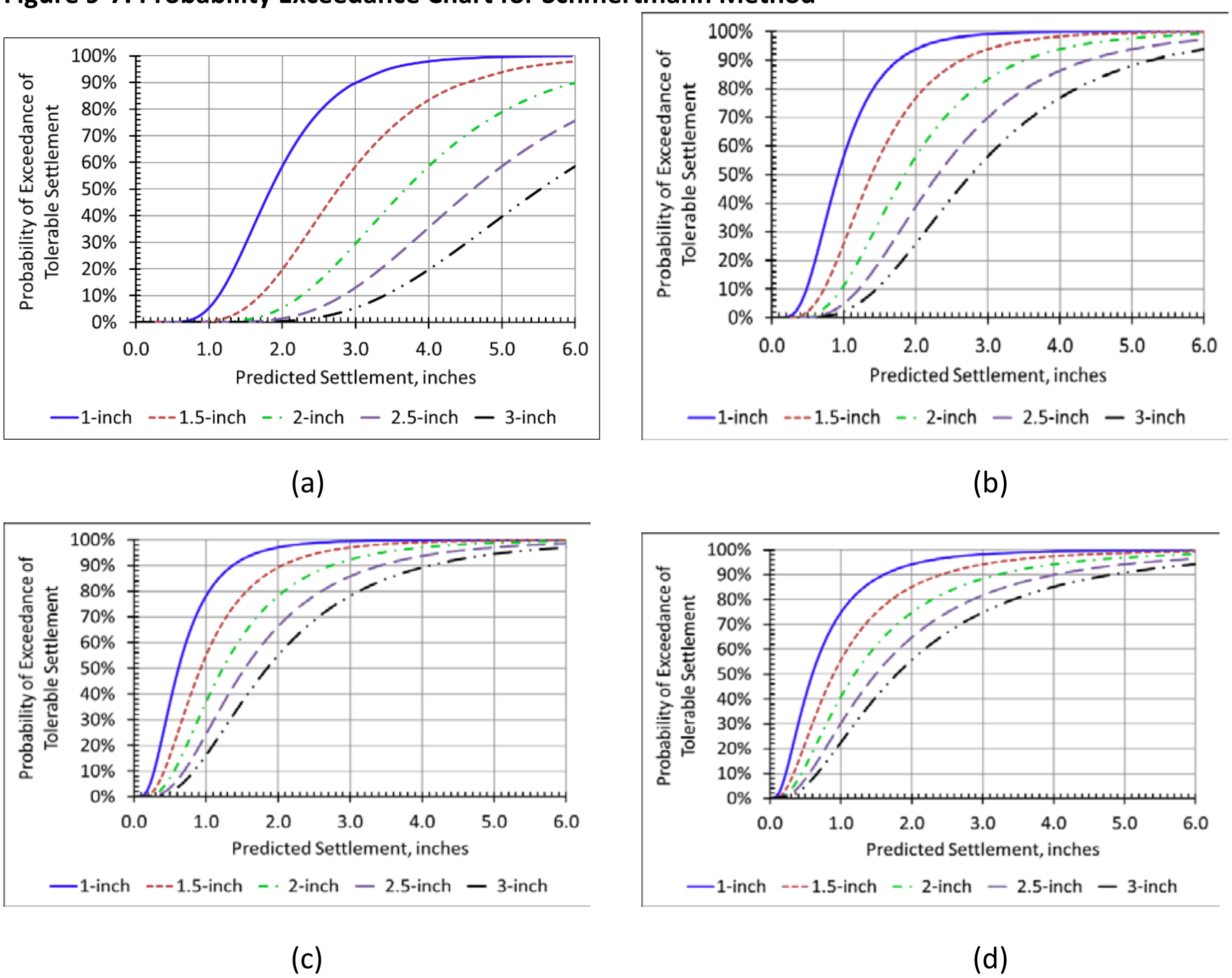


Figure 9-8. Probability Exceedance Charts for (a) Hough Method, (b) D'Appolonia Method, (c) Peck and Bazaraa Method, and (d) Burland and Burbridge Method

9.2.5 Step 5: Apply the Reliability Analysis Procedure

The estimation of load factor for settlement, γ_{SE}, in terms of probability of exceedance was demonstrated in Step 4. In the *AASHTO LRFD* framework, calibrations are expressed in terms of reliability index (β). β can be expressed in terms of P_e of a predicted value by using Equation 9-3 based on the NORMSINV function in Microsoft Excel, which applies to normally distributed data.

$$\beta = \text{NORMSINV}(1\text{-}P_e) \tag{9-3}$$

However, as observed from Step 4, lognormal distributions are needed to adequately represent the Accuracy data. As discussed in Appendix C, for a normal random variable, the relationship between β and P_e depends only on the COV (that is, one parameter), but for a lognormal distribution, it depends on the mean and standard deviation, or the mean and COV (that is, two parameters). Therefore, if lognormal distributions are used to model data, then the reliability index should be theoretically based on lognormal function. Appendix C includes a discussion on the reliability index based on normal and lognormal PDFs. Based on the discussions in Appendix C, the following features are noted:

- As a practical matter, for $\beta < 2.0$, there is not a significant difference in the P_e values for data that are normally or lognormally distributed for a wide range of COVs noted in Table 9-4.
- An assumption of a normal distribution is generally conservative in the sense that for a given β, it gives a larger P_e compared to a lognormal distribution.

The normal distribution has been conventionally assumed for strength limit states in the *AASHTO LRFD* (as well as other international codes), which have reliability index values larger than 1.0. The key consideration is that the type of distribution is not as important as being consistent and not mixing different distributions while comparing β values. Based on these considerations and above-noted features, the use of the Microsoft Excel formula in Equation 9-3 assumes normally distributed data are considered to be acceptable for the service limit state calibrations of movement.

Table 9-8 and Figure 9-9 were generated by using Equation 9-3. The correlation between β and P_e, can now be used to rephrase the discussion earlier with respect to Figure 9-7. In that discussion, as an example, it was assumed that the geotechnical engineer has predicted a settlement of 0.85 in. From Figure 9-7 it was determined that the probability of exceedance of 1, 1.5, 2, 2.5, and 3 in. was approximately 32 percent, 14 percent, 6 percent, 3 percent, and 2 percent, respectively. Using Table 9-8 (or Figure 9-9 or Equation 9-3), the results can now be expressed in terms of reliability index values. If the predicted settlement is 0.85 in., then the assumption of tolerable settlement values of 1, 1.5, 2, 2.5, and 3 in. means a reliability index of approximately 0.45, 1.10, 1.55, 1.90, and > 2.00, respectively.

Table 9-8. Values of β and Corresponding P_e Based on Normally Distributed Data

β	P_e, %	β	P_e, %	β	P_e, %	β	P_e, %
2.00	2.28	1.50	6.68	1.00	15.87	0.50	30.85
1.95	2.56	1.45	7.35	0.95	17.11	0.45	32.64
1.90	2.87	1.40	8.08	0.90	18.41	0.40	34.46
1.85	3.22	1.35	8.85	0.85	19.77	0.35	36.32
1.80	3.59	1.30	9.68	0.80	21.19	0.30	38.21
1.75	4.01	1.25	10.56	0.75	22.66	0.25	40.13
1.70	4.46	1.20	11.51	0.70	24.20	0.20	42.07
1.65	4.95	1.15	12.51	0.65	25.78	0.15	44.04
1.60	5.48	1.10	13.57	0.60	27.43	0.10	46.02
1.55	6.06	1.05	14.69	0.55	29.12	0.05	48.01
						0.00	50.00

Note: Linear interpolation may be used as an approximation for intermediate values.

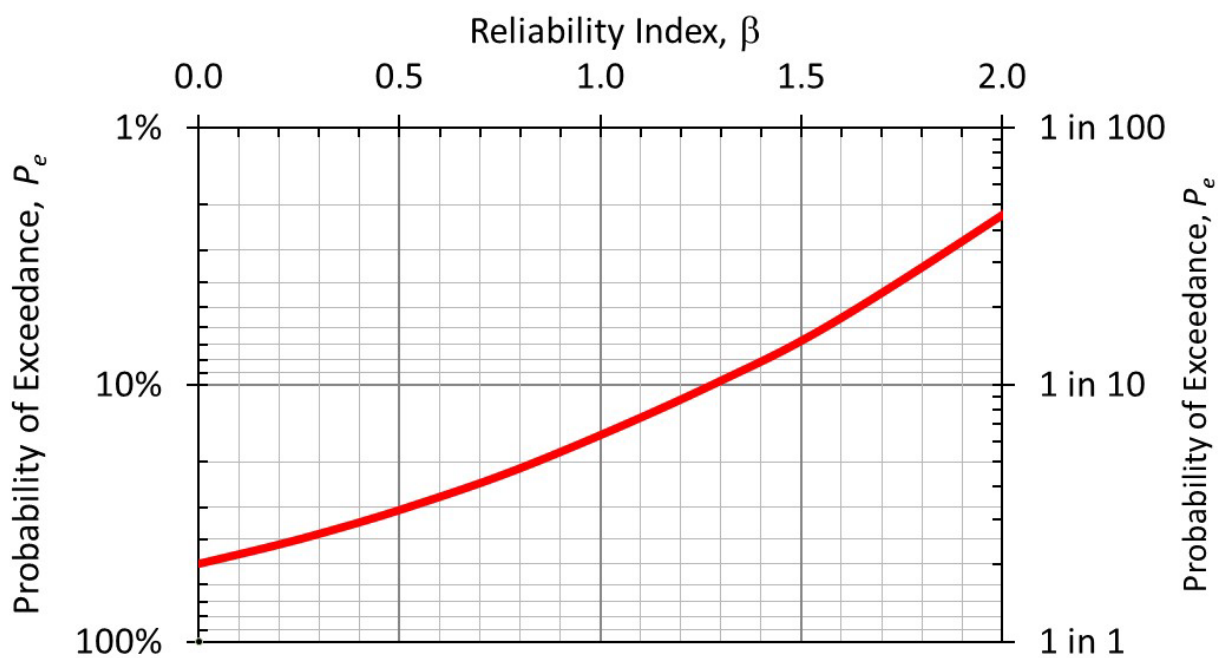


Figure 9-9. Relationship between β and P_e for the Case of a Single Load and Single Resistance

The following example demonstrates the determination of γ_{SE} in terms of β by using Microsoft Excel.

Example: The geotechnical engineer has predicted settlement S_P = 0.85 in. using the Schmertmann method. The owner has specified that the service limit state design for the bridge shall be performed using a reliability index (β) of 0.50. What is the value of γ_{SE} and the tolerable settlement that the bridge designer should use?

Solution: The load factor, γ_{SE}, is a function of the probability of exceedance, P_e, of the foundation movement under consideration, which in this example is the immediate settlement of spread footings calculated by using the analytical method of Schmertmann. Based on either Equation 9-3 or Table 9-8, a value of $P_e \approx 0.3085$ (or 30.85 percent) is obtained for β = 0.50.

Equation 9-4 is the formula used in Microsoft Excel to determine a value of Accuracy (X) in terms of P_e, the mean value ($\mu_{LNA\text{-}X}$), and the standard deviation ($\sigma_{LNA\text{-}X}$) of the lognormal distribution function as computed in Step 4. The value of X represents the probability of the Accuracy value (S_P/S_T) being less than a specified value.

$$P_e = \mathrm{LOGNORMDIST}(X, \mu_{LNA\text{-}X}, \sigma_{LNA\text{-}X}) \tag{9-4}$$

From Table 9-7, for the Schmertmann method, $\mu_{LNA\text{-}X}$ = 0.1173, $\sigma_{LNA\text{-}X}$ = 0.6479. The goal is to determine the value of X that gives P_e = 0.3085. For this example, the expression for P_e can be written as follows:

$$P_e = \mathrm{LOGNORMDIST}(X, 0.1173, 0.6479) = 0.3085 \text{ or } 30.85\% \tag{9-5}$$

Using Goal Seek in Microsoft Excel, X (that is, S_P/S_T) ≈ 0.813. Note that in the 2010 and later versions of Microsoft Excel, another function LOGNORM.DIST is also available that can be used. In this case, the same result ($X \approx 0.813$) is obtained by using the following syntax and using the Goal Seek function to determine X ("TRUE" indicates the use of a CDF):

P_e = LOGNORM.DIST(X,0.1173,0.6479,TRUE) = 0.3085

In the context of the *AASHTO LRFD* framework, the load factor, γ_{SE}, is the reciprocal of X. For immediate settlement of spread footings based on the method of Schmertmann, $\gamma_{SE} = 1/0.813 \approx 1.23$.

As per the *AASHTO LRFD* framework, the load factor is rounded up to the nearest 0.05; therefore, γ_{SE} = 1.25 should be used.

In the bridge design example, the bridge designer should use a settlement value of $(\gamma_{SE})(S_P)$ = (1.25)(0.85 in.) = 1.06 in. to assess the effect of settlement on the bridge structure. This value can also be obtained using the graphical solution explained earlier with respect to Figure 9-7. The example that was demonstrated with respect to Figure 9-7, also assumed a tolerable settlement of 0.85 in., where it was found that a settlement of 1 in. would imply a 32 percent probability of exceedance. These values are close to the value of 1.06 in. for a 30.85 percent probability of exceedance obtained here. Given that the load factor is rounded to the nearest 0.05, the result from the graphical solution is sufficiently accurate.

Table 9-9 presents the values of γ_{SE} results for various analytical methods shown on Figure 9-1 and Table 9-2. The values of γ_{SE} should be rounded to the nearest 0.05 because not doing so implies a level of confidence that is not justified by the available data. Further, the values of load factors are typically bounded by a value of 1.0. Table 9-10 presents values of γ_{SE} that are bounded by 1.0 and rounded to the nearest 0.05.

Table 9-9. Computed Values of γ_{SE} for Various Methods to Estimate Immediate Settlement of Spread Footings on Cohesionless Soils Based on Arithmetic ln(*X*) Statistics ($\mu_{LNA\text{-}X}$ and $\sigma_{LNA\text{-}X}$) in Table 9-7

Reliability Index, β	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
0.00	0.89	0.54	1.08	1.62	1.68
0.50	1.23	0.66	1.39	2.22	2.47
1.00	1.70	0.79	1.79	3.03	3.63
1.50	2.35	0.96	2.30	4.13	5.34
2.00	3.25	1.16	2.96	5.64	7.86
2.50	4.49	1.41	3.81	7.71	11.58
3.00	6.21	1.70	4.89	10.52	17.04
3.50	8.59	2.06	6.29	14.36	25.08

Table 9-10. Proposed Values of γ_{SE} for Various Methods to Estimate Immediate Settlement of Spread Footings on Cohesionless Soils Based on Arithmetic ln(*X*) Statistics ($\mu_{LNA\text{-}X}$ and $\sigma_{LNA\text{-}X}$) in Table 9-7

Reliability Index, β	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
0.00	1.00	1.00	1.10	1.60	1.70
0.50	1.25	1.00	1.40	2.20	2.45
1.00	1.70	1.00	1.80	3.05	3.65
1.50	2.35	1.00	2.30	4.15	5.35
2.00	3.25	1.15	2.95	5.65	7.85
2.50	4.50	1.40	3.80	7.70	11.60
3.00	6.20	1.70	4.90	10.50	17.05
3.50	8.60	2.05	6.30	14.35	25.10

9.2.5.1 Closed Form Solution in Terms of Accuracy

Using the nonlinear regression techniques and data in Table 9-9, the following closed form solution for γ_{SE} is obtained:

$$\gamma_{SE} = e^{J} \tag{9-6}$$

where $J = \beta(\sigma_{LNA\text{-}X}) - \mu_{LNA\text{-}X}$. The closed form solution in Equation 9-6 is valid only for lognormal distribution that was used to develop the data in Table 9-9. The statistics ($\sigma_{LNA\text{-}X}$ and $\mu_{LNA\text{-}X}$) in Equation 9-6 are based on ln(*X*) data.

For the example problem, $\beta = 0.50$, $\mu_{LNA\text{-}X} = 0.1173$, $\sigma_{LNA\text{-}X} = 0.6479$. Substituting these values in Equation 9-6 gives $\gamma_{SE} = 1.23$ as follows:

$$J = \beta(\sigma_{LNA\text{-}X}) - \mu_{LNA\text{-}X} = 0.50(0.6479) – 0.1173 = 0.2067$$

$$\gamma_{SE} = e^{J} = e^{0.2067} = 1.23$$

This result is the same as that obtained from the graphical solution or the Microsoft Excel-based procedure and reported in Table 9-9. As noted earlier, in the *AASHTO LRFD* framework, the load factor is rounded to the nearest 0.05; therefore, $\gamma_{SE} = 1.25$ should be used.

The Microsoft Excel procedure and/or the closed form solution demonstrated in the above example can be used to develop the values of γ_{SE} for any desired β using the lognormal distribution of *X* for the Schmertmann method. A similar approach can be used for other analytical methods and distributions.

9.2.6 Step 6: Review the Results and Develop the *SE* Load Factor for Target Reliability Indices

Figure 9-10 shows a plot of γ_{SE} versus β based on the data shown in Table 9-10. The current practice based on the *AASHTO LRFD* is as follows:

1. Use the Hough method to estimate immediate settlements.
2. Use $\gamma_{SE} = 1.0$.

The data in Table 9-10 and the graph on Figure 9-10 imply that $\beta \approx 1.65$ corresponds to the current practice noted above. $\beta \approx 1.65$ is based on the dataset in Table 9-2. If additional data were included, or if a different regional dataset were to be used, then the value of β may be different. However, based on a review of state practices performed as part of Samtani and Nowatzki (2006) and Samtani et al. (2010), it is anticipated that, based on its inherent conservatism, the value of β is anticipated to be large and greater than 1.0 for the Hough method and $\gamma_{SE} = 1.0$. The majority of the data points for the Hough method plot are below $\gamma_{SE} = 1.0$, which suggests significant conservatism in the Hough method. This is consistent with the earlier observation that the Hough method is conservative (overpredicts) by a factor of approximately two (see Table 9-4), which leads to the unnecessary use of deep foundations instead of spread footings.

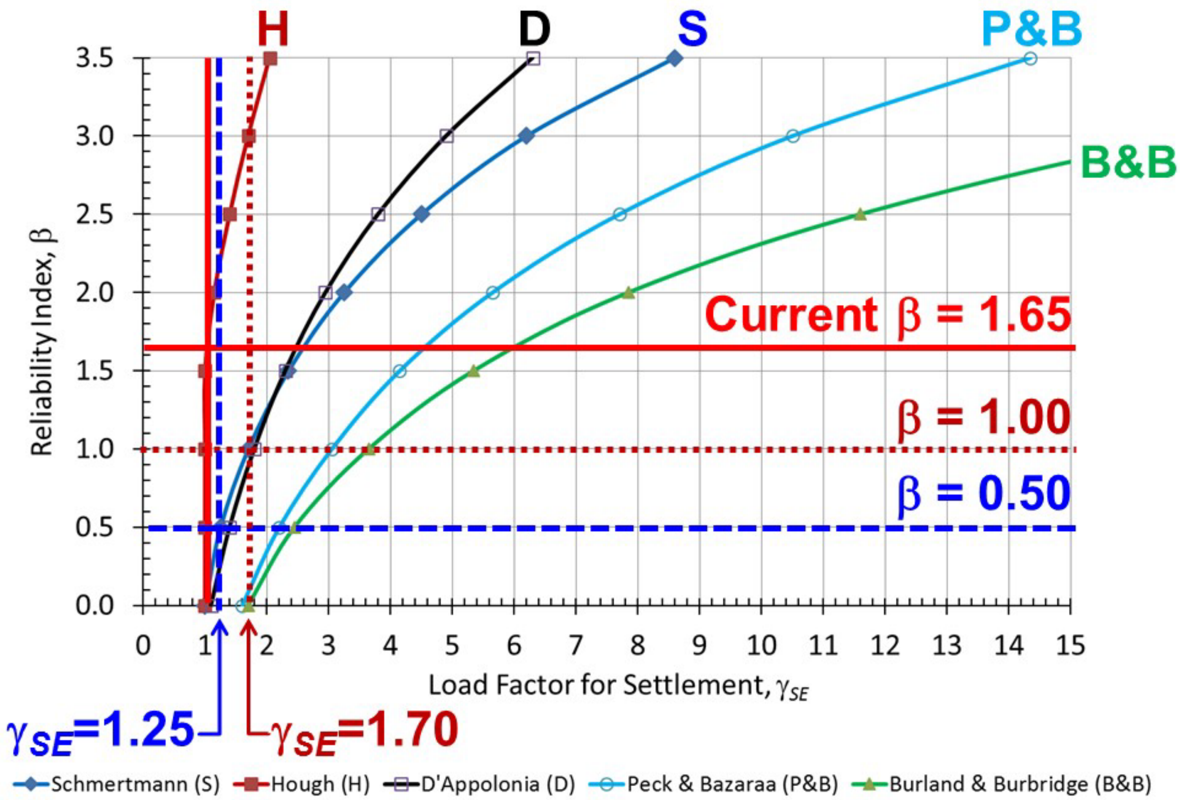


Figure 9-10. Evaluation of γ_{SE} Based on Current and Target Reliability Indices

Based on a consideration of reversible and irreversible limit states for bridge superstructures, as shown earlier, a target reliability index (β_T) in the range of 0.50 to 1.00, respectively, for the calibration of load factor γ_{SE} for the foundation movement limit state is acceptable. Settlement is clearly an irreversible limit state with respect to the foundation elements but may be reversible through intervention with respect to the superstructure. This type of logic would lead to consideration of 0.50 as the target reliability index for the calibration of immediate settlements under spread footings on cohesionless soils.

On Figure 9-10, the horizontal bold dashed line corresponds to β = 0.50 for service limit state evaluation. For β = 0.50, if a γ_{SE} = 1.25 is adopted, then it would encompass three of the five methods. The value of γ_{SE} = 1.25 includes the Schmertmann method, which is currently recommended by Samtani and Nowatzki (2006) and Samtani et al. (2010) and is commonly used in United States practice. Based on these observations, a γ_{SE} = 1.25 is recommended. Using similar approach, for β = 1.00, a γ_{SE} = 1.70 can be adopted.

9.2.7 Step 7: Select the *SE* Load Factor

As demonstrated in Steps 5 and 6, the γ_{SE} value can be determined for any reliability index (β) for various analytical methods. Use of the format shown on Figure 9-10 will lead to better regional practices in the sense that owners desiring to calibrate their local practices can readily see the implication of a certain method on the selection and cost of a foundation system. This is because the Figure 9-10 chart shows the reliability of various methods and permits the selection of an appropriate method that would lead to selection of a proper foundation system for a given set of β and γ_{SE}; that is, not use a deep foundation system when a spread foundation would be feasible. The agency that is calibrating a value of γ_{SE} based on a locally accepted analytical method must ensure that the chosen value of γ_{SE} is consistent with the serviceability of the substructure and superstructure design, as discussed in Step 6.

9.3 Calibration Using Concept of Bias

In Section 9.2, the data were analyzed in terms of Accuracy, X, which is defined as the ratio of predicted to measured settlement; that is, $X=S_P/S_M$. However, the data can also be analyzed in terms of Bias, λ, which is defined as the ratio of the measured to the predicted settlement, $\lambda=S_M/S_P$, as noted in Section 8.3.1. Thus, Bias is the inverse of Accuracy (that is, $\lambda = 1/X$). As was found with Accuracy data, a lognormal distribution is also applicable for Bias values, and in this case, the $\ln(\lambda)$ and $\ln(X)$ are correlated as follows:

$$\ln(\lambda) = \ln(1/X) = \ln(X^{-1}) = -\ln(X) \qquad (9\text{-}7)$$

Based on the above equation, the following statistics can be expected for $\ln(\lambda)$ data in comparison with $\ln(X)$ data:

Minimum $\ln(\lambda)$ = - Maximum $\ln(X)$

Maximum $\ln(\lambda)$ = - Minimum $\ln(X)$

Arithmetic mean of ln values: $\mu_{LNA\text{-}\lambda}$ for λ = - $\mu_{LNA\text{-}X}$ for X

Arithmetic standard deviation of ln values: $\sigma_{LNA\text{-}\lambda}$ for λ = $\sigma_{LNA\text{-}X}$ for X

Table 9-11 and Table 9-13 based on Bias, λ, correspond to the Accuracy, X, based on Table 9-3 and Table 9-4, respectively. Similarly, Table 9-12 and Table 9-14 correspond to Table 9-6 and 9-7, respectively. A review of the values in these corresponding tables show that they are in accordance with the above expectations.

Table 9-11. Bias ($\lambda = S_M/S_P$) Values Based on Data Shown in Table 9-2

Site	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
#1	0.4430	0.4667	0.5385	1.2069	1.1667
#2	0.3622	0.7128	1.7179	4.1875	5.5833
#3	1.0930	0.7769	3.1333	4.9474	7.2308
#4	1.6522	0.5205	1.3103	2.1111	1.9487
#5	2.0333	0.6224	1.6053	1.4524	1.0702
#6	0.8077	0.6885	0.8400	2.4706	1.2353
#7	3.3889	1.5250	3.2105	2.0333	3.2105
#8	0.9333	0.4667	1.0769	1.7500	2.0000
#9	1.4444	0.4906	1.3000	1.6250	2.3636
#10	1.0000	0.7250	1.2609	1.8125	3.2222
#11	0.6944	0.5319	0.8621	1.5625	4.1667
#14	1.1220	0.3622	0.8070	0.9200	1.1500
#15	0.2166	0.2329	0.4595	0.2500	0.2112
#16	0.8846	0.3108	0.5897	1.3529	1.3529
#17	1.1000	0.5366	0.9565	1.5714	1.9130
#20	0.5289	0.6095	1.3061	3.0476	1.1852
#21	1.5862	0.5476	0.8214	0.8846	1.4839
#22	1.2222	0.4748	1.0820	1.9412	1.0313
#23	0.5980	0.6162	1.0339	1.8485	1.3864
#24	0.4375	0.4590	0.7778	1.1200	0.7778

Table 9-12. Lognormal of Bias Values [ln(λ)] Based on Data Shown in Table 9-11

Site	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
#1	-0.8141	-0.7621	-0.6190	0.1881	0.1542
#2	-1.0157	-0.3386	0.5411	1.4321	1.7198
#3	0.0889	-0.2525	1.1421	1.5989	1.9783
#4	0.5021	-0.6529	0.2703	0.7472	0.6672
#5	0.7097	-0.4741	0.4733	0.3732	0.0678
#6	-0.2136	-0.3732	-0.1744	0.9045	0.2113
#7	1.2205	0.4220	1.1664	0.7097	1.1664
#8	-0.0690	-0.7621	0.0741	0.5596	0.6931
#9	0.3677	-0.7122	0.2624	0.4855	0.8602
#10	0.0000	-0.3216	0.2318	0.5947	1.1701
#11	-0.3646	-0.6313	-0.1484	0.4463	1.4271
#14	0.1151	-1.0155	-0.2144	-0.0834	0.1398
#15	-1.5299	-1.4572	-0.7777	-1.3863	-1.5550
#16	-0.1226	-1.1686	-0.5281	0.3023	0.3023
#17	0.0953	-0.6225	-0.0445	0.4520	0.6487
#20	-0.6369	-0.4951	0.2671	1.1144	0.1699
#21	0.4613	-0.6022	-0.1967	-0.1226	0.3947
#22	0.2007	-0.7448	0.0788	0.6633	0.0308
#23	-0.5141	-0.4842	0.0333	0.6144	0.3267
#24	-0.8267	-0.7787	-0.2513	0.1133	-0.2513

Table 9-13. Statistics of Bias, λ, Values Based on Data Shown in Table 9-11

Statistic	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
Count	20	20	20	20	20
Minimum	0.2166	0.2329	0.4595	0.2500	0.2112
Maximum	3.3889	1.5250	3.2105	4.9474	7.2308
μ_λ	1.0774	0.5838	1.2345	1.9048	2.1845
σ_λ	0.7212	0.2610	0.7406	1.0968	1.7402
COV_λ	0.6694	0.4471	0.5999	0.5758	0.7966

Note: μ_λ = Mean; σ_λ = Standard Deviation; COV_λ = Coefficient of Variation ($=\sigma_\lambda/\mu_\lambda$)

Table 9-14. Statistics of ln(λ) Values Based on Data Shown in Table 9-12

Statistic	Schmertmann	Hough	D'Appolonia	Peck and Bazaraa	Burland and Burbridge
Count	20	20	20	20	20
Minimum	-1.5299	-1.4572	-0.7777	-1.3863	-1.5550
Maximum	1.2205	0.4220	1.1664	1.5989	1.9783
$\mu_{LNA\text{-}\lambda}$	-0.1173	-0.6114	0.0793	0.4854	0.5161
$\sigma_{LNA\text{-}\lambda}$	0.6479	0.3807	0.5029	0.6226	0.7731

Note: $\mu_{LNA\text{-}\lambda}$ = arithmetic mean of ln(λ) values; $\sigma_{LNA\text{-}\lambda}$ = arithmetic standard deviation of ln(λ) values

9.3.1 Closed Form Solution in Terms of Bias

When the data are analyzed in terms of Bias, λ, the *SE* load factor γ_{SE} can be computed using Equation 9-8:

$$\gamma_{SE} = e^{K} \tag{9-8}$$

where $K = \beta(\sigma_{LNA\text{-}\lambda}) + \mu_{LNA\text{-}\lambda}$ and the different terms are as defined before.

For the example problem in Section 9.2.5, the goal is to compute load factor, γ_{SE}, for Schmertmann's method for a target reliability index, $\beta = 0.50$. Based on the data analyzed in terms of Bias, λ, from Table 9-14, $\mu_{LNA\text{-}\lambda} = -0.1173$, $\sigma_{LNA\text{-}\lambda} = 0.6479$. Substituting these values in Equation 9-8 gives $\gamma_{SE} = 1.23$ as follows:

$$K = \beta(\sigma_{LNA\text{-}\lambda}) + \mu_{LNA\text{-}\lambda} = 0.50(0.6479) + (-0.1173) = 0.2067$$

$$\gamma_{SE} = e^{K} = e^{0.2067} = 1.23$$

This result is the same as that obtained from the closed form solution based on Accuracy data in Section 9.2.5.1. As noted in Section 9.2.5, in the *AASHTO LRFD* framework, the load factor is rounded up to the nearest 0.05; therefore, γ_{SE} = 1.25 should be used.

Chapter 10. Application and Effect of *SE* Load Factor in Bridge Design Process

The *SE* load factor depends on the analytical method chosen to determine the settlement. Depending on the subsurface conditions, it is possible that different analytical methods may be used at different foundation locations along the bridge necessitating careful interpretation and application of the *SE* load factor. The application of the *SE* load factor is demonstrated in Appendix D through the use of a numerical example problem.

The meaning and use of the *SE* load factor, γ_{SE}, must be understood in the specific context of structural implications within the *AASHTO LRFD* framework. The main point is that the value of γ_{SE} is used to assess the structural implications such as the generation of additional (secondary) moments within a given span because of movement of one of the support elements, and the effect on the riding surface, and conceivably even appearance and roadway damage issues. Consider the example of γ_{SE} = 1.25 corresponding to a reliability index of 0.50 for vertical movement (settlement) based on the Schmertmann method. If taken literally, the value of γ_{SE} = 1.25 could be misinterpreted to mean that the settlement, δ_P, predicted by the Schmertmann method, needs to be increased by 25 percent, which will lead to 25 percent more total force effects (for example, moments). However, this interpretation is not entirely correct because the value of γ_{SE} (1.25 in this case) is just one of the many load factors in the service and strength limit state load combinations within the overall *AASHTO LRFD* framework.

Appendix E presents numerical example problems that explore the effect of including γ_{SE} in the bridge design process. These example problems demonstrate the application of the process to incorporate the effect of foundation movements in the bridge design process. Three bridges, a two-span, a four-span, and a five-span, are considered in the example problems. Settlements ranging from 0.6 in. to 4.8 in. at various support locations are considered along with the construction-point concept. Further, *SE* load factors of 1.25 and 1.75 are considered in addition the base case of *SE* load factor of 1.0 as per the current *AASHTO LRFD*; that is, before possible adoption of the revisions proposed as a result of this work. A total of eight examples are presented. Two key observations based on the examples are as follows:

- Use of the γ_{SE} and construction-point concept results in much less effects on controlling total moments and shears than would be indicated by the value of γ_{SE}.
- Even if the value of γ_{SE} changes from 1.25 to 1.75, a 40 percent increase, the difference in the force effects is less than approximately 6 percent.

These key observations are as expected because, as noted above, γ_{SE} is just one of the many load factors in the service and strength limit state load combinations within the overall

AASHTO LRFD framework and the additional (induced) force effects due to settlement are much smaller than the primary force effects due to dead load and live load. Refer to Appendix E for a more detailed discussion on the effect of the *SE* load factor.

The additional moments because of the effect of settlement are dependent on the stiffness of the bridge and the angular distortion. A limited study (Schopen, 2010) of several two- and three-span steel and prestressed concrete continuous bridges selected from the National Cooperative Highway Research Program Project 12-78 (Mlynarski et al., 2011) database showed that allowing the full angular distortion suggested in Table 5-1 could result in an increase in the factored Strength I moments, as little as 10 percent for the more flexible units considered to more than double the moment from only the factored dead and live load moments for the stiffer units. These order of magnitude estimates are based on elastic analysis without consideration of creep or change in structure stiffness based on construction sequence, which could significantly reduce the moments, especially for relatively stiff concrete bridges. For example, a W 36 x 194 rolled beam with a 10 in. x 1-7/8 in. bottom cover plate composite with a 96 in. x 7-3/4 in. deck is presented in Sen et al. (2011). The computed moments of inertia for the basic beam, short-term composite and long-term composite sections were in the approximate ratio 1:2:3. This indicates consideration of construction sequence, an appropriate choice of section properties, and possibly a time-dependent calculation of creep effects could be beneficial. Use of the construction-point concept would also mitigate the settlement moments. Schopen's results suggest that the use of permissible angular distortions approaching those currently allowed by the *AASHTO LRFD* requires careful consideration of the particular bridge and its design objectives. This suggests that if the computed angular distortions are between the current practice of various agencies (as discussed in Chapter 5) and the *AASHTO LRFD* limiting angular distortion criteria shown in Table 5-1, the resulting angular distortions may be tolerable and yet economy may be realized.

Chapter 11. Incorporating Values of *SE* Load Factor in the *AASHTO LRFD*

The calibration process discussed in Chapters 8 and 9 leads to *SE* load factors equal to or greater than 1.0 based on the analytical method for settlement and the chosen target reliability index. Table 3.4.1-3 of the *AASHTO LRFD* (see Figure 3-3) can either be expanded to include values of the *SE* load factor because this table includes load factors for superimposed deformations, or a similar additional table can be developed. The latter approach is proposed because it is anticipated that further research will lead to additional values of *SE* load factors related to various foundation types and movements. Table 11-1 presents proposed *SE* load factors, γ_{SE}.

Table 11-1. Load Factors for *SE* Loads

Foundation Movement and Movement Estimation Method	*SE*
Immediate settlement (effect of foundation movements on the bridge superstructure will be reversed by intervention; for example, shimming, jacking, etc.)	
• Hough method	1.00
• Schmertmann method	1.25
• Local method	*
Immediate settlement (effect of foundation movements on the bridge superstructure may not be reversed by intervention; for example, shimming, jacking, etc.)	
• Hough method	1.00
• Schmertmann method	1.70
• Local method	*
Consolidation settlement	1.00
Lateral movement	
• Soil-structure interaction method (P-y or strain wedge)	1.00
• Local method	*

*To be determined by the owner based on local geologic conditions.

In Table 11-1, the values of γ_{SE} for immediate settlement in row 1 are based on a target reliability index of 0.50, which assumes that the effect of irreversible foundation movements on the bridge superstructure will be reversed by intervention (for example, shimming, jacking, etc.). The values of γ_{SE} for immediate settlement in row 2 are based on reliability index of 1.00, which assumes that effect of irreversible foundation movements on the bridge superstructure may not be reversed by intervention (for example, shimming, jacking, etc.).

An owner may choose to use a local method that provides better estimation of foundation movement for local geologic conditions compared to methods noted in Section 10 (Foundations) of the *AASHTO LRFD*. In such cases, the owner will have to calibrate the γ_{SE} value for the local method using the procedures described in Chapters 8 and 9.

The value of γ_{SE}=1.00 for consolidation (long-term settlement time-dependent) settlement assumes that the estimation of consolidation settlement is based on appropriate laboratory and field tests to determine parameters (rather than correlations with index properties of soils) in the consolidation settlement equations in Article 10.6.2.4.3 of the *AASHTO LRFD*.

The value of γ_{SE} for soil-structure interaction methods in Table 11-1 for estimation of lateral movements may be increased to larger than 1.00 based on local experience and calibration using procedures described in Chapter 9.

Chapter 12. The "S_f-0" Concept

Chapters 8 and 9 have demonstrated a method to quantify the uncertainty of predicted movements for analytical models. The model uncertainty was calibrated and expressed through the load factor γ_{SE}. While all analytical models for estimating settlements have some degree of uncertainty, the uncertainty of the calculated differential settlement is larger than the uncertainty of the predicted total settlement at each of the two support elements used to calculate the differential settlement (for example, between an abutment and a pier, orbetween two adjacent piers). If one support element actually settles less than the amount predicted while the other support element actually settles the amount predicted, the actual differential settlement will be different than the difference between the two values of predicted settlement at the support elements.

The larger uncertainty of calculated differential settlement could be because of a number of factors. One such factor is the temporal and spatial uncertainties that are associated with inherent randomness of natural processes. The temporal uncertainties are from a time-related variability that may occur at a given support location and the possibility that this variability is not the same at all support locations. In contrast, variability that can occur over different support locations at a given time is referenced as spatial variability. Mathematical models, such as those discussed in Chapter 9, use simplified assumptions to account for these variabilities, but their success in doing so is a function of the level of subsurface investigations (field and laboratory) and interpretations of the subsurface data. These uncertainties can be reduced by increased and better subsurface investigations using appropriate investigative and interpretive techniques, but can never be completely addressed. This is further complicated by factors such as uncertainties due to variabilities in regional design and construction practices, maintenance protocols, and local environment leading to deterioration. Such uncertainties cannot be accounted for in a national code, which includes specific methods that were developed in a certain geographical region based on geologic formations specific to that region. For example, use of a prediction model that was developed based on data in the northeast United States for glacial till may not produce reliable results when applied to other regional geologic conditions such as cemented soil in the desert southwest United States. Although some uncertainties can be addressed by a load factor, such as γ_{SE} for a certain model, additional uncertainties must be accounted for, particularly when differential settlements are considered. Quantification of such additional uncertainties (sometimes categorized as epistemic uncertainties) may not be possible; therefore, practical limit state criteria need to be established to incorporate movement into the bridge design process.

As noted in Section 3.1, Article 3.12.6 of the *AASHTO LRFD* states, "Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered.” This requirement is consistent with the knowledge that not all uncertainties associated with foundation movements can be accounted for by a single load factor γ_{SE} for a certain model for prediction of movement. Based on these considerations and guidance in Barker et al. (1991) and Samtani et al. (2010), the following limit state criteria are suggested to estimate a realistic value of differential settlement and angular distortion:

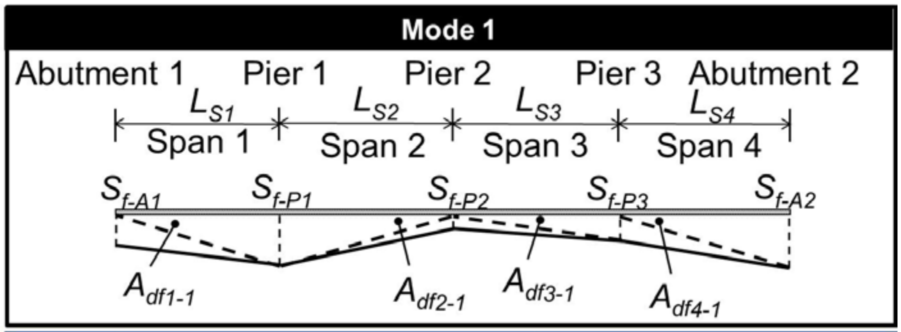
- The actual factored settlement of any support element could be as large as the factored settlement value calculated by using a given method.
- The actual factored settlement of the adjacent support element could be less, taken as zero in the limit, instead of the value calculated by using the same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

This concept is referred herein as the “S_f-0” concept,[2] with a value of S_f representing full factored settlement at one support of a span and a value of “0” representing zero settlement at an adjacent support. Use of the S_f-0 approach would result in an estimated maximum possible differential settlement between two adjacent supports equal to the larger of the two factored total settlements calculated at either end of any span. This approach also helps create the extreme values of differential settlement as required by Article 3.12.6 of the *AASHTO LRFD*, which states, “Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered.”

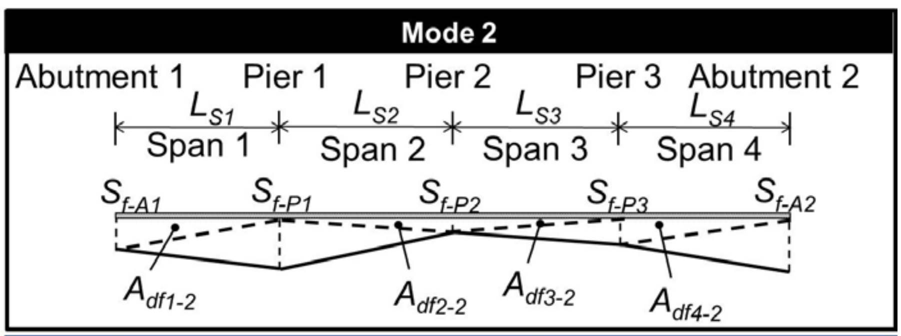
The application of the S_f -0 concept can be illustrated by considering the example of the four-span bridge on Figure 3-2. Before the application of the S_f-0 concept, the computed settlements S_{A1}, S_{P1}, S_{P2}, S_{P3} and S_{A2} are factored by multiplying each settlement by the γ_{SE} load factor applicable to the method that was used to compute that particular settlement. The factored settlement values are labeled as $S_{f\text{-}A1}$, $S_{f\text{-}P1}$, $S_{f\text{-}P2}$, $S_{f\text{-}P3}$ and $S_{f\text{-}A2}$. The factored differential settlement, Δ_f, and the corresponding factored angular distortion, A_{df}, values computed using the S_f-0 approach are shown on Figure 12-1. There are two possible modes, Mode 1 and Mode 2, depending on which support settlement is assumed to be zero. The values of factored differential settlement and corresponding factored angular distortions in the inset tables on Figure 12-1 represent the maximum values for each span according to the criteria above and should be used for design. The symbols are in accordance with $\Delta_{fi\text{-}j}$ and $A_{dfi\text{-}j}$ 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 factored angular distortion for each span is represented by the dashed lines on Figure 12-1. It should not be confused with the calculated factored total settlement profile that is represented by the solid lines. From the viewpoint of the damage to the bridge superstructure, the concept shown on Figure 12-1 is more important for continuous

[2] This discussion is based on the consideration of settlement (vertical movement). The S_f-0 concept can be considered in general terms as δ-0 concept where δ is a general symbol to designate any movement (vertical, lateral or rotational).

span structures than simple span structures because of the ability of the latter to permit larger movements at support elements.



Span	Factored Differential Settlement	Factored Angular Distortion
1	$\Delta_{f1\text{-}1} = S_{f\text{-}P1}$ (assume $S_{f\text{-}A1} = 0$)	$A_{df1\text{-}1} = \Delta_{f1\text{-}1}/L_{S1}$
2	$\Delta_{f2\text{-}1} = S_{f\text{-}P1}$ (assume $S_{f\text{-}P2} = 0$)	$A_{df2\text{-}1} = \Delta_{f2\text{-}1}/L_{S2}$
3	$\Delta_{f3\text{-}1} = S_{f\text{-}P3}$ (assume $S_{f\text{-}P2} = 0$)	$A_{df3\text{-}1} = \Delta_{f3\text{-}1}/L_{S3}$
4	$\Delta_{f4\text{-}1} = S_{f\text{-}A2}$ (assume $S_{f\text{-}P3} = 0$)	$A_{df4\text{-}1} = \Delta_{f4\text{-}1}/L_{S4}$

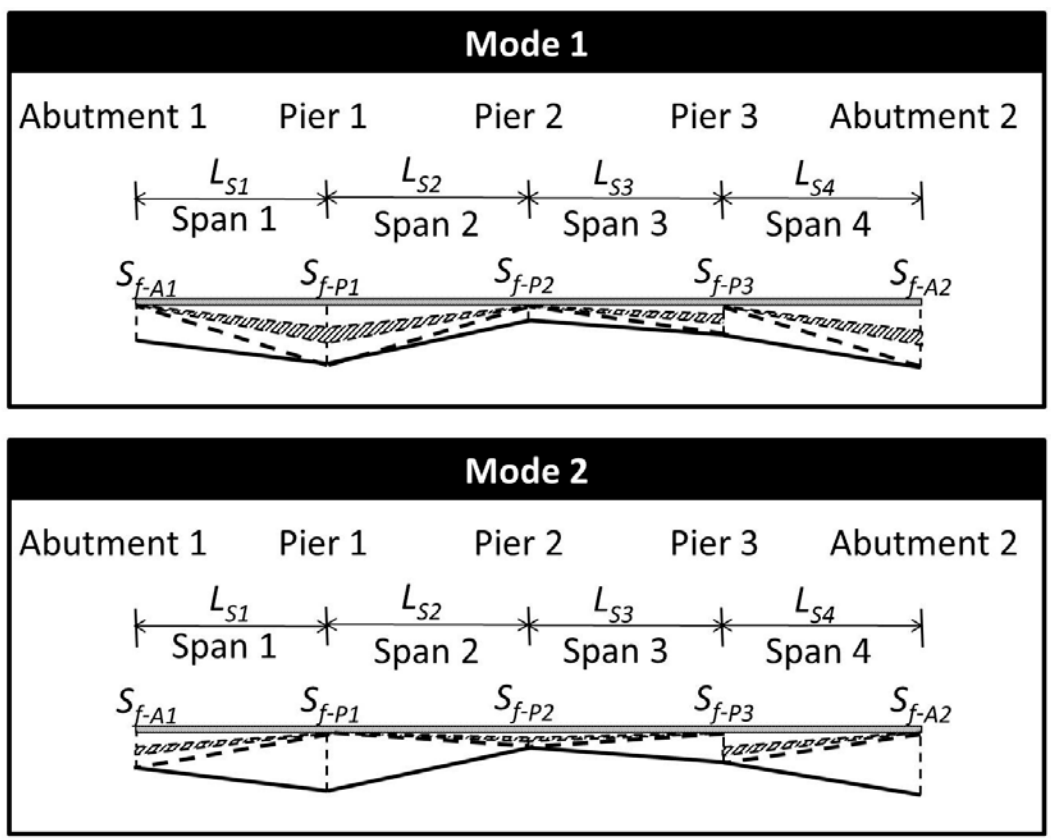


Span	Factored Differential Settlement	Factored Angular Distortion
1	$\Delta_{f1\text{-}2} = S_{f\text{-}A1}$ (assume $S_{f\text{-}P1} = 0$)	$A_{df1\text{-}2} = \Delta_{f1\text{-}2}/L_{S1}$
2	$\Delta_{f2\text{-}2} = S_{f\text{-}P2}$ (assume $S_{f\text{-}P1} = 0$)	$A_{df2\text{-}2} = \Delta_{f2\text{-}2}/L_{S2}$
3	$\Delta_{f3\text{-}2} = S_{f\text{-}P2}$ (assume $S_{f\text{-}P3} = 0$)	$A_{df3\text{-}2} = \Delta_{f3\text{-}2}/L_{S3}$
4	$\Delta_{f4\text{-}2} = S_{f\text{-}P3}$ (assume $S_{f\text{-}A2} = 0$)	$A_{df4\text{-}2} = \Delta_{f4\text{-}2}/L_{S4}$

Figure 12-1. Estimation of Maximum Factored Angular Distortion in Bridges – Mode 1 and Mode 2

All possible angular distortions shown on Figure 12-1 can be efficiently evaluated using a two-step process. In Step 1, for each span, divide the factored total relevant settlement, S_f, at one end of the span by the span length. In Step 2, repeat the calculation using the factored total relevant settlement at the other end of each span. By following this systematic two-step process, all viable modes of vertical movement profiles shown on Figure 12-1 will be evaluated.

With respect to the example of the four-span bridge and the angular distortions as shown in the inset tables on Figure 12-1, the use of the construction-point concept (Figure 6-2) would result in smaller angular distortions to be considered in the structural design. This will be true for any bridge evaluation. Using Figure 12-1 as a reference, Figure 12-2 shows a comparison of the profiles of the factored total settlements (solid lines), hypothetical maximum angular distortions (dashed lines), and the actual relevant angular distortions (hatched pattern zones) based on the construction-point concept. The range of the hatched pattern zone can be 25 to 75 percent of the factored total settlement value at the location where full settlement is assumed. 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. The relevant angular distortion would then be compared with the limit state criteria for angular distortions provided in the *AASHTO LRFD* Article 10.5.2.2 and Table 5-1 herein.



Calculated factored total settlement profile (refer to Figure 4-2)

Hypothetical factored settlement profile assumed for computation of maximum angular distortion

Range of factored relevant angular distortions using construction-point concept

Figure 12-2. Factored Angular Distortion in Bridges Based On Construction-point Concept

12.1 Foundations Proportioned for Equal Settlement

Geotechnical and structural specialists will occasionally try to proportion foundations for equal settlement. In this case, the argument is made that there will be no differential settlement. While this concept may work for a building structure because the footprint is localized, it is incorrect to assume a zero differential settlement for a long linear highway structure, such as a bridge or a wall because of the inevitable variation of the geomaterial properties along the length of the structure. Further, as noted earlier, the prediction of settlements from any given method is uncertain in itself. Hence, for highway structures, even where the foundations are proportioned for equal settlement, evaluation of differential settlement is recommended; this is 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 would be zero (that is, using the S_f-0 concept).

Chapter 13. Flowchart to Consider Foundation Movements in Bridge Design Process

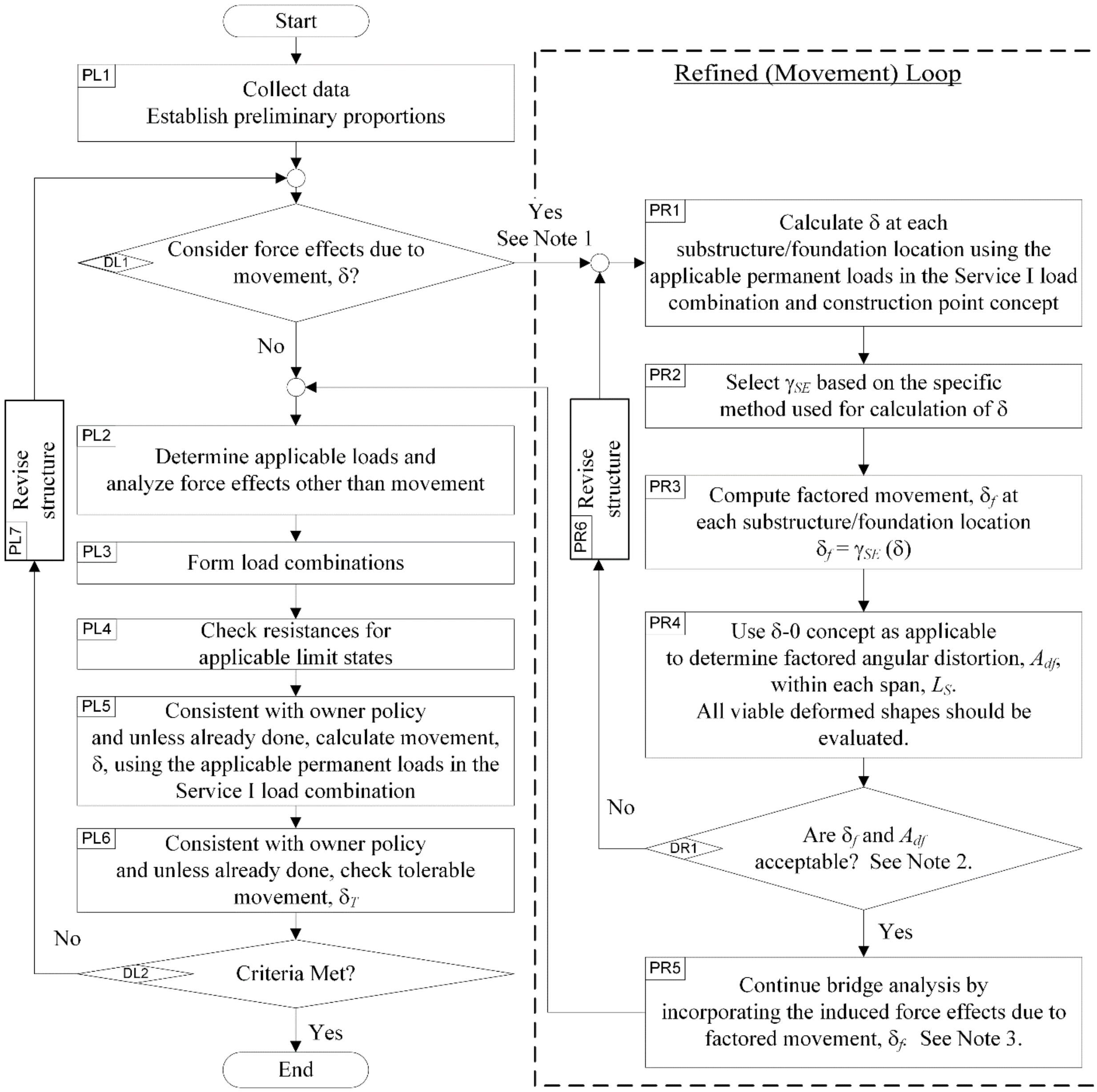
Figure 13-1 shows a flowchart to consider foundation movement in the bridge design process. The flowchart has two distinct parts, left and right. The left part outlines the process that a bridge designer may use without explicit consideration of foundation movements other than what is required in the 8th Edition of the *AASHTO LRFD*; that is, without considering the method-specific load factor, γ_{SE}, the construction-point concept, or the δ-0 concept. For convenience, this will be called the "legacy loop." The right part provides the recommended procedure to factor the movements and evaluate the effect on the structure using the factored movements. The sequence of activities in the right part of the flowchart is based on the discussions in this report, which includes the method-specific load factor, γ_{SE}, the construction-point concept, and the δ-0 concept. For convenience, this will be called the "refined (movement) loop." The flowchart applies to any type of foundation movement and hence the symbol δ is used for movements. If the flowchart is used for settlement, then symbol "*S*" may be substituted for δ.

It is not the intention of the illustrated design process to universally require additional design effort beyond what is required by the 8th Edition of the *AASHTO LRFD*, or approved owner policies that take advantage of well-documented past geotechnical practice. For example, if the geomaterials at a site are well-understood, and past experience shows that a deep foundation is the best option, or that a given service-bearing pressure results in acceptable foundation movements with minimal structural or geometric consequences, then the decision to base a new design on legacy practices is a viable option. If, on the other hand, site conditions are not within past successful practice, there is a desire to consider possible economies of design that alter the experience base, or the structure requires more careful consideration of possible foundation movements, then the additional provisions embodied in the refined (movement) loop will result in a more thorough assessment of the implications of foundation movements and the associated impact on the design and economy of the bridge.

Three notes are provided in the flowchart to include additional guidance for the designer.

Some of the key points associated with the flowchart are as follows:

1. The process ("P") related steps are indicated in rectangular boxes (▭). In the left ("L") part, there are six process boxes labeled PL1 to PL6. In the right ("R") part, there are five process boxes labeled PR1 to PR5.



Note 1: It may be efficient to run some early design iterations without including this loop until the proportions of the bridge are well developed, and then include this loop to consider the force effects from differential movements.

Note 2: Compare A_{df} to permissible angular distortion criteria and δ_f to permissible values at abutment interfaces and within spans in terms of vertical clearance under bridge. Guidance in Article 10.5.2 may be used to establish permissible values. Owner may establish other permissible values.

Note 3: Note that the γ_{SE} is used to factor the movements as shown in this flow chart. γ_{SE} also appears in Table 3.4.1-1 (Load Combinations and Load Factors). This does not imply a second application of γ_{SE} in the load combinations but rather it is an acknowledgement that the movements have already been factored. Use of the factored movements in a structural analysis program ensures that the output is factored value.

Figure 13-1. Consideration of Foundation Movements in the Bridge Design Process

2. The decision (“D”) related steps are indicated by diamond boxes (◇). In the left part, there are two decision boxes labeled DL1 and DL2. The right part contains one decision box labeled DR1.
3. The left and right parts are connected at two levels. The first connection is established when a bridge designer decides to proceed with either the legacy or refined (movement) loop in box DL1. The second connection is established after box PR5, once the designer has determined a favorable resolution of “Yes” to the decision in box DR1.
4. If the resolution to box DL2 is “No,” then the structure is revised and the flowchart is re-entered at box DL1. Likewise, if the resolution at DR1 is “No,” the structure is revised and the flowchart is re-entered at box PR1
5. If the answer is “No” at box DL1, then the designer goes through the process provided in boxes PL2 to PL6 using the legacy approach as follows:
    - In box PL2, structural analysis proceeds without use of the construction-point or δ-0 concepts as they are not incorporated into the legacy approach. Consideration of foundation movements is consistent with the owner’s implementation of the 8th Edition of the *AASHTO LRFD*.
    - Box PL3 indicates use of Table 3.4.1-1 of the *AASHTO LRFD* as applicable to the situation at hand. Depending on the owner’s policies, the values of γ_{SE} will effectively be zero or unity. In this case, the movement may be evaluated based on past local experience with similar structures.
6. If the answer is “Yes” at box DL1, then the designer goes through the process provided in boxes PR1 to PR5, using the refined (movement) approach. Note 1 is provided as guidance about entering the right side. The design proceeds as follows:
    - After the calculation of δ for the indicated loads in box PR1 and adjusting them for the construction-point concept, they are scaled (factored) as indicated in box PR3 using the method-specific values of γ_{SE} determined in box PR2.
    - These factored movements, δ_f, are used along with the δ-0 concept to calculate the factored angular distortions, A_{df} in box PR4.
    - In box DR1, the values of δ_f and A_{df} are compared to the applicable criteria. These criteria are geometric, not structural. Note 2 providers additional guidance.
    - If the results are not acceptable, the structure is revised and the design process returns to box PR1 to evaluate the modified structure.
    - If the results at box DR1 are acceptable, the structural force effects from the factored movements, δ_f, are calculated and are carried into the remaining steps of the legacy

loop. Note 3 is vital to the correct formulation of load combinations using Table 3.4.1-1 in box PL5.

7. The “Criteria” in box DL2 can include any criteria related to bridge design, such as deck grades, joint distress, crack control, and moment and shear resistance.
8. In boxes PL5 and PL6, the phrase “unless already done” acknowledges the possibility that the actions in these boxes may already have been performed by a designer who is entering these boxes after completing the right part of the flowchart.
9. If all structural and geometric criteria are satisfied in box DL2, the design is satisfactory; if not, the structure is modified and the design process returns to box DL1.

Chapter 14. Proposed Modifications to *AASHTO LRFD Bridge Design Specifications*

The work presented in this report can be considered for modifications of Section 3 and Section 10 of the *AASHTO LRFD* as follows:

- In Article 3.4.1, include a new load factor table for *SE* as shown in Table 11-1, with appropriate specifications and commentary to explain the various values of the *SE* load factor.
- In Article 10.5.2.2, include a step-by-step procedure and appropriate commentary for implementation of the *SE* load factor in conjunction with the construction-point and S_f-*0* concept.
- In Section 3, as an appendix, include the flowchart for incorporation of foundation movements in the bridge design process.
- In Article 10.6.2.4.2, include the Schmertmann method.

These proposed draft modifications for Section 3 of the *AASHTO LRFD* are included in Appendix F, while those for Section 10 of *AASHTO LRFD* are included in Appendix G.

Chapter 15. Application of Calibration Procedures

Although the focus of this report is the calibration of foundation movements, the calibration procedures described are general and can be considered for calibration of any civil engineering feature. The calibration procedure developed in the SHRP2 Project R19B and explained herein provides additional tools for the continued development of reliability-based design specifications. Two particular classes of problems can be treated with the calibration procedure used in this report:

- Class A: This involves situations where consideration of movements is required to inform the "two-hump" distribution of load and resistance, or their proxies, as illustrated on Figure 8-2. In the illustrated situation, the calibration must account for load-movement characteristics and the distribution of load and resistance.
- Class B: This involves situations where there is so little data on the distribution of either loads or resistances, or their proxies, that one of them must be considered as determinant, where there is no variability as shown on Figure 8-6 and Monte Carlo simulation is unstable.

Class A problems are typical of geotechnical features where the load-movement (Q-δ) curves have a much flatter initial portion compared to the steep initial portions for structural materials, such as concrete and steel. During the calibration of service limit states in the SHRP2 Project R19B, Class B problems arose several times, where the variability of resistance proxy could not be established and the calibration process described in this report for a geotechnical service limit state was adapted for structural service limit states. Extension to the strength limit state calibration is also possible.

One use of the calibration procedure described in this report is further research and development of *SE* load factors for other types of movements, features such as retaining structures, and the use of other movement calculation methods than those documented herein. The *SE* load factors that are developed in the future can be included in Table 11-1. Three examples within the geotechnical field are as follows:

- Lateral movement of deep foundations: In this case, a figure similar to Figure 9-1 will need to be developed based on data from methods such as the *P-y* method and the strain wedge method. The remainder of the calibration process will remain identical, as shown in Chapter 9.
- Face movements of MSE walls: In this case, a figure similar to Figure 9-1 will need to be developed based on data from MSE walls with inextensible and extensible reinforcements. Within each category, different reinforcement materials types and configurations can be

included (for example, steel strips, steel grids, geogrids, and geotextiles). The remainder of the calibration process will remain identical, as shown in Chapter 9.

- Pullout resistance of soil reinforcements: In this case, a figure similar to Figure 9-1 will need to be developed based on pullout test data for soil reinforcements embedded in different soil types (such as native, compacted, sand, and clay) and different soil reinforcements (such as anchors and nails). The remainder of the calibration process will remain identical, as shown in Chapter 9.

Chapter 16. Summary

This report was developed as part of an IAP and focuses on the work related to foundation movements developed as part of the SHRP2 Project R19B. Its purpose is to explain the implementation of calibrations for foundation movements into the bridge design process. The scope was to bring together the relevant content of the SHRP2 Project R19B and additional materials developed since the issuance of the report by Kulicki et al. (2015). These additional materials include expanded discussions to further clarify concepts, a flowchart to guide the bridge design process, and parametric analysis using actual design examples. These additional materials are also useful as background information as part of AASHTO's balloting process for incorporation of the *SE* load factor and other associated modifications in the *AASHTO LRFD Bridge Design Specifications*.

The *SE* load factors in this report were developed based on a limited dataset of 20 points from the northeast United States to demonstrate the calibration process. All the data points showed measured immediate settlement values smaller than 1.0 in. The minimum value was 0.23 in. and the maximum value was 0.94 in. Use of larger datasets, which also include larger settlements, and from different parts of the United States (that is, different regional geologies) may result in different values of *SE* load factors. However, changes in the *SE* load factors within the range of parametric analyses documented in Appendix E are not expected to affect the observations made in this report. This report will serve as a useful reference for researchers as well as agencies desiring to develop *SE* load factors based on expanded databases or on local methods that are better suited to their regional geologies and subsurface investigation techniques.

The consideration of foundation movements in the bridge design process can lead to the use of cost-effective structures with more efficient foundation systems. The proposed approach and modifications will help avoid overly conservative criteria that can lead to (a) foundations that are larger than needed, or (b) a choice of less economical foundation type (such as using a deep foundation at a location where a shallow foundation would be adequate).

Implementation of the proposed procedures may often allow consideration of larger foundation movements. The associated structural and geometric impacts can be mitigated by the use of the construction-point concept and the S_f-0 concept. These are incorporated into the design process following the recommended specification revisions illustrated in the flowchart in Chapter 13. The revised design procedures and the method-specific load factor are combined to produce flexibility in comparing the alternative foundations and structures and provide more uniform serviceability and safety.

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Appendix A
Conventions

Appendix A. Conventions

Documents from various sources such as the American Association of State Highway and Transportation Officials (AASHTO) , Federal Highway Administration (FHWA), and the second Strategic Highway Research Program (SHRP2) are referenced in this report. Each reference document has its own style and organization, which often creates confusion during cross-referencing of documents. For instance, the *AASHTO LRFD Bridge Design Specifications* based on the load and resistance factor design (LRFD) platform are organized in sections and articles in a two-column format, while FHWA documents are organized in chapters and sections in single-column format. Different fonts (for example, Times New Roman, and Calibri), font styles (such as regular and *italic*), and font sizes (for example, 12 point and 10 point) are used in different documents. Finally, different styles for referencing other documents are used. This report has been formatted as per the FHWA style guidelines.

The following are the important points with respect to conventions used in this report:

1. The *AASHTO LRFD Bridge Design Specifications* are referenced as the *AASHTO LRFD* to fulfill AASHTO's citation requirements. Similarly, the format of AASHTO's *Standard Specifications for Highway Bridges* is used to refer to the *AASHTO LRFD Bridge Design Specifications* based on the allowable stress design (ASD) and load factor design (LFD) platform.
2. The *AASHTO LRFD* refers to the 8th Edition issued in 2017 and its subsequent interims.
3. A document reference that is unique and often cited is referenced with a single word after the first usage. For example, after an initial reference as Moulton et al. (1985), it is subsequently referenced in the body of the report simply as Moulton.
4. A specific section within the *AASHTO LRFD* is referenced as Section # of the *AASHTO LRFD.* Similar convention is followed for a specific Article, figure, or table in the *AASHTO LRFD*.
5. The approach of chapter and section in a single-column format with 12 point Calibri font is used except for Appendices F and G, which use the two-column section and article format with 10 point Times New Roman font because these appendices include proposed modifications for the *AASHTO LRFD*.
6. Because this report will be used for input related to modifications in the *AASHTO LRFD*, the notations are italicized to be consistent with the *AASHTO LRFD*. For example, S and Δ_f.
7. The *AASHTO LRFD* uses the word "deformation" and "movement" interchangeably when discussing foundations or bridge supports. The word "movement" is used with foundation in this report unless a direct quote is provided from a document where the word "deformation" was used.

Appendix B
Equal Load Probability Formulation for Estimation of Load Factors

Appendix B. Equal Load Probability Formulation for Estimation of Load Factors

B.1 General *AASHTO LRFD* Calibration Framework

Figure 8-1 in Chapter 8 shows the basic *AASHTO LRFD* framework in terms of distributions of loads and resistances. The objective of the framework is to develop a probabilistic-based specification that is able to define load factors, γ, and resistance factors, ϕ, such that the design of a feature (for example, bridge superstructure, foundation, etc.) can meet a target reliability index, β_T. A reliability index, β, is an alternative representation of probability of exceedance, P_e.

Thus, the *AASHTO LRFD* framework involves a relationship between three primary variables: load factor, γ, resistance factor, ϕ, and reliability index, β. Such a three-way relationship makes determination of the values of variables difficult. The *AASHTO* LRFD framework used the following steps to develop its design specifications:

1. Establish a target reliability index, β_T.
2. Select load factors, γ.
3. Using a given set of load factors, γ, calibrate resistance factors, ϕ, and calculate reliability index, β, for a series of representative designs.
4. If the calculated values of reliability indices, β, are closely clustered near the target reliability index, β_T, then the combinations of chosen load factors and resistance factors are acceptable.
5. If the calculated values of reliability indices, β, are not acceptable, then revise the values of resistance factors, ϕ, and repeat Steps 3 to 5.
6. If varying the resistance factors, ϕ, in Step 5 still does not result in an acceptable value of reliability indices, β, then adjust the load factors in Step 2 and repeat Steps 3 to 6.

Step 1 is done by a code-writing body (for example, AASHTO) and at an owner level (for example, a Department of Transportation). Step 3 is performed in a variety of ways ranging from judgment based on expert elicitation (an element of the Delphi process), back-fitting with past successful practice based on allowable stress design (ASD) or load factor design (LFD), reliability theory (for example, Monte Carlo approach), and a combination of these different ways. But, prior to calibration of resistance factors, it is necessary to develop load factors, γ, in Step 2. This appendix provides the background of how load factors were developed for the *AASHTO LRFD* framework.

B.2 Determination of Load Factors

The determination of load factors involves recognition of uncertainty in the loads under consideration. Because different loads have different amounts of uncertainty, it can be expected that there will be different load factors whose values reflect the relative level of uncertainty. For example, the uncertainty in the dead load is expected to be smaller than that in the live load. Thus, the load factor of dead load will be smaller than that of the live load.

The uncertainty in a given load, *Q*, can be characterized by the following statistical parameters:

- The coefficient of variation, COV_Q, which is defined as the standard deviation, σ, divided by the mean value, μ, of the ratio of the measured to predicted values of the load.
- The Bias factor, λ_Q, which is the mean value of the ratio of measured to nominal (design, calculated, or predicted) values of the load.

Using the above statistical parameters, one way to determine the starting values of load factor, γ, in Step 2 above is to assign an equal probability of exceedance to each load component under consideration; for example, dead load of components, wearing surface and live load; hence, the term equal load probability (ELP) formulation. Thus, the load factors may be formulated using the following equation (Kulicki and Mertz, 1993; Nowak, 1999; Nowak and Collins, 2000; Allen et al. 2005):

$$\gamma = \lambda_Q \, [1 + (\eta)(COV_Q)] \qquad \text{(B-1)}$$

where the value of η depends on the desired β_T (or alternatively a corresponding value of P_{eT}). For calibration of load factors for strength limit state, the loads were assumed to have a normal (bell-shaped) distribution. It is important to understand that, while Equation B-1 appears to have great accuracy, the starting values are not sacrosanct, and some deviation is to be expected as the calibration process proceeds. In fact, there is no requirement to use Equation B-1, but it has been shown to yield reasonable starting values in some past calibrations.

Examples of past calibrations include the calibration of dead load and live load for the Strength I load combination in the *AASHTO LRFD*. Approximately 200 representative bridges were selected from various regions of the United States and a set of hypothetical bridges was developed based on these 200 bridges (Kulicki and Mertz, 1993). Values of η ranging from 1.5 to 2.5 were evaluated and a value of η = 2 was selected to estimate the starting values of load factors. The effect of the resulting load factors on the reliability indices for the bridges in the calibration set was observed and evaluated. Initially, the dead load was considered to have two subsets (factory-made components and cast-in-place components), but they were combined for simplicity even through strict application of Equation B-1 would have produced individual starting load factors for each considered subset of the dead load. Further, the load factors were rounded to the nearest 0.05. The need to adjust the starting values from Equation B-1 for practical reasons is further illustrated by considering live load. From the point of view of the end user of design specifications, it is convenient for the load factors to be constants and same single value for all force effects; that is, not span or structure-type dependent. Because the Bias on the HL93 live load model was not a constant compared to the 75-year mean maximum values, and the Bias and COVs were somewhat different for shear and moment, Equation B-1 would not yield a constant load factor for live load for all cases. An average value of Bias and COV could be used to develop a single load factor for live load from Equation B-1, but that is just another way of selecting a reasonable starting value of a load factor and proceeding with the reliability analysis to see if the calculated reliability indices, β, are suitably clustered near the target reliability index, β_T. Thus, some practical accommodations were made. Eventually, a slightly conservative load factor for live load was selected.

The key point based on the examples of the historical development of load factors for dead load and live load is that the use of Equation B-1 with a value of η = 2 is not sacrosanct but merely a starting point having the attraction of equal probability for all the loads under consideration. Due to practical reasons, some deviations from the starting values are to be expected. As long as the combination of load factors, resistance factors, load models, and resistance models used yields the target reliability index, β_T, within acceptable tolerances, those deviations do not matter. The corollary is that the starting values for load and resistance factors could be selected arbitrarily as long as the results of the reliability analysis are acceptable.

B.3 Considerations for Foundation Movements

The load factors from strength limit state give an approximate P_{eT} = 0.02 based on assumption of normal distribution for loads with small values of Bias (typically less than 1±0.05 for dead loads) and COV (typically less than 0.20) as noted in Kulicki and Mertz (1993) and Nowak (1999). In contrast, as can be seen from the data in Table 9-13 in Chapter 9, foundation movements have much larger and wider range of Bias and COV values depending on the analytical prediction methods. From Chapter 9, it can be seen that the data for foundation movements based on different analytical prediction methods are better modeled using a lognormal probability distribution function. For calibration of foundation movements in terms of implications for structural limit states such as cracking, the P_{eT} values are close to 0.15 (that is, 15 percent), which is equivalent to a target reliability index of β_T = 1.0, as indicated in Table 8-2 in Chapter 8. Further, the uncertainty in a given analytical method for predicting movements must be considered in the context of target tolerable movements established by a bridge designer based on consequences of exceeding different structural service limit states as discussed in Section 8.2. Thus, the ELP formulation for strength limit states, which is based on normal distribution, small Bias, and COV values, and small P_{eT} values, may not be as applicable for the case of foundation movements.

Nevertheless, the core concept of ELP of achieving approximately uniform level of reliability must still be considered across different structural service limit states. In other words, the reliability of foundation movements must be consistent with the level of reliability that is considered in the structural service limit states. This means that the formulation for estimating the load factors for movements must have a mechanism that can index the predicted movements, δ_P, to target tolerable movements, δ_T, and ensure a uniform level of reliability for different structural limit states. This formulation is based on the ratio of predicted to tolerable movements (δ_P/δ_T), or vice versa (δ_T/δ_P), and then calibrating load factors for movements for different analytical prediction methods using a target $P_{eT} = 0.15$, which is equivalent to a target reliability index of $\beta_T = 1.0$. This formulation is discussed in Section 8.3.3 and Section 8.3.4.

Appendix C
Reliability Index Based on Normal and Lognormal Probability Distribution Functions

Appendix C. Reliability Index Based on Normal and Lognormal Probability Distribution Functions

Article 3.2 (Definitions) of the *AASHTO LRFD* defines reliability index, β as “a quantitative assessment of safety expressed as the ratio of the difference between the mean resistance and mean force effect to the combined standard deviation of resistance and force effect.” This definition is an approximation of Equation C-1 for reliability index where P_f is the probability of failure and Φ^{-1} is the inverse standard normal distribution.

$\beta = - \Phi^{-1}(P_f)$ (C-1)

The AASHTO definition is the same as Equation C-1, if the following assumptions are satisfied:

1. Both resistance and force effect (that is, load) are normally distributed random variables, and
2. The resistance and force effect (that is, load) are statistically independent; that is, uncorrelated.

The limit state function, g, can be expressed as

$g = R – Q$ with $P_f = P\ (g< 0)$ (C-2)

or

$g = R/Q$ with $P_f = P\ (g< 1)$ C-3)

If g is a normal random variable, then

$\beta = \mu_g/\sigma_g$ (C-4)

where, μ_g = mean value of g, and σ_g = standard deviation of g

and $P_f = \Phi\ (- \beta)$ (C-5)

The coefficient of variation, COV, is defined as the ratio of the standard deviation and the mean, and hence COV for g, COV_g, can be expressed as follows:

$COV_g = \sigma_g/\mu_g$ (C-6)

and $\beta = 1/COV_g$ (C-7)

Thus, for the case of normally distributed random variable, β is a function of only COV_g; that is, only one parameter.

If g is a lognormal random variable, then

$$P_f = \Phi\,(-\,\mu_{lng}/\sigma_{lng}) \tag{C-8}$$

where, μ_{lng} = mean value of ln(g), and σ_{lng} = standard deviation of ln(g). The formulas for calculation of μ_{lng} and σ_{lng} can be found in textbooks (for example, Nowak and Collins, 2000; Haldar and Mahadevan, 2000).

Equation (C-8) can then be used with Equation (C-1) to calculate the value of reliability index, β, for the case of a lognormally distributed random variable. In this case, β is a function of two parameters; that is, μ_{lng} and σ_{lng} (or COV_{lng}). Therefore, for lognormally distributed random variables, the relationship between β and P_f is not unique but depends on the COV.

Figure C-1 shows the variation of β with P_f for a range of COV values for a lognormal distribution. The line labeled "Normal" on Figure C-1 shows the variation of reliability index, β, as a function of probability of failure, P_f, for the case a normally distributed random variable. The line for normal distribution also represents a bound for the lognormal distribution when the value of COV approaches zero. As can be seen from Figure C-1, for most of the range of reliability index, β, an assumption of a normal distribution as made by the *AASHTO LRFD* is generally conservative in the sense that for a given reliability index, it gives a larger probability of failure compared to a lognormal distribution.

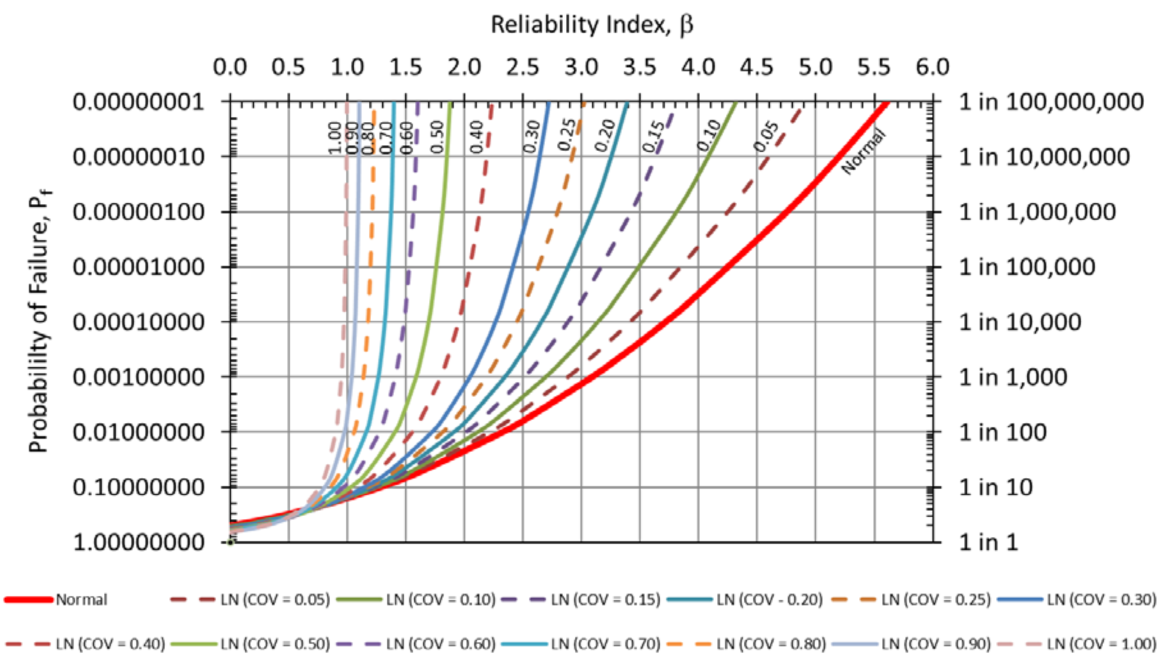

Figure C-1. Variation of Reliability Index as a Function of Normal and Lognormal (LN) Distributions (Numbers on Curves Represent Values of Coefficient of Variation, COV)

Figure C-2 shows an alternative representation of the data on Figure C-1. Each curve is for a given probability of failure, P_f, as identified in the legend. The upper horizontal axis is a bound for the lognormal distribution when the value of COV approaches zero. The intersection of each of the curves with the upper horizontal axis also represents the limiting case for normal distribution. As discussed in Chapter 8, the reliability index for various structural service limit states in context of foundation movements is recommended to be 0.50 for reversible-irreversible case and 1.0 for irreversible case. As can be seen from Figure C-2, the curves are virtually straight for reliability indices of 0.5 and 1.0. This observation indicates that for lognormal distributions with large values of COV, the value of the reliability index based on normal distribution (that is, the value on the upper horizontal axis) can be used in the *AASHTO LRFD* calibrations.

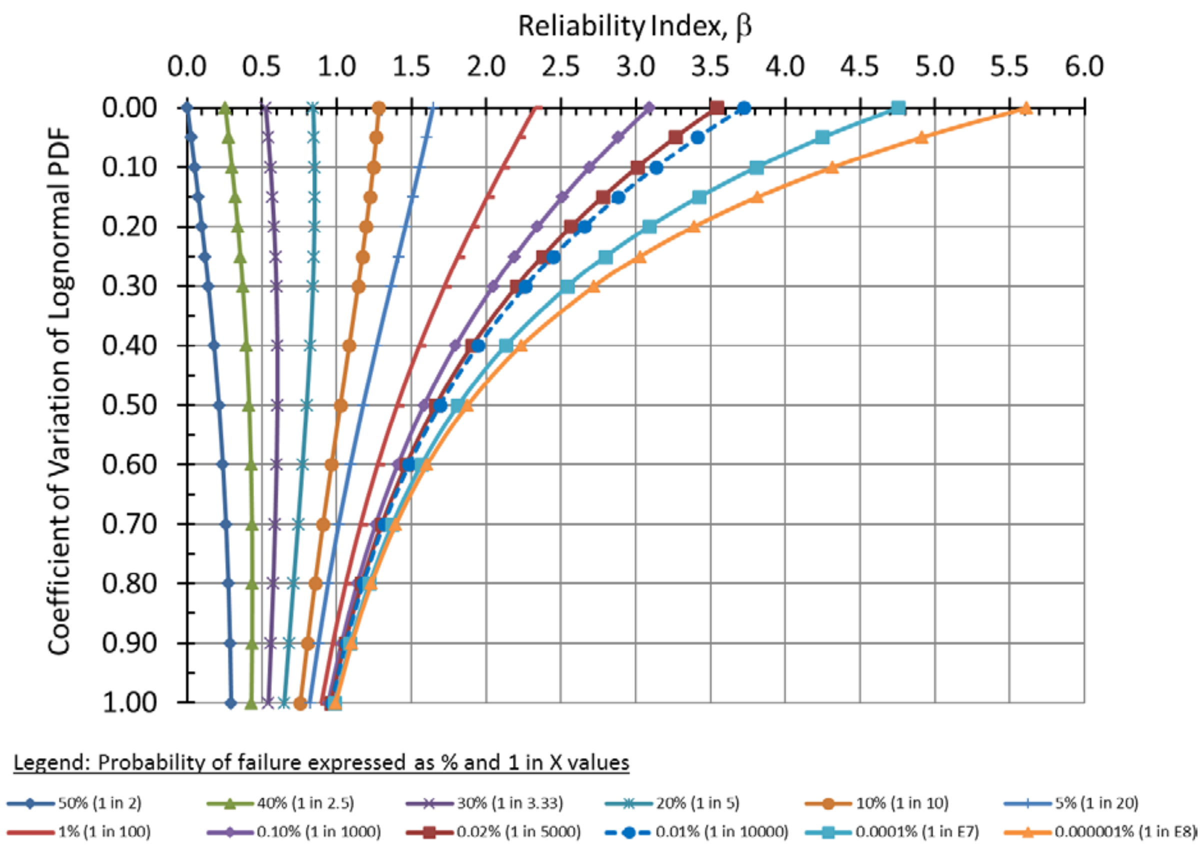

Figure C-2. Variation of Reliability Index as a Function of Coefficient of Variation, COV, of Lognormal (LN) Probability Distribution Function

In contrast to the strength limit states, the service limit states are user-defined limiting conditions that affect the function of the structure under expected service conditions. Violation of the service limit states occurs at loads much smaller than those for the strength limit states. Because there is no danger of collapse if a service limit state is violated, a smaller value of target reliability index may be used for the service limit states. Project R19B (Kulicki et al., 2015) indicates that reliability index values of less than approximately 1.0 are found for the probability of failure associated with limit states that may be attained during service conditions; for example, cracking, fatigue, etc. Therefore, it is important to understand the implications of the various curves on Figure C-1 for values of reliability index less than 1.0.

Figure C-3 shows a close-up view of the curves on Figure C-1. The various curves for a lognormal distribution converge rapidly toward the curve for a normal distribution for reliability index values typical of service limit states. In fact, for reliability index values less than approximately 0.60, the probability of failure based on the lognormal distributions for the range of COVs from 0.05 to 1.00 is actually smaller compared to that for the normal distribution. Interpretations of the trends of various curves on Figure C-3 need to be tempered based on practical considerations. It must be realized that the differences in the probability of failure for a given reliability index observed on Figure C-3 occur in the tail areas of the normal and lognormal distributions. In these tail areas, such theoretical differences are of little practical significance in view of the relatively sparse data that are typically available for a given limit state. Further, the accuracy of the data does not justify selecting a certain curve based on a lognormal distribution over the normal distribution curve for values of reliability index less than approximately 1.0.

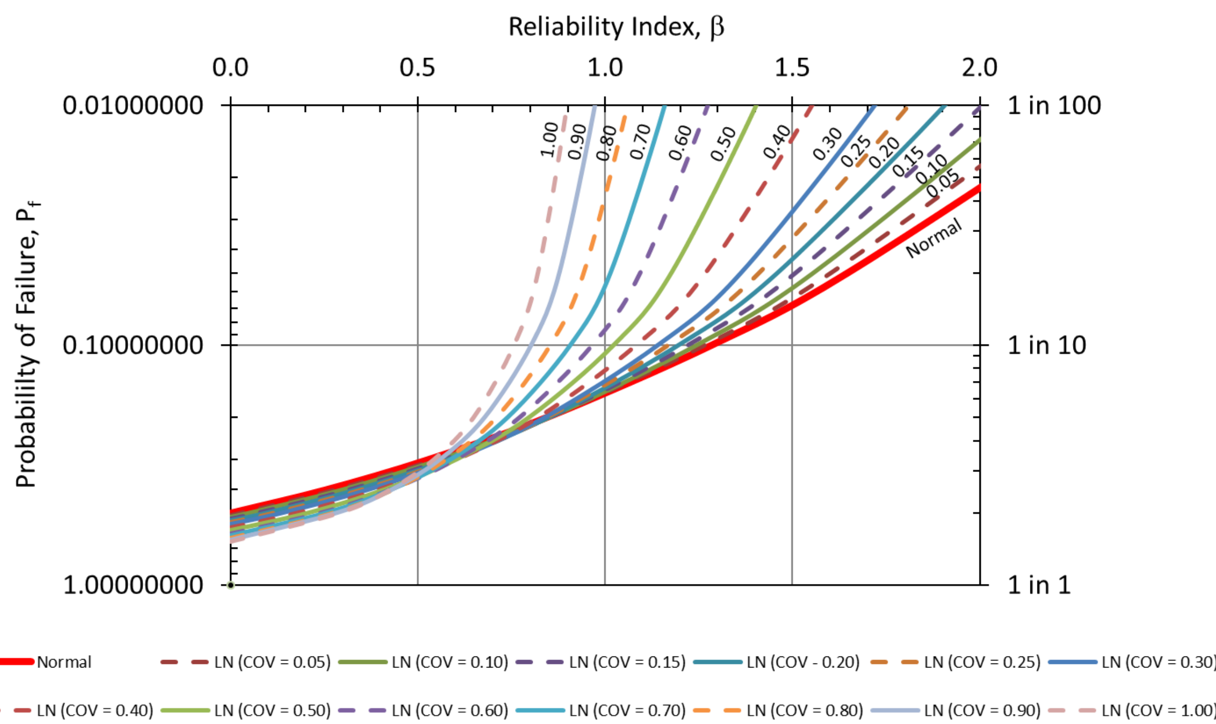

Figure C-3. Variation of Reliability Index as a Function of Normal and Lognormal (LN) Distributions in the Zone of Interest for Service Limit State Calibration (Numbers on Curves Represent Values of Coefficient of Variation, COV)

In general, a force (load) effect or resistance value represents the cumulative effect of a large number of underlying random variables. For example, a crack in a concrete structure may be due the effect of variables such as dead load, live load, temperature change, foundation settlement, strength of concrete, etc. Each of these variables may in turn be a function of other variables. For example, foundation settlement at the support element of a bridge is affected by a number of geological features such as the vertical and lateral distribution of various soil types and their geotechnical properties. Regardless of whether the limit state evaluated is structural or geotechnical, each of the variables that contributes to the chosen value of load or resistance can have its own distribution. As per the central limit theorem, the distribution of a sum of a large number of statistically independent random variables each with its own distribution converges to a normal distribution if none of the random variables tends to dominate the sum (Nowak and Collins, 2000; Haldar and Mahadevan, 2000). Based on the central limit theorem and the practical considerations discussed previously, the normal distribution is used in Section 9.2.5 as the basis for the relationship between reliability index and probability of failure (exceedance) in the calibration of service limit states within the *AASHTO LRFD* framework.

Appendix D
Application of *SE* Load Factor

Appendix D. Application of *SE* Load Factor

Figure 6-1 in Chapter 6 shows the construction-point concept. The horizontal dashed line on Figure 6-1b is annotated with “Long-term settlement (if applicable).” In Chapter 6, in the text related to Figure 6-1,, it is stated that “…long-term settlements will continue under the total construction load (*Z*) as shown by the dashed line on Figure 6-1.” The proposed design approach incorporates the construction-point concept in conjunction with the γ_{SE} load factor. In Chapter 11, Table 11-1 provides the value of *SE* load factors for the immediate and consolidation type settlements. This appendix provides a numerical example to illustrate the application of the *SE* load factor for the case where a support element such as an abutment or a pier may experience long-term consolidation settlement after the short-term immediate settlement.

Example: Assume a four-span bridge similar to that shown in Chapter 12, Figure 12-2. Table D-1 provides the unfactored predicted settlements along with the methods used for computing the settlements. For the data given in Table D-1, develop the factored total relevant settlement, S_f, values that will be used for bridge structural analysis for the target reliability index, β_T, of 0.50; that is, assuming that the owner commits to reversing the detrimental effect of settlement by intervention (for example, shimming, jacking, etc.).

Table D-1. Unfactored Predicted Settlements

	Immediate Settlement (Note 1)			Consolidation Settlement (in.) (Note 2)	Total Relevant Settlement, *Str* (in.) (Note 3)
Support Element	**Total (in.)**	**Relevant (in.)**	**Prediction Method**		
Abutment 1	1.90	0.80	Schmertmann	2.00	2.80
Pier 1	3.20	1.90	Hough	3.60	5.50
Pier 2	2.00	0.90	Hough	3.20	4.10
Pier 3	2.10	1.20	Schmertmann	4.00	5.20
Abutment 2	1.50	0.70	Schmertmann	1.90	2.60

Note 1: The total immediate settlement is based on the assumption of instantaneous application of all loads, while the relevant settlement is based on the assumption of loads due to superstructure only. With respect to Figure 6-1, the relevant immediate settlement is based on loads after the completion of the substructure. In other words, the difference between the total and relevant values represents the magnitude of settlement that occurs prior to the construction of the superstructure.

Note 2: The consolidation settlement is based on the total load of the structure.

Note 3: The total relevant settlement is obtained by adding the relevant immediate settlement and the consolidation settlement.

The computations of the factored total relevant settlement, S_f, at each support element are as follows:

Abutment 1: From Table 11-1, γ_{SE} = 1.25 for Schmertmann method with β_T of 0.50, and γ_{SE} = 1.00 for consolidation settlement. Thus, S_f = (1.25)(0.80 in.) + (1.00)(2.00 in.) = 3.00 in.

Pier 1: From Table 11-1, γ_{SE} = 1.00 for Hough method with β_T of 0.50, and γ_{SE} = 1.00 for consolidation settlement. Thus, S_f = (1.00)(1.90 in.) + (1.00)(3.60 in.) = 5.50 in.

Pier 2: From Table 11-1, γ_{SE} = 1.00 for Hough method with β_T of 0.50, and γ_{SE} = 1.00 for consolidation settlement. Thus, S_f = (1.00)(0.90 in.) + (1.00)(3.20 in.) = 4.10 in.

Pier 3: From Table 11-1, γ_{SE} = 1.25 for Schmertmann method with β_T of 0.50, and γ_{SE} = 1.00 for consolidation settlement. Thus, S_f = (1.25)(1.20 in.) + (1.00)(4.00 in.) = 5.50 in.

Abutment 2: From Table 11-1, γ_{SE} = 1.25 for Schmertmann method with β_T of 0.50, and γ_{SE} = 1.00 for consolidation settlement. Thus, S_f = (1.25)(0.70 in.) + (1.00)(1.90 in.) = 2.78 in.

Table D-2 summarizes the computed factored total relevant settlements, S_f, at each support element. These values are used for the bridge structural analysis.

Table D-2. Summary of Factored Total Relevant Settlements

Support Element	Factored Total Relevant Settlement, S_f (in.)
Abutment 1	3.00
Pier 1	5.50
Pier 2	4.10
Pier 3	5.50
Abutment 2	2.78

This example deliberately used different methods to predict immediate settlement at different support elements to illustrate the process of computation of factored total relevant settlement, S_f. In practice, the same method of predicting immediate settlement is often used.

Appendix E
Examples and Parametric Study

Appendix E. Examples and Parametric Study

The purpose of this appendix is to explore the effect of applying the proposed specification revisions on the controlling moments and shears in several bridges through a parametric study using actual bridges. Detailed design examples are presented to demonstrate implementation of the process to incorporate the effect of foundation movements in the bridge design process. The right side of the flowchart in Chapter 13, Figure 13-1, is used in the demonstration.

The examples are based on several recently constructed continuous span steel I-girder bridges by AECOM. Table E-1 shows the characteristic of the bridges.

Table E-1. Bridge Characteristics

Bridge	Material	Span lengths (ft)	Girder Spacing
A	Steel I-Girders	50, 50	7 ft-2 in.
B	Steel I-Girders	168, 293, 335, 165	11 ft-2 in.
C	Steel I-Girders	120, 140, 140, 140, 120	12 ft-3 in.

The examples examine the following force effects due to foundation settlement (*SE*):

- Maximum positive moment within each span along with the minimum (that is, maximum negative) moment at the same location
- Maximum positive moment and minimum moment (that is, maximum negative) at each intermediate support
- Maximum shear at each abutment
- Maximum and minimum shear on both sides of intermediate supports

During the design procedure, the magnitude of settlement at each support at different stages of construction will be determined by bridge designer based on input provided by the geotechnical engineer using an owner-approved method for settlement prediction. For these examples, the following assumptions have been made:[3]

- Use of the same owner-approved method to predict immediate settlement at all support locations. For this method, the *SE* load factor is 1.25 and 1.75 corresponding to reliability index, β, of 0.50 and 1.00, respectively. These values were chosen based on a request by the AASHTO T-15 committee to explore the effect of a large (40 percent) increase in *SE* load factor on the bridge design.

[3] Use site-specific data and owner-approved method for settlement analysis on actual projects. The values chosen for the examples are for illustration purpose only.

- No long-term consolidation settlements. Thus, the total settlement, S_t, in these example problems is equal to the immediate settlement.
- The values of the total settlements, S_t, were chosen such that a large range of settlement magnitudes can be evaluated to allow for a comprehensive parametric study.
- The values of total relevant settlements, S_{tr}, are 50 percent of total settlement, S_t.

The settlement data can be organized using the format shown in Tables E-2 to E-5. Each of these tables is briefly discussed below.

- Table E-2 shows the assumed predicted total settlements, S_t. These values should be computed as per the first part of box PR1 of the flowchart in Chapter 13, Figure 13-1. These are settlements based on the assumption of instantaneous application of all loads. The settlement values in Table E-2 were chosen to explore the effect of a wide range (0.6 in. to 4.8 in.) of settlements on the bridge design process. These values, coupled with the large range of *SE* load factors up to 1.75, explore most of the possibilities that may be encountered in practice.

Table E-2. Unfactored Predicted Total Settlements, S_t

Example	Bridge	Unfactored Predicted Settlement, S_t (in.)					
		Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
E1	A	0.8	1.6	N/A	N/A	N/A	0.6
E2	B	1.9	3.9	4.8	1.9	N/A	2.5
E3	B	0.5	1.0	1.2	0.5	N/A	0.6
E4	C	0.9	1.5	1.8	1.0	2.3	1.4

Note: N/A = not applicable

- Table E-3 shows the estimated total relevant settlements, S_{tr}. These values are as per the second part of box PR1 of the flowchart shown on Figure 13-1. These are settlements computed after consideration of construction-point concept. For this example problem, these values are assumed to be 50 percent of the predicted total settlements, S_t.

Table E-3. Estimated Unfactored Total Relevant Settlements, S_{tr}

Example	Bridge	Unfactored Total Relevant Settlement, S_{tr} (in.)					
		Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
E1	A	0.40	0.80	N/A	N/A	N/A	0.30
E2	B	0.95	1.95	2.40	0.95	N/A	1.25
E3	B	0.25	0.50	0.60	0.25	N/A	0.30
E4	C	0.45	0.75	0.90	0.50	1.15	0.70

- Table E-4 and Table E-5 show the computed factored total relevant settlements, S_f, corresponding to *SE* load factor, γ_{SE}, of 1.25 and 1.75, respectively. These values are computed by multiplying the estimated total relevant settlements, S_{tr}, from Table E-3 by the applicable *SE* load factor for the method used to estimate the settlement. This is done as per box PR3 of the flowchart shown on Figure 13-1. The values in Table E-4 and Table E-5, as appropriate, are those that will be used in the structural analysis as per box PR4 and box PR5 of the flowchart shown on Figure 13-1.

Table E-4. Factored Total Relevant Settlements, S_f, for γ_{SE} = 1.25

Example	Bridge	Factored Total Relevant Settlement, S_f (in.)					
		Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
E1(1)	A	0.50	1.00	N/A	N/A	N/A	0.38
E2(1)	B	1.19	2.44	3.00	1.19	N/A	1.56
E3(1)	B	0.31	0.63	0.75	0.31	N/A	0.38
E4(1)	C	0.56	0.94	1.13	0.63	1.44	0.88

Note: $S_f = \gamma_{SE} (S_{tr})$

Table E-5. Factored Total Relevant Settlements, S_f, for γ_{SE} = 1.75

Example	Bridge	Factored Total Relevant Settlement, S_f (in.)					
		Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
E1(2)	A	0.70	1.40	N/A	N/A	N/A	0.53
E2(2)	B	1.66	3.41	4.20	1.66	N/A	2.19
E3(2)	B	0.44	0.88	1.05	0.44	N/A	0.53
E4(2)	C	0.79	1.31	1.58	0.88	2.01	1.23

Note: $S_f = \gamma_{SE} (S_{tr})$

- Table E-6 and Table E-7 show the factored angular distortion, A_{df}, for *SE* load factor, γ_{SE}, of 1.25 and 1.75, respectively, calculated as per box PR4 of the flowchart on Figure 13-1. The angular distortions were computed using the two-step process discussed in Chapter 12. In this process, the angular distortion for any span is first calculated by dividing the factored total relevant settlement, S_f, at one end of the span (taken from Table E-4 and Table E-5) by the span length, and then the calculation is repeated using the factored total relevant settlement at the other end of each span. By following this two-step process, all viable modes of vertical movement profiles will be efficiently evaluated (for example, see different patterns in Mode 1 and Mode 2 on Figure 12-1 for a four-span bridge).

Table E-6: Compute Factored Angular Distortions, A_{df}, for γ_{SE} = 1.25

Example	Bridge	Factored Angular Distortion, A_{df} (rad.)				
		S_f at the left end of the span divided by the span length				
		Span 1	Span 2	Span 3	Span 4	Span 5
E1(1)	A	0.0008	0.0017	N/A	N/A	N/A
E2(1)	B	0.0006	0.0007	0.0007	0.0006	N/A
E3(1)	B	0.0002	0.0002	0.0002	0.0002	N/A
E4(1)	C	0.0004	0.0006	0.0007	0.0004	0.0010
Example	Bridge	S_f at the right end of the span divided by the span length				
		Span 1	Span 2	Span 3	Span 4	Span 5
E1(1)	A	0.0017	0.0006	N/A	N/A	N/A
E2(1)	B	0.0012	0.0009	0.0003	0.0008	N/A
E3(1)	B	0.0003	0.0002	0.0001	0.0002	N/A
E4(1)	C	0.0007	0.0007	0.0004	0.0009	0.0006

Table E-7: Compute Factored Angular Distortions, A_{df}, for γ_{SE} = 1.75

Example	Bridge	Factored Angular Distortion, A_{df} (rad.)				
		S_f at the left end of the span divided by the span length				
		Span 1	Span 2	Span 3	Span 4	Span 5
E1(2)	A	0.0012	0.0023	N/A	N/A	N/A
E2(2)	B	0.0008	0.0010	0.0010	0.0008	N/A
E3(2)	B	0.0002	0.0002	0.0003	0.0002	N/A
E4(2)	C	0.0005	0.0008	0.0009	0.0005	0.0014
Example	Bridge	S_f at the right end of the span divided by the span length				
		Span 1	Span 2	Span 3	Span 4	Span 5
E1(2)	A	0.0023	0.0009	N/A	N/A	N/A
E2(2)	B	0.0017	0.0012	0.0004	0.0011	N/A
E3(2)	B	0.0004	0.0003	0.0001	0.0003	N/A
E4(2)	C	0.0009	0.0009	0.0005	0.0012	0.0009

At this stage, the calculated factored total relevant settlement, S_f, and the factored angular movement, A_{df}, now need to be evaluated in accordance to box DR1 of the flowchart on Figure 13-1. Note 2 in that flowchart that is applicable to box DR1 provides guidance on the comparison of angular distortion values. For the examples herein, assume that the factored settlements in Table E-4 and Table E-5 meet project-specific criteria and are therefore acceptable. Further, assume that the guidance in Article 10.5.2 of the *AASHTO LRFD* is applicable based on when the limiting angular distortion is 0.004 for continuous-span bridges. In Table E-6 and Table E-7, no value is larger than 0.004. Because both the factored total relevant settlement and factored angular distortion values are found to be acceptable, the design process can proceed further to box PR5 of the flowchart. As per box PR5, the design process now needs to evaluate induced force effects due to the factored settlement values in Table E-4 and Table E-5 and incorporate these induced force effects into the bridge design by going to box PL2 of the flowchart. The process to incorporate these induced force effects in bridge design process is discussed next.

E.1 Incorporating Induced Force Effects in Bridge Design Process

Eight example problems are discussed in this appendix and are arranged as follows at the end of this appendix: Example E1(1), Example E1(2), Example E2(1), Example E2(2), Example E3(1), Example E3(2), Example E4(1), and Example E4(2). Each example problem has (i) a set of

14 tables with 7 tables applicable to moment and 7 applicable to shear considerations, (ii) a total of 4 pages with 2 pages applicable to moment computations and 2 pages applicable to shear computations, and (iii) page numbers in the upper right-hand corner of each page. The computations in all tables were performed using a spreadsheet.

The organization of the tables in each example problem is demonstrated below using Example E1(1) as an illustration. The tables in other examples are organized in a similar manner.

- Tables E1(1)-M1 and E1(1)-S1 provide the values of moment and shear that were computed by a bridge design program.
- Table E1(1)-M2 provides the values of the predicted unfactored total settlement, S_t, at each support element. For convenience, the values in Table E1(1)-S2 are repeated from Table E1(1)-M2. The values in these tables are based on the applicable values from Table E-2.
- Table E1(1)-M3 provides the values of the estimated unfactored total relevant settlement, S_{tr}, at each support element. For convenience, the values in Table E1(1)-S3 are repeated from Table E1(1)-M3. The values in these tables are based on the applicable values from Table E-3.
- Table E1(1)-M4 provides the values of the factored total relevant settlement, S_f, at each support element. For convenience, the values in Table E1(1)-S4 are repeated from Table E1(1)-M4. The values in these tables are based on the applicable values from Table E-4 (for $\gamma_{SE} = 1.25$) and Table E-5 (for $\gamma_{SE} = 1.75$).
- Table E1(1)-M5 contains computations for moments. The top three rows of Table E1(1)-M5 contain the unfactored moments (copied from the corresponding top three rows of Table E1(1)-M1). The next three rows in Table E1(1)-M5 contain values of moments calculated by scaling the moments determined based on unit (1 in.) settlement in the last three rows of Table E1(1)-M1. The computation of values in last eleven rows of Table E1(1)-M5 are demonstrated below for moment at Pier 1 location (similar computations apply at other support locations):[4]
    - Effect of unfactored S_{tr} at Abutment 1: (0.40 in./1.00 in.)(-277 kip-ft) = -111 kip-ft
    - Effect of unfactored S_{tr} at Pier 1: (0.80 in./1.00 in.)(555 kip-ft) = 444 kip-ft
    - Effect of unfactored S_{tr} at Abutment 2: (0.30 in./1.00 in.)(-277 kip-ft) = -83 kip-ft
    - Total unfactored effect of S_{tr} at all supports:
        - +ve value: 0 kip-ft + 444 kip-ft = 444 kip-ft

[4] Values are rounded to nearest whole number.

  - -ve value: -111 kip-ft – 83 kip-ft = -194 kip-ft

- Total factored effect of settlement using γ_{SE} = 1.00 and S_t:
  - +ve value: (1.60 in. /1.00 in.)(555 kip-ft) (1.00) = 888 kip-ft
  - -ve value: (0.80 in./1.00 in.)(-277 kip-ft)(1.00) + (0.60 in. /1.00 in.)(-277 kip-ft) (1.00) = -388 kip-ft
- Total factored effect of settlement using γ_{SE} = 1.25 and S_{tr}:
  - +ve value: (0.80 in./1.00 in.) (1.25)(555 kip-ft) = 555 kip-ft
  - -ve value: (0.40 in./1.00 in.) (1.25)(-277 kip-ft) + (0.30 in./1.00 in.) (1.25) (-277 kip-ft) = 242 kip-ft
- Total factored effect of settlement using γ_{SE} = 1.00 and S_{tr} (since γ_{SE} = 1.00, these values are the same as those above for total unfactored effect of S_{tr} at all supports):
  - +ve value: (0.80 in./1.00 in.) (1.00)(555 kip-ft) = 444 kip-ft
  - -ve value: (0.40 in./1.00 in.) (1.00)(-277 kip-ft) + (0.30 in./1.00 in.) (1.00) (-277 kip-ft) = -194 kip-ft

For comparison purposes, four cases of Service 1 and Strength 1 load combinations were developed as follows in Table E1(1)-M6 and Table E1(1)-M7:

- **Case 1**: DL + LL with no settlement. This case represents designs performed without consideration of settlement. In this case, dead load (DL) and live load (LL) are factored according to the load factors for Service I and Strength I load combinations.
- **Case 2**: DL + LL with settlement using load factor γ_{SE} = 1.00. This case represents the procedure as per the current *AASHTO LRFD* provisions related to incorporation of the settlement (assuming γ_{SE} = 1.00). In this case, (a) dead load and live load are factored according to the load factors for Service I and Strength I load combinations, and (b) the total settlement, S_t, with a load factor γ_{SE} = 1.00 is used.
- **Case 3**: DL + LL with settlement using construction-point concept and method-specific load factor γ_{SE}. This case represents the proposed design procedure that incorporates the construction-point concept along with method-specific load factor γ_{SE}. In this case, (a) dead load and live load are factored according to the load factors for Service I and Strength I load combinations, and (b) the relevant settlements, S_{tr}, is used with the load factor appropriate to the method used to estimate the settlement. In the examples, as noted earlier, an owner-approved method for settlement prediction method is used with load factor γ_{SE} = 1.25 and 1.75.

- **Case 4**: This case is the same as Case 2 except that the total relevant settlement, S_{tr}, with a load factor γ_{SE} = 1.00 is used. This case represents the practice in some Departments of Transportation (for example, Washington State Department of Transportation), where total relevant settlements are used instead of total settlements.

- The numerical computations in Table E1(1)-M6 and Table E1(1)-M7 are based on the equations in the first columns of these tables and the corresponding values from Table E1(1)-M5. In these computations, consideration of individual settlements or groups of settlements is required by Article 3.12.6 of the *AASHTO LRFD* that states, "Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered." The purpose of the provision is to account for the possibility that some of the foundation units may settle less that predicted, or even undergo no settlement. Use of the word "considered" in the *AASHTO LRFD* denotes that judgment may be used to reduce the number of conditions to be investigated. Further, Article 3.4.1 of the *AASHTO LRFD* indicates that, "Load combinations which include settlement shall also be applied without settlement." The purpose of this provision is to alert the designer to make sure that the force effect due to settlement must not be used to reduce the permanent force effects. For demonstration purposes, these examples were developed assuming that the worst possible set of settlements for each individual force effect was realized. In the examples, these requirements were efficiently implemented by summing all the positive and all the negative moments and shears at each point of interest in the example structure. The sum of the positive settlement contributions, factored as shown, should be combined with the dead load and positive live load contributions to determine the maximum value of the force effect under consideration. The sum of the negative settlement contributions, factored as shown, should be combined with the dead load and the negative live contributions to determine the minimum (that is, maximum negative) value of the force effect under consideration. For convenience in the spreadsheet, all combinations were calculated; however, only the controlling values are carried forward in design.
- The numerical computations in Tables E1(1)-S5, E1(1)-S6, and E1(1)-S7 follow the approach similar to the corresponding tables E1(1)-M5, E1(1)-M6, and E1(1)-M7, respectively.

E.2 Evaluation of Results

For each example, the results were evaluated based on comparisons between the four cases through ratios for both of the moment and shear values. Table E-8 lists the five ratios that were evaluated. In these ratios, relevant settlement is estimated using construction-point concept while total settlement is not based on construction-point concept.

Table E-8. Summary of Ratios Evaluated

#	Ratio	Comparison
1	Case 3 to Case 1 (Note 1)	Proposed specification using relevant settlement and $\gamma_{SE} \geq 1$ with current specification without consideration of settlement
2	Case 3 to Case 2	Proposed specification using relevant settlement and $\gamma_{SE} \geq 1$ with current specification using total settlement and $\gamma_{SE} = 1$
3	Case 2 to Case 1 (Note 1)	Current specification using total settlement and $\gamma_{SE} = 1$ with no consideration of settlement
4	Case 3 to Case 4	Proposed specification using relevant settlement and $\gamma_{SE} \geq 1$ with current specification using relevant settlement and $\gamma_{SE} = 1$
5	Case 4 to Case 2	Current specification using relevant settlement and $\gamma_{SE} = 1$ with current specification using total settlement and $\gamma_{SE} = 1$

Note 1: This ratio provides information regarding the level of potential under-design when the effect of settlement is not considered in analysis.

Because all results are compared for both the maximum and minimum values of the force effects, the ratios representing the controlling force effects are shown in bold large font typeface in the Table E1(1)-M6 and Table E1(1)-M7. Conversely, all non-controlling force effects are shown in a smaller font.

The following general observations are made based on the eight examples:

1. Generally, the values from Case 3 may be larger or smaller than the results from Case 1 or Case 2 depending on the magnitude of the differential settlement and the direction of the angular rotation in different spans.
2. For the three steel I-girder bridges with the settlements assumed for the examples, the difference in the controlling moments and shears is not significant for Bridges 2 and 3 regardless of whether Case 3 is compared to Case 1 or Case 2. The difference is more significant for Bridge 1. This indicates that the factored design force effects for shorter spans will be affected by the proposed provisions more than longer spans.
3. Based on a comparison of the ratios, it is observed that the induced force effects for Case 3 ($\gamma_{SE} > 1.0$) as compared to Case 2 ($\gamma_{SE} = 1.0$) in accordance with current *AASHTO LRFD* specifications are not in direct proportion to the value of the load factor, that is, $\gamma_{SE} = 1.25$ or 1.75. This is to be expected because the effect of the settlement is one of several components combined to determine the design load effect for a load combination. The

exact value of the change in total force effects would be a function of many factors such as bridge superstructure type and configuration, substructure type, foundation type, and use of construction-point concept. In general, the use of the construction-point concept reduces the effect of the settlement on the total force effects. In the example problems, the changes in total force effects did not significantly alter the controlling values for design. In such cases, consideration could be given to use of more efficient and cost-effective foundation types as well as other appropriate members of the bridge structure.

4. Figure E-1 shows a summary of the results from Example E2(1) and Example E2(2). In these examples, large settlements ranging up to 4.80 in. were applied at the supports of the four-span bridge. In Example E2(1), γ_{SE} = 1.25 was used while in Example E2(2) γ_{SE} =1.75 was used. Thus, a direct comparison of the effect of change in value of γ_{SE} can be performed. In this comparison, the increase in load factor is 40 percent (1.25 to 1.75). On Figure E-1, the results are tabulated in terms of ratios of Case 3 to Case 2 and Case 3 to Case 4 for Service I moments, Strength I moments, Service I shears, and Strength I shears. A description of the ratios is included in Table E-8.

   Based on a ratio of Case 3 to Case 2, a 40 percent increase in γ_{SE} (that is, 1.25 to 1.75) results in the following:

   - 2.4 to 6.4 percent increase in moments for Service I load combination
   - 1.7 to 4.5 percent increase in moments for Strength I load combination
   - 0.9 to 3.3 percent increase in shears for Service I load combination
   - 0.6 to 2.3 percent increase in shears for Strength I load combination

   Based on a ratio of Case 3 to Case 4, a 40 percent increase in γ_{SE} (that is, 1.25 to 1.75) results in the following:

   - 2.4 to 6.3 percent increase in moments for Service I load combination
   - 1.7 to 4.5 percent increase in moments for Strength I load combination
   - 0.9 to 3.3 percent increase in shears for Service I load combination
   - 0.6 to 2.3 percent increase in shears for Strength I load combination

   The ranges of increases for Case 3 to Case 2 and Case 3 to Case 4 are similar. For the four-span bridge example, the controlling force effects for moments and shears did not change appreciably when the proposed provisions are implemented. Comparable effects were noted in the other two examples (two-span and five-span bridges).

Ratios of Governing Force Effects: Case 3 to Case 2			
Service I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	0.913	0.971	6.4%
Pier 1	0.915	0.972	6.2%
Span 2-0.5L	0.944	0.981	3.9%
Pier 2	0.966	0.989	2.4%
Span 3-0.5L	0.965	0.988	2.4%
Pier 3	0.949	0.983	3.6%
Span 4-0.6L	0.949	0.983	3.6%

Ratios of Governing Force Effects: Case 3 to Case 2			
Strength I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	0.939	0.980	4.4%
Pier 1	0.937	0.979	4.5%
Span 2-0.5L	0.961	0.987	2.7%
Pier 2	0.975	0.992	1.7%
Span 3-0.5L	0.975	0.992	1.7%
Pier 3	0.963	0.988	2.6%
Span 4-0.6L	0.967	0.989	2.3%

Ratios of Governing Force Effects: Case 3 to Case 2			
Service I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	0.953	0.984	3.3%
Left of Pier 1	0.970	0.990	2.1%
Right of Pier 1	0.971	0.990	2.0%
Left of Pier 2	0.980	0.993	1.3%
Right of Pier 2	0.987	0.996	0.9%
Left of Pier 3	0.981	0.994	1.3%
Right of Pier 3	0.979	0.993	1.4%
Left of Abut 2	0.963	0.988	2.6%

Ratios of Governing Force Effects: Case 3 to Case 2			
Strength I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	0.967	0.989	2.3%
Left of Pier 1	0.979	0.993	1.4%
Right of Pier 1	0.979	0.993	1.4%
Left of Pier 2	0.985	0.995	1.0%
Right of Pier 2	0.991	0.997	0.6%
Left of Pier 3	0.986	0.995	0.9%
Right of Pier 3	0.985	0.995	1.0%
Left of Abut 2	0.975	0.992	1.7%

Ratios of Governing Force Effects: Case 3 to Case 4			
Service I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	1.033	1.098	6.3%
Pier 1	1.032	1.096	6.2%
Span 2-0.5L	1.020	1.061	4.0%
Pier 2	1.012	1.036	2.4%
Span 3-0.5L	1.012	1.037	2.5%
Pier 3	1.018	1.055	3.6%
Span 4-0.6L	1.018	1.055	3.6%

Ratios of Governing Force Effects: Case 3 to Case 4			
Strength I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	1.022	1.066	4.3%
Pier 1	1.023	1.069	4.5%
Span 2-0.5L	1.014	1.041	2.7%
Pier 2	1.009	1.026	1.7%
Span 3-0.5L	1.009	1.026	1.7%
Pier 3	1.013	1.039	2.6%
Span 4-0.6L	1.012	1.035	2.3%

Ratios of Governing Force Effects: Case 3 to Case 4			
Service I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	1.017	1.051	3.3%
Left of Pier 1	1.010	1.031	2.1%
Right of Pier 1	1.010	1.030	2.0%
Left of Pier 2	1.007	1.021	1.4%
Right of Pier 2	1.004	1.013	0.9%
Left of Pier 3	1.007	1.020	1.3%
Right of Pier 3	1.007	1.022	1.5%
Left of Abut 2	1.013	1.039	2.6%

Ratios of Governing Force Effects: Case 3 to Case 4			
Strength I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	1.011	1.034	2.3%
Left of Pier 1	1.007	1.022	1.5%
Right of Pier 1	1.007	1.021	1.4%
Left of Pier 2	1.005	1.015	1.0%
Right of Pier 2	1.003	1.009	0.6%
Left of Pier 3	1.005	1.014	0.9%
Right of Pier 3	1.005	1.016	1.1%
Left of Abut 2	1.009	1.026	1.7%

Figure E-1. Summary of Ratios Based on Example E2(1) and Example E2(2)

5. Figure E-2 shows a summary of the results from Example E3(1) and Example E3(2). These examples are based on the same four-span bridge as in corresponding Example E2(1) and Example E2(2) discussed above except that approximately 75 percent smaller settlements ranging up to 1.20 in. were applied at the supports of the four-span bridge. Similar to the comparison made above, in Example E3(1), γ_{SE} = 1.25 was used while in Example E3(2) γ_{SE} =1.75 was used. Thus, a direct comparison of the effect of change in value of γ_{SE} can be performed for the case of smaller settlements and between these examples and those [Examples E2(1) and E2(2)] discussed above. In this comparison also, the increase in load factor is 40 percent (1.25 to 1.75). On Figure E-2, the results are tabulated in terms of ratios of Case 3 to Case 2 and Case 3 to Case 4 for Service I moments, Strength I moments, Service I shears, and Strength I shears. A description of the ratios is included in Table E-8.

   Based on ratio of Case 3 to Case 2, a 40 percent increase in γ_{SE} (that is, 1.25 to 1.75) results in the following:

   - 0.6 to 1.7 percent increase in moments for Service I load combination
   - 0.5 to 1.2 percent increase in moments for Strength I load combination
   - 0.2 to 0.9 percent increase in shears for Service I load combination
   - to 0.6 percent increase in shears for Strength I load combination

   Based on a ratio of Case 3 to Case 4, a 40 percent increase in γ_{SE} (that is, 1.25 to 1.75) results in the following:

   - 0.6 to 1.8 percent increase in moments for Service I load combination
   - 0.5 to 1.2 percent increase in moments for Strength I load combination
   - 0.2 to 0.9 percent increase in shears for Service I load combination
   - 0.1 to 0.6 percent increase in shears for Strength I load combination

   The ranges of increases for Case 3 to Case 2 and Case 3 to Case 4 are similar. For the four-span bridge example, the controlling force effects for moments and shears did not change appreciably when the proposed provisions are implemented. Comparable effects were noted in the other two examples (two-span and five-span bridges).

6. Based on the datasets on Figure E-1 and Figure E-2 and related discussions above, it is clear that the increase in moments and shears is much smaller for smaller settlements (Figure E-2) compared to that for larger settlements (Figure E-1). Designers typically limit the settlements to less than approximately 1.0 in., and the values on Figure E-2 that are based on similar smaller settlements indicate that the increase in moments and shears due to the proposed specifications is expected to be negligible.

Ratios of Governing Force Effects: Case 3 to Case 2			
Service I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	0.974	0.991	1.7%
Pier 1	0.974	0.991	1.7%
Span 2-0.5L	0.984	0.995	1.1%
Pier 2	0.991	0.997	0.6%
Span 3-0.5L	0.990	0.997	0.7%
Pier 3	0.986	0.995	0.9%
Span 4-0.6L	0.986	0.995	0.9%

Ratios of Governing Force Effects: Case 3 to Case 4			
Service I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	1.009	1.027	1.8%
Pier 1	1.009	1.027	1.8%
Span 2-0.5L	1.005	1.016	1.1%
Pier 2	1.003	1.009	0.6%
Span 3-0.5L	1.003	1.010	0.7%
Pier 3	1.005	1.014	0.9%
Span 4-0.6L	1.005	1.014	0.9%

Ratios of Governing Force Effects: Case 3 to Case 2			
Strength I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	0.983	0.994	1.1%
Pier 1	0.982	0.994	1.2%
Span 2-0.5L	0.989	0.996	0.7%
Pier 2	0.993	0.998	0.5%
Span 3-0.5L	0.993	0.998	0.5%
Pier 3	0.990	0.997	0.7%
Span 4-0.6L	0.991	0.997	0.6%

Ratios of Governing Force Effects: Case 3 to Case 4			
Strength I - Moments			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Span 1-0.4L	1.006	1.018	1.2%
Pier 1	1.006	1.019	1.3%
Span 2-0.5L	1.004	1.011	0.7%
Pier 2	1.002	1.007	0.5%
Span 3-0.5L	1.002	1.007	0.5%
Pier 3	1.003	1.010	0.7%
Span 4-0.6L	1.003	1.009	0.6%

Ratios of Governing Force Effects: Case 3 to Case 2			
Service I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	0.987	0.996	0.9%
Left of Pier 1	0.992	0.997	0.5%
Right of Pier 1	0.992	0.997	0.5%
Left of Pier 2	0.995	0.998	0.3%
Right of Pier 2	0.997	0.999	0.2%
Left of Pier 3	0.995	0.998	0.3%
Right of Pier 3	0.994	0.998	0.4%
Left of Abut 2	0.990	0.997	0.7%

Ratios of Governing Force Effects: Case 3 to Case 4			
Service I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	1.004	1.013	0.9%
Left of Pier 1	1.003	1.008	0.5%
Right of Pier 1	1.003	1.008	0.5%
Left of Pier 2	1.002	1.005	0.3%
Right of Pier 2	1.001	1.003	0.2%
Left of Pier 3	1.002	1.005	0.3%
Right of Pier 3	1.002	1.006	0.4%
Left of Abut 2	1.003	1.010	0.7%

Ratios of Governing Force Effects: Case 3 to Case 2			
Strength I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	0.991	0.997	0.6%
Left of Pier 1	0.994	0.998	0.4%
Right of Pier 1	0.995	0.998	0.3%
Left of Pier 2	0.996	0.999	0.3%
Right of Pier 2	0.998	0.999	0.1%
Left of Pier 3	0.996	0.999	0.3%
Right of Pier 3	0.996	0.999	0.3%
Left of Abut 2	0.993	0.998	0.5%

Ratios of Governing Force Effects: Case 3 to Case 4			
Strength I - Shears			
Location	Load Factor, γ_{SE}		Difference
	1.25	1.75	
Right of Abut 1	1.003	1.009	0.6%
Left of Pier 1	1.002	1.006	0.4%
Right of Pier 1	1.002	1.005	0.3%
Left of Pier 2	1.001	1.004	0.3%
Right of Pier 2	1.001	1.002	0.1%
Left of Pier 3	1.001	1.004	0.3%
Right of Pier 3	1.001	1.004	0.3%
Left of Abut 2	1.002	1.007	0.5%

Figure E-2. Summary of Ratios Based on Example E3(1) and Example E3(2)

E.3 Summary

Following are key observations based on the examples and the parametric study:

- Use of γ_{SE} and construction-point concept results in much less effects on controlling total moments and shears than would be indicated by the value of γ_{SE}.
- Even if the value of γ_{SE} changes from 1.25 to 1.75, a 40% increase, the difference in the controlling force effects is less than approximately 6%.
- The changes in the controlling force effects reduce as the settlements decrease and vice versa.

These key observations are as expected because (1) the *SE* load factor, γ_{SE}, is just one of the many load factors in the Service and Strength limit state load combinations within the overall *AASHTO LRFD* framework, (2) the additional (induced) force effects due to settlement are much smaller than the primary force effects due to dead load and live load, and (3) the induced force effects are directly proportional to the differential settlement.

The following 32 pages include the tables for structural computations for the eight examples discussed in this appendix (four pages per example).

Example E1(1)
Two-Span Bridge, Span Lengths 50 ft and 50 ft, Girder Spacing 7 ft-2 in.

Moment Comparison

Table E1(1)-M1		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Unfactored DL moment (No Settlement)		256	-453	256
Unfactored LL moment	+ve	486	0	486
	-ve	-116	-370	-116
Unfactored effect of 1 in. settlement at Abutment 1		-111	-277	-111
Unfactored effect of 1 in. settlement at Pier 1		222	555	222
Unfactored effect of 1 in. settlement at Abutment 2		-111	-277	-111

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E1(1)-M2

Predicted Unfactored Total Settlements, S_t (in.)		
Abutment 1	Pier 1	Abutment 2
0.80	1.60	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E1(1)-M3

Estimated Unfactored Relevant Settlements, S_{tr} (in.)		
Abutment 1	Pier 1	Abutment 2
0.40	0.80	0.30

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.25

Table E1(1)-M4

Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25		
Abutment 1	Pier 1	Abutment 2
0.50	1.00	0.38

Table E1(1)-M5		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Unfactored DL moment (No Settlement)		256	-453	256
Unfactored LL moment	+ve	486	0	486
	-ve	-116	-370	-116
Effect of unfactored S_{tr} at Abutment 1		-44	-111	-44
Effect of unfactored S_{tr} at Pier 1		178	444	178
Effect of unfactored S_{tr} at Abutment 2		-33	-83	-33
Total unfactored effect of S_{tr} at all supports	+ve	*178*	*444*	*178*
	-ve	*-78*	*-194*	*-78*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*355*	*888*	*355*
	-ve	*-155*	*-388*	*-155*
Total factored effect of sett using g_{SE} = 1.25 and S_{tr}	+ve	*222*	*555*	*222*
	-ve	*-97*	*-242*	*-97*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*178*	*444*	*178*
	-ve	*-78*	*-194*	*-78*

Table E1(1)-M6 Service I Comparison		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	742	-453	742
	Min	140	-823	140
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	1097	435	1097
	Min	-15	-1211	-15
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	964	102	964
	Min	43	-1065	43
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	920	-9	920
	Min	62	-1017	62
Ratio of Case 3 to Case 1	Max	1.299	-0.225	1.299
	Min	0.306	1.295	0.306
Ratio of Case 3 to Case 2	Max	*0.879*	*0.234*	*0.879*
	Min	*-2.784*	*0.880*	*-2.784*
Ratio of Case 2 to Case 1	Max	1.479	-0.960	1.479
	Min	-0.110	1.471	-0.110
Ratio of Case 3 to Case 4	Max	1.048	-11.333	1.048
	Min	0.688	1.048	0.688
Ratio of Case 4 to Case 2	Max	0.838	-0.021	0.838
	Min	-4.045	0.840	-4.045

Table E1(1)-M7 Strength I Comparison		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	1171	-566	1171
	Min	117	-1214	117
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	1526	322	1526
	Min	-38	-1602	-38
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	1393	-11	1393
	Min	20	-1456	20
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	1348	-122	1348
	Min	39	-1408	39
Ratio of Case 3 to Case 1	Max	1.190	0.020	1.190
	Min	0.170	1.200	0.170
Ratio of Case 3 to Case 2	Max	*0.913*	-0.035	*0.913*
	Min	-0.518	*0.909*	-0.518
Ratio of Case 2 to Case 1	Max	1.303	-0.568	1.303
	Min	-0.328	1.320	-0.328
Ratio of Case 3 to Case 4	Max	1.033	0.092	1.033
	Min	0.506	1.034	0.506
Ratio of Case 4 to Case 2	Max	0.884	-0.380	0.884
	Min	-1.023	0.879	-1.023

Example E1(1)
Two-Span Bridge, Span Lengths 50 ft and 50 ft, Girder Spacing 7 ft-2 in.

Shear Comparison

Table E1(1)-S1		Shear (kip)			
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Unfactored DL shear (No settlement)		27.4	-45.5	45.5	-27.4
Unfactored LL shear	+ve	67.4	0.0	79.1	7.5
	-ve	-7.5	-79.1	0.0	-67.4
Unfactored effect of 1 in. settlement at Abutment 1		-5.5	-5.5	5.5	5.5
Unfactored effect of 1 in. settlement at Pier 1		11.1	11.1	-11.1	-11.1
Unfactored effect of 1 in. settlement at Abutment 2		-5.5	-5.5	5.5	5.5

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E1(1)-S2		
Predicted Unfactored Total Settlements, S_t (in.)		
Abutment 1	Pier 1	Abutment 2
0.80	1.60	0.60

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E1(1)-S3		
Estimated Unfactored Relevant Settlements, S_{tr} (in.)		
Abutment 1	Pier 1	Abutment 2
0.40	0.80	0.30

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.25

Table E1(1)-S4		
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25		
Abutment 1	Pier 1	Abutment 2
0.50	1.00	0.38

Table E1(1)-S5		Shear (kip)			
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Unfactored DL shear (No settlement)		27.4	-45.5	45.5	-27.4
Unfactored LL shear	+ve	67.4	0.0	79.1	7.5
	-ve	-7.5	-79.1	0.0	-67.4
Effect of unfactored S_{tr} at Abutment 1		-2.2	-2.2	2.2	2.2
Effect of unfactored S_{tr} at Pier 1		8.9	8.9	-8.9	-8.9
Effect of unfactored S_{tr} at Abutment 2		-1.7	-1.7	1.7	1.7
Total unfactored effect of S_{tr} at all supports	+ve	*9*	*9*	*4*	*4*
	-ve	*-4*	*-4*	*-9*	*-9*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*18*	*18*	*8*	*8*
	-ve	*-8*	*-8*	*-18*	*-18*
Total factored effect of settlement using g_{SE} = 1.25 and S_{tr}	+ve	*11*	*11*	*5*	*5*
	-ve	*-5*	*-5*	*-11*	*-11*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*9*	*9*	*4*	*4*
	-ve	*-4*	*-4*	*-9*	*-9*

Table E1(1)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	95	-46	125	-20
	Min	20	-125	46	-95
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	113	-28	132	-12
	Min	12	-132	28	-113
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	106	-34	129	-15
	Min	15	-129	34	-106
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	104	-37	129	-16
	Min	16	-129	37	-104
Ratio of Case 3 to Case 1	Max	1.117	0.756	1.039	0.757
	Min	0.757	1.039	0.756	1.117
Ratio of Case 3 to Case 2	Max	0.941	1.240	0.978	1.239
	Min	1.239	0.978	1.240	0.941
Ratio of Case 2 to Case 1	Max	1.187	0.610	1.062	0.611
	Min	0.611	1.062	0.610	1.187
Ratio of Case 3 to Case 4	Max	1.021	0.939	1.008	0.940
	Min	0.940	1.008	0.939	1.021
Ratio of Case 4 to Case 2	Max	0.921	1.320	0.971	1.319
	Min	1.319	0.971	1.320	0.921

Table E1(1)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	152	-57	195	-21
	Min	21	-195	57	-152
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	170	-39	203	-13
	Min	13	-203	39	-170
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	163	-46	200	-16
	Min	16	-200	46	-163
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	161	-48	199	-17
	Min	17	-199	48	-161
Ratio of Case 3 to Case 1	Max	1.073	0.805	1.025	0.771
	Min	0.771	1.025	0.805	1.073
Ratio of Case 3 to Case 2	Max	0.961	1.170	0.986	1.217
	Min	1.217	0.986	1.170	0.961
Ratio of Case 2 to Case 1	Max	1.117	0.688	1.040	0.634
	Min	0.634	1.040	0.688	1.117
Ratio of Case 3 to Case 4	Max	1.014	0.954	1.005	0.944
	Min	0.944	1.005	0.954	1.014
Ratio of Case 4 to Case 2	Max	0.948	1.227	0.981	1.289
	Min	1.289	0.981	1.227	0.948

Example E1(2)
Two-Span Bridge, Span Lengths 50 ft and 50 ft, Girder Spacing 7 ft-2 in.

Moment Comparison

Table E1(2)-M1		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Unfactored DL moment (No Settlement)		256	-453	256
Unfactored LL moment	+ve	486	0	486
	-ve	-116	-370	-116
Unfactored effect of 1 in. settlement at Abutment 1		-111	-277	-111
Unfactored effect of 1 in. settlement at Pier 1		222	555	222
Unfactored effect of 1 in. settlement at Abutment 2		-111	-277	-111

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E1(2)-M2		
Predicted Unfactored Total Settlements, S_t (in.)		
Abutment 1	Pier 1	Abutment 2
0.80	1.60	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E1(2)-M3		
Estimated Unfactored Relevant Settlements, S_{tr} (in.)		
Abutment 1	Pier 1	Abutment 2
0.40	0.80	0.30

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.75

Table E1(2)-M4		
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75		
Abutment 1	Pier 1	Abutment 2
0.70	1.40	0.53

Table E1(2)-M5		Moment (kip-ft)		
		Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Unfactored DL moment (No Settlement)		256	-453	256
Unfactored LL moment	+ve	486	0	486
	-ve	-116	-370	-116
Effect of unfactored S_{tr} at Abutment 1		-44	-111	-44
Effect of unfactored S_{tr} at Pier 1		178	444	178
Effect of unfactored S_{tr} at Abutment 2		-33	-83	-33
Total unfactored effect of S_{tr} at all supports	+ve	*178*	*444*	*178*
	-ve	*-78*	*-194*	*-78*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*355*	*888*	*355*
	-ve	*-155*	*-388*	*-155*
Total factored effect of sett using g_{SE} = 1.75 and S_{tr}	+ve	*311*	*777*	*311*
	-ve	*-136*	*-339*	*-136*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*178*	*444*	*178*
	-ve	*-78*	*-194*	*-78*

Table E1(2)-M6 Service I Comparison		Moment (kip-ft) Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	742	-453	742
	Min	140	-823	140
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	1097	435	1097
	Min	-15	-1211	-15
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	1053	324	1053
	Min	4	-1162	4
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	920	-9	920
	Min	62	-1017	62
Ratio of Case 3 to Case 1	Max	1.419	-0.715	1.419
	Min	0.029	1.412	0.029
Ratio of Case 3 to Case 2	Max	*0.960*	*0.745*	*0.960*
	Min	*-0.261*	*0.960*	*-0.261*
Ratio of Case 2 to Case 1	Max	1.479	-0.960	1.479
	Min	-0.110	1.471	-0.110
Ratio of Case 3 to Case 4	Max	1.145	-36.000	1.145
	Min	0.065	1.143	0.065
Ratio of Case 4 to Case 2	Max	0.838	-0.021	0.838
	Min	-4.045	0.840	-4.045

Table E1(2)-M7 Strength I Comparison		Moment (kip-ft) Span 1 - 0.4L	Pier 1	Span 2 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	1171	-566	1171
	Min	117	-1214	117
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	1526	322	1526
	Min	-38	-1602	-38
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	1481	211	1481
	Min	-19	-1553	-19
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	1348	-122	1348
	Min	39	-1408	39
Ratio of Case 3 to Case 1	Max	1.266	-0.372	1.266
	Min	-0.162	1.280	-0.162
Ratio of Case 3 to Case 2	Max	*0.971*	0.655	*0.971*
	Min	0.494	*0.970*	0.494
Ratio of Case 2 to Case 1	Max	1.303	-0.568	1.303
	Min	-0.328	1.320	-0.328
Ratio of Case 3 to Case 4	Max	1.099	-1.724	1.099
	Min	-0.483	1.103	-0.483
Ratio of Case 4 to Case 2	Max	0.884	-0.380	0.884
	Min	-1.023	0.879	-1.023

Example E1(2)

Two-Span Bridge, Span Lengths 50 ft and 50 ft, Girder Spacing 7 ft-2 in.

Shear Comparison

Table E1(2)-S1		Shear (kip)			
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Unfactored DL shear (No settlement)		27.4	-45.5	45.5	-27.4
Unfactored LL shear	+ve	67.4	0.0	79.1	7.5
	-ve	-7.5	-79.1	0.0	-67.4
Unfactored effect of 1 in. settlement at Abutment 1		-5.5	-5.5	5.5	5.5
Unfactored effect of 1 in. settlement at Pier 1		11.1	11.1	-11.1	-11.1
Unfactored effect of 1 in. settlement at Abutment 2		-5.5	-5.5	5.5	5.5

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E1(2)-S2		
Predicted Unfactored Total Settlements, S_t (in.)		
Abutment 1	Pier 1	Abutment 2
0.80	1.60	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E1(2)-S3		
Estimated Unfactored Relevant Settlements, S_{tr} (in.)		
Abutment 1	Pier 1	Abutment 2
0.40	0.80	0.30

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.75

Table E1(2)-S4		
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75		
Abutment 1	Pier 1	Abutment 2
0.70	1.40	0.53

Table E1(2)-S5		Shear (kip)			
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Abutment 2
Unfactored DL shear (No settlement)		27.4	-45.5	45.5	-27.4
Unfactored LL shear	+ve	67.4	0.0	79.1	7.5
	-ve	-7.5	-79.1	0.0	-67.4
Effect of unfactored S_{tr} at Abutment 1		-2.2	-2.2	2.2	2.2
Effect of unfactored S_{tr} at Pier 1		8.9	8.9	-8.9	-8.9
Effect of unfactored S_{tr} at Abutment 2		-1.7	-1.7	1.7	1.7
Total unfactored effect of S_{tr} at all supports	+ve	*9*	*9*	*4*	*4*
	-ve	*-4*	*-4*	*-9*	*-9*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*18*	*18*	*8*	*8*
	-ve	*-8*	*-8*	*-18*	*-18*
Total factored effect of settlement using g_{SE} = 1.75 and S_{tr}	+ve	*16*	*16*	*7*	*7*
	-ve	*-7*	*-7*	*-16*	*-16*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*9*	*9*	*4*	*4*
	-ve	*-4*	*-4*	*-9*	*-9*

Table E1(2)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Shear (kip) Left of Pier 1	Shear (kip) Right of Pier 1	Shear (kip) Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	95	-46	125	-20
	Min	20	-125	46	-95
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	113	-28	132	-12
	Min	12	-132	28	-113
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	110	-30	131	-13
	Min	13	-131	30	-110
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	104	-37	129	-16
	Min	16	-129	37	-104
Ratio of Case 3 to Case 1	Max	1.164	0.659	1.054	0.659
	Min	0.659	1.054	0.659	1.164
Ratio of Case 3 to Case 2	Max	0.980	1.080	0.993	1.080
	Min	1.080	0.993	1.080	0.980
Ratio of Case 2 to Case 1	Max	1.187	0.610	1.062	0.611
	Min	0.611	1.062	0.610	1.187
Ratio of Case 3 to Case 4	Max	1.064	0.818	1.023	0.819
	Min	0.819	1.023	0.818	1.064
Ratio of Case 4 to Case 2	Max	0.921	1.320	0.971	1.319
	Min	1.319	0.971	1.320	0.921

Table E1(2)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Shear (kip) Left of Pier 1	Shear (kip) Right of Pier 1	Shear (kip) Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	152	-57	195	-21
	Min	21	-195	57	-152
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	170	-39	203	-13
	Min	13	-203	39	-170
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	168	-41	202	-14
	Min	14	-202	41	-168
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	161	-48	199	-17
	Min	17	-199	48	-161
Ratio of Case 3 to Case 1	Max	1.102	0.727	1.035	0.680
	Min	0.680	1.035	0.727	1.102
Ratio of Case 3 to Case 2	Max	0.987	1.057	0.995	1.072
	Min	1.072	0.995	1.057	0.987
Ratio of Case 2 to Case 1	Max	1.117	0.688	1.040	0.634
	Min	0.634	1.040	0.688	1.117
Ratio of Case 3 to Case 4	Max	1.041	0.861	1.015	0.832
	Min	0.832	1.015	0.861	1.041
Ratio of Case 4 to Case 2	Max	0.948	1.227	0.981	1.289
	Min	1.289	0.981	1.227	0.948

Example E2(1)

Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Moment Comparison

Table E2(1)-M1		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Unfactored effect of 1 in. settlement at Abutment 1		-329	-822	-273	278	84	-110	-22
Unfactored effect of 1 in. settlement at Pier 1		702	1753	609	-534	-161	212	43
Unfactored effect of 1 in. settlement at Pier 2		-469	-1174	-79	1016	344	-328	-65
Unfactored effect of 1 in. settlement at Pier 3		192	452	-479	-1409	321	2050	411
Unfactored effect of 1 in. settlement at Abutment 2		-82	-208	221	651	-587	-1825	-364

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E2(1)-M2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.90	3.90	4.80	1.90	2.50

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E2(1)-M3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.95	1.95	2.40	0.95	1.25

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.25

Table E2(1)-M4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.19	2.44	3.00	1.19	1.56

Table E2(1)-M5		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Effect of unfactored S_{tr} at Abutment 1		-313	-781	-259	264	80	-105	-21
Effect of unfactored S_{tr} at Pier 1		1369	3418	1188	-1041	-314	413	84
Effect of unfactored S_{tr} at Pier 2		-1126	-2818	-190	2438	826	-787	-156
Effect of unfactored S_{tr} at Pier 3		182	429	-455	-1339	305	1948	390
Effect of unfactored S_{tr} at Abutment 2		-103	-260	276	814	-734	-2281	-455
Total unfactored effect of S_{tr} at all supports	+ve	*1551*	*3848*	*1464*	*3516*	*1210*	*2361*	*474*
	-ve	*-1541*	*-3859*	*-904*	*-2380*	*-1048*	*-3173*	*-632*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*3103*	*7696*	*2928*	*7033*	*2421*	*4722*	*949*
	-ve	*-3081*	*-7717*	*-1808*	*-4760*	*-2095*	*-6346*	*-1264*
Total factored effect of sett using g_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*1551*	*3848*	*1464*	*3516*	*1210*	*2361*	*474*
	-ve	*-1541*	*-3859*	*-904*	*-2380*	*-1048*	*-3173*	*-632*

Table E2(1)-M6 Service I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	10285	-12754	16640	-32725	23254	-23162	6030
	Min	713	-26170	4827	-47099	11256	-40406	-619
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	13388	-5059	19568	-25693	25675	-18440	6979
	Min	-2368	-33887	3019	-51859	9161	-46752	-1883
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	12224	-7944	18470	-28330	24767	-20211	6623
	Min	-1213	-30993	3697	-50074	9946	-44372	-1409
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	11836	-8906	18104	-29209	24464	-20801	6504
	Min	-828	-30029	3923	-49479	10208	-43579	-1251
Ratio of Case 3 to Case 1	Max	1.189	0.623	1.110	0.866	1.065	0.873	1.098
	Min	-1.701	1.184	0.766	1.063	0.884	1.098	2.276
Ratio of Case 3 to Case 2	Max	*0.913*	*1.570*	*0.944*	*1.103*	*0.965*	*1.096*	*0.949*
	Min	*0.512*	*0.915*	*1.225*	*0.966*	*1.086*	*0.949*	*0.748*
Ratio of Case 2 to Case 1	Max	1.302	0.397	1.176	0.785	1.104	0.796	1.157
	Min	-3.322	1.295	0.625	1.101	0.814	1.157	3.042
Ratio of Case 3 to Case 4	Max	1.033	0.892	1.020	0.970	1.012	0.972	1.018
	Min	1.465	1.032	0.942	1.012	0.974	1.018	1.126
Ratio of Case 4 to Case 2	Max	0.884	1.761	0.925	1.137	0.953	1.128	0.932
	Min	0.349	0.886	1.299	0.954	1.114	0.932	0.664

Table E2(1)-M7 Strength I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	16057	-14539	25120	-40323	33938	-27622	9727
	Min	-694	-38017	4447	-65478	12942	-57799	-1909
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	19159	-6844	28047	-33291	36359	-22900	10676
	Min	-3776	-45734	2639	-70237	10846	-64144	-3173
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	17996	-9729	26949	-35928	35451	-24670	10320
	Min	-2620	-42840	3317	-68453	11632	-61765	-2699
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	17608	-10691	26583	-36807	35148	-25261	10201
	Min	-2235	-41876	3543	-67858	11894	-60971	-2541
Ratio of Case 3 to Case 1	Max	1.121	0.669	1.073	0.891	1.045	0.893	1.061
	Min	3.774	1.127	0.746	1.045	0.899	1.069	1.414
Ratio of Case 3 to Case 2	Max	*0.939*	1.422	*0.961*	1.079	*0.975*	1.077	*0.967*
	Min	0.694	*0.937*	1.257	*0.975*	1.072	*0.963*	0.851
Ratio of Case 2 to Case 1	Max	1.193	0.471	1.117	0.826	1.071	0.829	1.098
	Min	5.438	1.203	0.593	1.073	0.838	1.110	1.662
Ratio of Case 3 to Case 4	Max	1.022	0.910	1.014	0.976	1.009	0.977	1.012
	Min	1.172	1.023	0.936	1.009	0.978	1.013	1.062
Ratio of Case 4 to Case 2	Max	0.919	1.562	0.948	1.106	0.967	1.103	0.956
	Min	0.592	0.916	1.343	0.966	1.097	0.951	0.801

Example E2(1)
Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Shear Comparison

Table E2(1)-S1		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Unfactored effect of 1 in. settlement at Abutment 1		-4.9	-4.9	3.8	3.8	-1.2	-1.2	0.7	0.7
Unfactored effect of 1 in. settlement at Pier 1		10.4	10.4	-7.8	-7.8	2.2	2.2	-1.3	-1.3
Unfactored effect of 1 in. settlement at Pier 2		-7.0	-7.0	7.5	7.5	-4.0	-4.0	2.0	2.0
Unfactored effect of 1 in. settlement at Pier 3		2.7	2.7	-6.4	-6.4	10.3	10.3	-12.4	-12.4
Unfactored effect of 1 in. settlement at Abutment 2		-1.2	-1.2	2.9	2.9	-7.4	-7.4	11.1	11.1

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E2(1)-S2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.90	3.90	4.80	1.90	2.50

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E2(1)-S3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.95	1.95	2.40	0.95	1.25

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.25

Table E2(1)-S4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.19	2.44	3.00	1.19	1.56

Table E2(1)-S5		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Effect of unfactored S_{tr} at Abutment 1		-4.7	-4.7	3.6	3.6	-1.1	-1.1	0.6	0.6
Effect of unfactored S_{tr} at Pier 1		20.4	20.4	-15.2	-15.2	4.3	4.3	-2.5	-2.5
Effect of unfactored S_{tr} at Pier 2		-16.8	-16.8	17.9	17.9	-9.6	-9.6	4.8	4.8
Effect of unfactored S_{tr} at Pier 3		2.5	2.5	-6.0	-6.0	9.8	9.8	-11.8	-11.8
Effect of unfactored S_{tr} at Abutment 2		-1.6	-1.6	3.7	3.7	-9.2	-9.2	13.8	13.8
Total unfactored effect of S_{tr} *at all supports*	+ve	*23*	*23*	*25*	*25*	*14*	*14*	*19*	*19*
	-ve	*-23*	*-23*	*-21*	*-21*	*-20*	*-20*	*-14*	*-14*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*46*	*46*	*50*	*50*	*28*	*28*	*38*	*38*
	-ve	*-46*	*-46*	*-43*	*-43*	*-40*	*-40*	*-29*	*-29*
Total factored effect of settlement using g_{SE} = 1.25 and S_{tr}	+ve	*29*	*29*	*31*	*31*	*18*	*18*	*24*	*24*
	-ve	*-29*	*-29*	*-27*	*-27*	*-25*	*-25*	*-18*	*-18*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*23*	*23*	*25*	*25*	*14*	*14*	*19*	*19*
	-ve	*-23*	*-23*	*-21*	*-21*	*-20*	*-20*	*-14*	*-14*

Table E2(1)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	317	-330	598	-514	796	-476	632	-38
	Min	114	-537	348	-751	554	-732	414	-259
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	362	-284	648	-463	825	-448	671	1
	Min	68	-583	305	-794	514	-772	385	-288
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	345	-302	629	-482	814	-459	656	-14
	Min	85	-566	321	-778	529	-757	396	-277
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	339	-307	623	-489	810	-462	651	-18
	Min	91	-560	326	-772	534	-752	399	-274
Ratio of Case 3 to Case 1	Max	1.090	0.913	1.053	0.939	1.022	0.963	1.038	0.362
	Min	0.747	1.053	0.924	1.035	0.955	1.034	0.957	1.069
Ratio of Case 3 to Case 2	Max	0.953	1.060	0.971	1.041	0.987	1.024	0.979	-17.149
	Min	1.254	0.970	1.052	0.980	1.029	0.981	1.028	0.963
Ratio of Case 2 to Case 1	Max	1.145	0.861	1.084	0.902	1.036	0.941	1.061	-0.021
	Min	0.596	1.086	0.878	1.057	0.928	1.055	0.931	1.110
Ratio of Case 3 to Case 4	Max	1.017	0.981	1.010	0.987	1.004	0.992	1.007	0.739
	Min	0.937	1.010	0.984	1.007	0.991	1.007	0.991	1.013
Ratio of Case 4 to Case 2	Max	0.937	1.081	0.961	1.054	0.983	1.032	0.971	-23.198
	Min	1.339	0.961	1.070	0.973	1.039	0.974	1.037	0.950

Table E2(1)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	475	-405	854	-636	1112	-583	892	-16
	Min	120	-767	416	-1051	688	-1030	510	-404
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	521	-359	905	-585	1140	-554	931	23
	Min	74	-813	374	-1093	648	-1070	481	-432
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	504	-376	886	-604	1129	-565	916	8
	Min	92	-796	390	-1078	663	-1055	492	-422
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	498	-382	880	-610	1126	-568	911	4
	Min	97	-790	395	-1072	668	-1050	495	-418
Ratio of Case 3 to Case 1	Max	1.060	0.929	1.037	0.951	1.016	0.970	1.027	-0.544
	Min	0.761	1.037	0.936	1.025	0.964	1.024	0.965	1.044
Ratio of Case 3 to Case 2	Max	0.967	1.048	0.979	1.032	0.991	1.019	0.985	0.370
	Min	1.231	0.979	1.043	0.985	1.023	0.986	1.022	0.975
Ratio of Case 2 to Case 1	Max	1.096	0.887	1.059	0.921	1.025	0.951	1.043	-1.471
	Min	0.618	1.060	0.898	1.040	0.942	1.039	0.944	1.071
Ratio of Case 3 to Case 4	Max	1.011	0.985	1.007	0.990	1.003	0.994	1.005	2.311
	Min	0.941	1.007	0.987	1.005	0.993	1.005	0.993	1.009
Ratio of Case 4 to Case 2	Max	0.956	1.064	0.972	1.043	0.988	1.025	0.979	0.160
	Min	1.309	0.972	1.057	0.981	1.031	0.981	1.030	0.967

Example E2(2)

Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Moment Comparison

Table E2(2)-M1		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Unfactored effect of 1 in. settlement at Abutment 1		-329	-822	-273	278	84	-110	-22
Unfactored effect of 1 in. settlement at Pier 1		702	1753	609	-534	-161	212	43
Unfactored effect of 1 in. settlement at Pier 2		-469	-1174	-79	1016	344	-328	-65
Unfactored effect of 1 in. settlement at Pier 3		192	452	-479	-1409	321	2050	411
Unfactored effect of 1 in. settlement at Abutment 2		-82	-208	221	651	-587	-1825	-364

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E2(2)-M2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.90	3.90	4.80	1.90	2.50

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E2(2)-M3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.95	1.95	2.40	0.95	1.25

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.75

Table E2(2)-M4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.66	3.41	4.20	1.66	2.19

Table E2(2)-M5		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Effect of unfactored S_{tr} at Abutment 1		-313	-781	-259	264	80	-105	-21
Effect of unfactored S_{tr} at Pier 1		1369	3418	1188	-1041	-314	413	84
Effect of unfactored S_{tr} at Pier 2		-1126	-2818	-190	2438	826	-787	-156
Effect of unfactored S_{tr} at Pier 3		182	429	-455	-1339	305	1948	390
Effect of unfactored S_{tr} at Abutment 2		-103	-260	276	814	-734	-2281	-455
Total unfactored effect of S_{tr} at all supports	+ve	*1551*	*3848*	*1464*	*3516*	*1210*	*2361*	*474*
	-ve	*-1541*	*-3859*	*-904*	*-2380*	*-1048*	*-3173*	*-632*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*3103*	*7696*	*2928*	*7033*	*2421*	*4722*	*949*
	-ve	*-3081*	*-7717*	*-1808*	*-4760*	*-2095*	*-6346*	*-1264*
Total factored effect of sett using g_{SE} = 1.75 and S_{tr}	+ve	*2715*	*6734*	*2562*	*6153*	*2118*	*4132*	*830*
	-ve	*-2696*	*-6752*	*-1582*	*-4165*	*-1833*	*-5553*	*-1106*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*1551*	*3848*	*1464*	*3516*	*1210*	*2361*	*474*
	-ve	*-1541*	*-3859*	*-904*	*-2380*	*-1048*	*-3173*	*-632*

Table E2(2)-M6		Moment (kip-ft)						
Service I Comparison		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	10285	-12754	16640	-32725	23254	-23162	6030
	Min	713	-26170	4827	-47099	11256	-40406	-619
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	13388	-5059	19568	-25693	25675	-18440	6979
	Min	-2368	-33887	3019	-51859	9161	-46752	-1883
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	13000	-6020	19202	-26572	25372	-19030	6860
	Min	-1983	-32922	3245	-51264	9423	-45959	-1725
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	11836	-8906	18104	-29209	24464	-20801	6504
	Min	-828	-30029	3923	-49479	10208	-43579	-1251
Ratio of Case 3 to Case 1	Max	1.264	0.472	1.154	0.812	1.091	0.822	1.138
	Min	-2.781	1.258	0.672	1.088	0.837	1.137	2.786
Ratio of Case 3 to Case 2	Max	*0.971*	*1.190*	*0.981*	*1.034*	*0.988*	*1.032*	*0.983*
	Min	*0.837*	*0.972*	*1.075*	*0.989*	*1.029*	*0.983*	*0.916*
Ratio of Case 2 to Case 1	Max	1.302	0.397	1.176	0.785	1.104	0.796	1.157
	Min	-3.322	1.295	0.625	1.101	0.814	1.157	3.042
Ratio of Case 3 to Case 4	Max	1.098	0.676	1.061	0.910	1.037	0.915	1.055
	Min	2.396	1.096	0.827	1.036	0.923	1.055	1.379
Ratio of Case 4 to Case 2	Max	0.884	1.761	0.925	1.137	0.953	1.128	0.932
	Min	0.349	0.886	1.299	0.954	1.114	0.932	0.664

Table E2(2)-M7		Moment (kip-ft)						
Strength I Comparison		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	16057	-14539	25120	-40323	33938	-27622	9727
	Min	-694	-38017	4447	-65478	12942	-57799	-1909
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	19159	-6844	28047	-33291	36359	-22900	10676
	Min	-3776	-45734	2639	-70237	10846	-64144	-3173
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	18772	-7805	27681	-34170	36056	-23490	10557
	Min	-3390	-44769	2865	-69642	11108	-63351	-3015
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	17608	-10691	26583	-36807	35148	-25261	10201
	Min	-2235	-41876	3543	-67858	11894	-60971	-2541
Ratio of Case 3 to Case 1	Max	1.169	0.537	1.102	0.847	1.062	0.850	1.085
	Min	4.884	1.178	0.644	1.064	0.858	1.096	1.579
Ratio of Case 3 to Case 2	Max	*0.980*	1.141	*0.987*	1.026	*0.992*	1.026	*0.989*
	Min	0.898	*0.979*	1.086	*0.992*	1.024	*0.988*	0.950
Ratio of Case 2 to Case 1	Max	1.193	0.471	1.117	0.826	1.071	0.829	1.098
	Min	5.438	1.203	0.593	1.073	0.838	1.110	1.662
Ratio of Case 3 to Case 4	Max	1.066	0.730	1.041	0.928	1.026	0.930	1.035
	Min	1.517	1.069	0.809	1.026	0.934	1.039	1.187
Ratio of Case 4 to Case 2	Max	0.919	1.562	0.948	1.106	0.967	1.103	0.956
	Min	0.592	0.916	1.343	0.966	1.097	0.951	0.801

Example E2(2)
Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Shear Comparison

Table E2(2)-S1		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Unfactored effect of 1 in. settlement at Abutment 1		-4.9	-4.9	3.8	3.8	-1.2	-1.2	0.7	0.7
Unfactored effect of 1 in. settlement at Pier 1		10.4	10.4	-7.8	-7.8	2.2	2.2	-1.3	-1.3
Unfactored effect of 1 in. settlement at Pier 2		-7.0	-7.0	7.5	7.5	-4.0	-4.0	2.0	2.0
Unfactored effect of 1 in. settlement at Pier 3		2.7	2.7	-6.4	-6.4	10.3	10.3	-12.4	-12.4
Unfactored effect of 1 in. settlement at Abutment 2		-1.2	-1.2	2.9	2.9	-7.4	-7.4	11.1	11.1

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E2(2)-S2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.90	3.90	4.80	1.90	2.50

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E2(2)-S3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.95	1.95	2.40	0.95	1.25

Factored Relevant Settlements, S_f
This example is based on load factor γ_{SE} = 1.75

Table E2(2)-S4				
Factored Relevant Settlements, S_f (in.) using γ_{SE} = 1.75				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
1.66	3.41	4.20	1.66	2.19

Table E2(2)-S5		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Effect of unfactored S_{tr} at Abutment 1		-4.7	-4.7	3.6	3.6	-1.1	-1.1	0.6	0.6
Effect of unfactored S_{tr} at Pier 1		20.4	20.4	-15.2	-15.2	4.3	4.3	-2.5	-2.5
Effect of unfactored S_{tr} at Pier 2		-16.8	-16.8	17.9	17.9	-9.6	-9.6	4.8	4.8
Effect of unfactored S_{tr} at Pier 3		2.5	2.5	-6.0	-6.0	9.8	9.8	-11.8	-11.8
Effect of unfactored S_{tr} at Abutment 2		-1.6	-1.6	3.7	3.7	-9.2	-9.2	13.8	13.8
Total unfactored effect of S_{tr} at all supports	+ve	*23*	*23*	*25*	*25*	*14*	*14*	*19*	*19*
	-ve	*-23*	*-23*	*-21*	*-21*	*-20*	*-20*	*-14*	*-14*
Total factored effect of settlement using γ_{SE} = 1.00 and S_t	+ve	*46*	*46*	*50*	*50*	*28*	*28*	*38*	*38*
	-ve	*-46*	*-46*	*-43*	*-43*	*-40*	*-40*	*-29*	*-29*
Total factored effect of settlement using γ_{SE} = 1.75 and S_{tr}	+ve	*40*	*40*	*44*	*44*	*25*	*25*	*34*	*34*
	-ve	*-40*	*-40*	*-37*	*-37*	*-35*	*-35*	*-25*	*-25*
Total factored effect of settlement using γ_{SE} = 1.00 and S_{tr}	+ve	*23*	*23*	*25*	*25*	*14*	*14*	*19*	*19*
	-ve	*-23*	*-23*	*-21*	*-21*	*-20*	*-20*	*-14*	*-14*

Table E2(2)-S6 Service I Comparison		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	317	-330	598	-514	796	-476	632	-38
	Min	114	-537	348	-751	554	-732	414	-259
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	362	-284	648	-463	825	-448	671	1
	Min	68	-583	305	-794	514	-772	385	-288
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	357	-290	642	-470	821	-452	666	-4
	Min	73	-577	311	-788	519	-767	389	-284
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	339	-307	623	-489	810	-462	651	-18
	Min	91	-560	326	-772	534	-752	399	-274
Ratio of Case 3 to Case 1	Max	1.127	0.879	1.074	0.914	1.031	0.948	1.053	0.107
	Min	0.646	1.075	0.893	1.050	0.937	1.048	0.939	1.096
Ratio of Case 3 to Case 2	Max	0.984	1.020	0.990	1.014	0.996	1.008	0.993	-5.050
	Min	1.085	0.990	1.017	0.993	1.010	0.994	1.009	0.988
Ratio of Case 2 to Case 1	Max	1.145	0.861	1.084	0.902	1.036	0.941	1.061	-0.021
	Min	0.596	1.086	0.878	1.057	0.928	1.055	0.931	1.110
Ratio of Case 3 to Case 4	Max	1.051	0.944	1.030	0.961	1.013	0.977	1.022	0.218
	Min	0.810	1.031	0.951	1.021	0.972	1.020	0.973	1.039
Ratio of Case 4 to Case 2	Max	0.937	1.081	0.961	1.054	0.983	1.032	0.971	-23.198
	Min	1.339	0.961	1.070	0.973	1.039	0.974	1.037	0.950

Table E2(2)-S7 Strength I Comparison		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	475	-405	854	-636	1112	-583	892	-16
	Min	120	-767	416	-1051	688	-1030	510	-404
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	521	-359	905	-585	1140	-554	931	23
	Min	74	-813	374	-1093	648	-1070	481	-432
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	515	-365	898	-592	1136	-558	926	18
	Min	80	-808	379	-1088	653	-1065	485	-429
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	498	-382	880	-610	1126	-568	911	4
	Min	97	-790	395	-1072	668	-1050	495	-418
Ratio of Case 3 to Case 1	Max	1.084	0.901	1.052	0.931	1.022	0.958	1.038	-1.162
	Min	0.666	1.052	0.911	1.035	0.949	1.034	0.951	1.062
Ratio of Case 3 to Case 2	Max	0.989	1.016	0.993	1.011	0.997	1.006	0.995	0.790
	Min	1.077	0.993	1.014	0.995	1.008	0.995	1.007	0.992
Ratio of Case 2 to Case 1	Max	1.096	0.887	1.059	0.921	1.025	0.951	1.043	-1.471
	Min	0.618	1.060	0.898	1.040	0.942	1.039	0.944	1.071
Ratio of Case 3 to Case 4	Max	1.034	0.955	1.021	0.969	1.009	0.981	1.016	4.934
	Min	0.823	1.022	0.960	1.015	0.978	1.014	0.978	1.026
Ratio of Case 4 to Case 2	Max	0.956	1.064	0.972	1.043	0.988	1.025	0.979	0.160
	Min	1.309	0.972	1.057	0.981	1.031	0.981	1.030	0.967

Example E3(1)

Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Moment Comparison

Table E3(1)-M1		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Unfactored effect of 1 in. settlement at Abutment 1		-329	-822	-273	278	84	-110	-22
Unfactored effect of 1 in. settlement at Pier 1		702	1753	609	-534	-161	212	43
Unfactored effect of 1 in. settlement at Pier 2		-469	-1174	-79	1016	344	-328	-65
Unfactored effect of 1 in. settlement at Pier 3		192	452	-479	-1409	321	2050	411
Unfactored effect of 1 in. settlement at Abutment 2		-82	-208	221	651	-587	-1825	-364

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E3(1)-M2

Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.50	1.00	1.20	0.50	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E3(1)-M3

Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.25	0.50	0.60	0.25	0.30

Factored Relevant Settlements, S_f

This example is based on load factor γ_{SE} = 1.25

Table E3(1)-M4

Factored Relevant Settlements, S_f (in.) using γ_{SE} = 1.25				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.31	0.63	0.75	0.31	0.38

Table E3(1)-M5		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Effect of unfactored S_{tr} at Abutment 1		-82	-206	-68	70	21	-28	-6
Effect of unfactored S_{tr} at Pier 1		351	877	305	-267	-81	106	22
Effect of unfactored S_{tr} at Pier 2		-281	-704	-47	610	206	-197	-39
Effect of unfactored S_{tr} at Pier 3		48	113	-120	-352	80	513	103
Effect of unfactored S_{tr} at Abutment 2		-25	-62	66	195	-176	-548	-109
Total unfactored effect of S_{tr} at all supports	+ve	*399*	*990*	*371*	*874*	*308*	*619*	*124*
	-ve	*-388*	*-972*	*-235*	*-619*	*-257*	*-772*	*-154*
Total factored effect of sett using γ_{SE} = 1.00 and S_t	+ve	*798*	*1979*	*742*	*1749*	*615*	*1237*	*249*
	-ve	*-777*	*-1945*	*-471*	*-1239*	*-513*	*-1544*	*-307*
Total factored effect of sett using γ_{SE} = 1.25 and S_{tr}	+ve	*499*	*1237*	*464*	*1093*	*385*	*773*	*155*
	-ve	*-485*	*-1215*	*-294*	*-774*	*-321*	*-965*	*-192*
Total factored effect of sett using γ_{SE} = 1.00 and S_{tr}	+ve	*399*	*990*	*371*	*874*	*308*	*619*	*124*
	-ve	*-388*	*-972*	*-235*	*-619*	*-257*	*-772*	*-154*

Table E3(1)-M6 Service I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	10285	-12754	16640	-32725	23254	-23162	6030
	Min	713	-26170	4827	-47099	11256	-40406	-619
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	11083	-10775	17382	-30976	23869	-21925	6279
	Min	-64	-28115	4356	-48338	10743	-41950	-926
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	10784	-11517	17104	-31632	23639	-22389	6185
	Min	228	-27385	4533	-47873	10935	-41371	-811
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	10684	-11765	17011	-31851	23562	-22544	6154
	Min	325	-27142	4592	-47718	10999	-41178	-773
Ratio of Case 3 to Case 1	Max	1.048	0.903	1.028	0.967	1.017	0.967	1.026
	Min	0.319	1.046	0.939	1.016	0.972	1.024	1.310
Ratio of Case 3 to Case 2	Max	*0.973*	*1.069*	*0.984*	*1.021*	*0.990*	*1.021*	*0.985*
	Min	*-3.586*	*0.974*	*1.041*	*0.990*	*1.018*	*0.986*	*0.876*
Ratio of Case 2 to Case 1	Max	1.078	0.845	1.045	0.947	1.026	0.947	1.041
	Min	-0.089	1.074	0.902	1.026	0.954	1.038	1.497
Ratio of Case 3 to Case 4	Max	1.009	0.979	1.005	0.993	1.003	0.993	1.005
	Min	0.701	1.009	0.987	1.003	0.994	1.005	1.050
Ratio of Case 4 to Case 2	Max	0.964	1.092	0.979	1.028	0.987	1.028	0.980
	Min	-5.114	0.965	1.054	0.987	1.024	0.982	0.834

Table E3(1)-M7 Strength I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	16057	-14539	25120	-40323	33938	-27622	9727
	Min	-694	-38017	4447	-65478	12942	-57799	-1909
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	16855	-12560	25861	-38574	34553	-26385	9976
	Min	-1471	-39962	3976	-66716	12428	-59342	-2216
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	16556	-13302	25583	-39230	34323	-26848	9882
	Min	-1180	-39232	4153	-66252	12621	-58763	-2101
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	16456	-13550	25490	-39449	34246	-27003	9851
	Min	-1083	-38989	4211	-66097	12685	-58570	-2062
Ratio of Case 3 to Case 1	Max	1.031	0.915	1.018	0.973	1.011	0.972	1.016
	Min	1.699	1.032	0.934	1.012	0.975	1.017	1.101
Ratio of Case 3 to Case 2	Max	*0.982*	1.059	*0.989*	1.017	*0.993*	1.018	*0.991*
	Min	0.802	*0.982*	1.044	*0.993*	1.015	*0.990*	0.948
Ratio of Case 2 to Case 1	Max	1.050	0.864	1.030	0.957	1.018	0.955	1.026
	Min	2.118	1.051	0.894	1.019	0.960	1.027	1.161
Ratio of Case 3 to Case 4	Max	1.006	0.982	1.004	0.994	1.002	0.994	1.003
	Min	1.090	1.006	0.986	1.002	0.995	1.003	1.019
Ratio of Case 4 to Case 2	Max	0.976	1.079	0.986	1.023	0.991	1.023	0.988
	Min	0.736	0.976	1.059	0.991	1.021	0.987	0.931

Example E3(1)
Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Shear Comparison

Table E3(1)-S1		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Unfactored effect of 1 in. settlement at Abutment 1		-4.9	-4.9	3.8	3.8	-1.2	-1.2	0.7	0.7
Unfactored effect of 1 in. settlement at Pier 1		10.4	10.4	-7.8	-7.8	2.2	2.2	-1.3	-1.3
Unfactored effect of 1 in. settlement at Pier 2		-7.0	-7.0	7.5	7.5	-4.0	-4.0	2.0	2.0
Unfactored effect of 1 in. settlement at Pier 3		2.7	2.7	-6.4	-6.4	10.3	10.3	-12.4	-12.4
Unfactored effect of 1 in. settlement at Abutment 2		-1.2	-1.2	2.9	2.9	-7.4	-7.4	11.1	11.1

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E3(1)-S2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.50	1.00	1.20	0.50	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E3(1)-S3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.25	0.50	0.60	0.25	0.30

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.25

Table E3(1)-S4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.31	0.63	0.75	0.31	0.38

Table E3(1)-S5		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Effect of unfactored S_{tr} at Abutment 1		-1.2	-1.2	0.9	0.9	-0.3	-0.3	0.2	0.2
Effect of unfactored S_{tr} at Pier 1		5.2	5.2	-3.9	-3.9	1.1	1.1	-0.6	-0.6
Effect of unfactored S_{tr} at Pier 2		-4.2	-4.2	4.5	4.5	-2.4	-2.4	1.2	1.2
Effect of unfactored S_{tr} at Pier 3		0.7	0.7	-1.6	-1.6	2.6	2.6	-3.1	-3.1
Effect of unfactored S_{tr} at Abutment 2		-0.4	-0.4	0.9	0.9	-2.2	-2.2	3.3	3.3
Total unfactored effect of S_{tr} at all supports	+ve	*6*	*6*	*6*	*6*	*4*	*4*	*5*	*5*
	-ve	*-6*	*-6*	*-5*	*-5*	*-5*	*-5*	*-4*	*-4*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*12*	*12*	*13*	*13*	*7*	*7*	*9*	*9*
	-ve	*-12*	*-12*	*-11*	*-11*	*-10*	*-10*	*-8*	*-7*
Total factored effect of settlement using g_{SE} = 1.25 and S_{tr}	+ve	*7*	*7*	*8*	*8*	*5*	*5*	*6*	*6*
	-ve	*-7*	*-7*	*-7*	*-7*	*-6*	*-6*	*-5*	*-5*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*6*	*6*	*6*	*6*	*4*	*4*	*5*	*5*
	-ve	*-6*	*-6*	*-5*	*-5*	*-5*	*-5*	*-4*	*-4*

Table E3(1)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	317	-330	598	-514	796	-476	632	-38
	Min	114	-537	348	-751	554	-732	414	-259
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	328	-318	611	-501	804	-469	642	-28
	Min	102	-549	337	-762	544	-742	406	-267
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	324	-323	606	-506	801	-472	638	-32
	Min	106	-544	341	-758	548	-739	409	-264
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	322	-324	604	-507	800	-473	637	-33
	Min	108	-543	342	-756	549	-737	410	-263
Ratio of Case 3 to Case 1	Max	1.023	0.978	1.013	0.985	1.006	0.990	1.009	0.845
	Min	0.936	1.013	0.980	1.009	0.989	1.008	0.989	1.018
Ratio of Case 3 to Case 2	Max	0.987	1.014	0.992	1.009	0.997	1.006	0.995	1.124
	Min	1.043	0.992	1.012	0.995	1.007	0.995	1.007	0.989
Ratio of Case 2 to Case 1	Max	1.037	0.964	1.021	0.975	1.009	0.985	1.015	0.752
	Min	0.898	1.022	0.968	1.015	0.982	1.013	0.982	1.029
Ratio of Case 3 to Case 4	Max	1.005	0.995	1.003	0.997	1.001	0.998	1.002	0.965
	Min	0.987	1.003	0.996	1.002	0.998	1.002	0.998	1.004
Ratio of Case 4 to Case 2	Max	0.982	1.018	0.990	1.013	0.995	1.008	0.993	1.165
	Min	1.057	0.989	1.016	0.993	1.009	0.993	1.009	0.986

Table E3(1)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	475	-405	854	-636	1112	-583	892	-16
	Min	120	-767	416	-1051	688	-1030	510	-404
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	487	-393	867	-623	1119	-575	901	-6
	Min	109	-779	406	-1062	678	-1040	502	-411
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	483	-398	862	-628	1116	-578	898	-10
	Min	113	-775	410	-1058	682	-1037	505	-408
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	481	-399	861	-629	1115	-579	897	-11
	Min	115	-773	411	-1056	683	-1035	506	-407
Ratio of Case 3 to Case 1	Max	1.015	0.982	1.009	0.988	1.004	0.992	1.007	0.624
	Min	0.940	1.009	0.984	1.007	0.991	1.006	0.991	1.012
Ratio of Case 3 to Case 2	Max	0.991	1.011	0.995	1.008	0.998	1.005	0.996	1.565
	Min	1.040	0.994	1.010	0.996	1.005	0.996	1.006	0.993
Ratio of Case 2 to Case 1	Max	1.025	0.971	1.015	0.980	1.007	0.987	1.010	0.399
	Min	0.904	1.015	0.974	1.010	0.986	1.010	0.985	1.019
Ratio of Case 3 to Case 4	Max	1.003	0.996	1.002	0.997	1.001	0.998	1.001	0.893
	Min	0.987	1.002	0.997	1.001	0.998	1.001	0.998	1.002
Ratio of Case 4 to Case 2	Max	0.988	1.015	0.993	1.010	0.997	1.006	0.995	1.753
	Min	1.053	0.993	1.014	0.995	1.007	0.995	1.007	0.991

Example E3(2)

Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Moment Comparison

Table E3(2)-M1		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Unfactored effect of 1 in. settlement at Abutment 1		-329	-822	-273	278	84	-110	-22
Unfactored effect of 1 in. settlement at Pier 1		702	1753	609	-534	-161	212	43
Unfactored effect of 1 in. settlement at Pier 2		-469	-1174	-79	1016	344	-328	-65
Unfactored effect of 1 in. settlement at Pier 3		192	452	-479	-1409	321	2050	411
Unfactored effect of 1 in. settlement at Abutment 2		-82	-208	221	651	-587	-1825	-364

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E3(2)-M2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.50	1.00	1.20	0.50	0.60

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E3(2)-M3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.25	0.50	0.60	0.25	0.30

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.75

Table E3(2)-M4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.44	0.88	1.05	0.44	0.53

Table E3(2)-M5		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Effect of unfactored S_{tr} at Abutment 1		-82	-206	-68	70	21	-28	-6
Effect of unfactored S_{tr} at Pier 1		351	877	305	-267	-81	106	22
Effect of unfactored S_{tr} at Pier 2		-281	-704	-47	610	206	-197	-39
Effect of unfactored S_{tr} at Pier 3		48	113	-120	-352	80	513	103
Effect of unfactored S_{tr} at Abutment 2		-25	-62	66	195	-176	-548	-109
Total unfactored effect of S_{tr} at all supports	+ve	*399*	*990*	*371*	*874*	*308*	*619*	*124*
	-ve	*-388*	*-972*	*-235*	*-619*	*-257*	*-772*	*-154*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*798*	*1979*	*742*	*1749*	*615*	*1237*	*249*
	-ve	*-777*	*-1945*	*-471*	*-1239*	*-513*	*-1544*	*-307*
Total factored effect of sett using g_{SE} = 1.75 and S_{tr}	+ve	*698*	*1732*	*649*	*1530*	*538*	*1082*	*217*
	-ve	*-679*	*-1702*	*-412*	*-1084*	*-449*	*-1351*	*-269*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*399*	*990*	*371*	*874*	*308*	*619*	*124*
	-ve	*-388*	*-972*	*-235*	*-619*	*-257*	*-772*	*-154*

Table E3(2)-M6 Service I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	10285	-12754	16640	-32725	23254	-23162	6030
	Min	713	-26170	4827	-47099	11256	-40406	-619
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	11083	-10775	17382	-30976	23869	-21925	6279
	Min	-64	-28115	4356	-48338	10743	-41950	-926
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	10983	-11022	17289	-31195	23792	-22080	6247
	Min	34	-27872	4415	-48183	10807	-41757	-888
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	10684	-11765	17011	-31851	23562	-22544	6154
	Min	325	-27142	4592	-47718	10999	-41178	-773
Ratio of Case 3 to Case 1	Max	1.068	0.864	1.039	0.953	1.023	0.953	1.036
	Min	0.047	1.065	0.915	1.023	0.960	1.033	1.435
Ratio of Case 3 to Case 2	Max	*0.991*	*1.023*	*0.995*	*1.007*	*0.997*	*1.007*	*0.995*
	Min	*-0.529*	*0.991*	*1.014*	*0.997*	*1.006*	*0.995*	*0.959*
Ratio of Case 2 to Case 1	Max	1.078	0.845	1.045	0.947	1.026	0.947	1.041
	Min	-0.089	1.074	0.902	1.026	0.954	1.038	1.497
Ratio of Case 3 to Case 4	Max	1.028	0.937	1.016	0.979	1.010	0.979	1.015
	Min	0.103	1.027	0.962	1.010	0.983	1.014	1.149
Ratio of Case 4 to Case 2	Max	0.964	1.092	0.979	1.028	0.987	1.028	0.980
	Min	-5.114	0.965	1.054	0.987	1.024	0.982	0.834

Table E3(2)-M7 Strength I Comparison		Moment (kip-ft)						
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	16057	-14539	25120	-40323	33938	-27622	9727
	Min	-694	-38017	4447	-65478	12942	-57799	-1909
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	16855	-12560	25861	-38574	34553	-26385	9976
	Min	-1471	-39962	3976	-66716	12428	-59342	-2216
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	16755	-12807	25768	-38793	34476	-26539	9944
	Min	-1374	-39719	4035	-66561	12492	-59149	-2178
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	16456	-13550	25490	-39449	34246	-27003	9851
	Min	-1083	-38989	4211	-66097	12685	-58570	-2062
Ratio of Case 3 to Case 1	Max	1.043	0.881	1.026	0.962	1.016	0.961	1.022
	Min	1.979	1.045	0.907	1.017	0.965	1.023	1.141
Ratio of Case 3 to Case 2	Max	*0.994*	1.020	*0.996*	1.006	*0.998*	1.006	*0.997*
	Min	0.934	*0.994*	1.015	*0.998*	1.005	*0.997*	0.983
Ratio of Case 2 to Case 1	Max	1.050	0.864	1.030	0.957	1.018	0.955	1.026
	Min	2.118	1.051	0.894	1.019	0.960	1.027	1.161
Ratio of Case 3 to Case 4	Max	1.018	0.945	1.011	0.983	1.007	0.983	1.009
	Min	1.269	1.019	0.958	1.007	0.985	1.010	1.056
Ratio of Case 4 to Case 2	Max	0.976	1.079	0.986	1.023	0.991	1.023	0.988
	Min	0.736	0.976	1.059	0.991	1.021	0.987	0.931

Example E3(2)
Four-Span Bridge, Span Lengths 168 FT, 293 FT, 335 FT, and 165 Ft, Girder Spacing 12 ft-3 in.

Shear Comparison

Table E3(2)-S1		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Unfactored effect of 1 in. settlement at Abutment 1		-4.9	-4.9	3.8	3.8	-1.2	-1.2	0.7	0.7
Unfactored effect of 1 in. settlement at Pier 1		10.4	10.4	-7.8	-7.8	2.2	2.2	-1.3	-1.3
Unfactored effect of 1 in. settlement at Pier 2		-7.0	-7.0	7.5	7.5	-4.0	-4.0	2.0	2.0
Unfactored effect of 1 in. settlement at Pier 3		2.7	2.7	-6.4	-6.4	10.3	10.3	-12.4	-12.4
Unfactored effect of 1 in. settlement at Abutment 2		-1.2	-1.2	2.9	2.9	-7.4	-7.4	11.1	11.1

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E3(2)-S2				
Predicted Unfactored Total Settlements, S_t (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.50	1.00	1.20	0.50	0.60

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E3(2)-S3				
Estimated Unfactored Relevant Settlements, S_{tr} (in.)				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.25	0.50	0.60	0.25	0.30

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.75

Table E3(2)-S4				
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75				
Abutment 1	Pier 1	Pier 2	Pier 3	Abutment 2
0.44	0.88	1.05	0.44	0.53

Table E3(2)-S5		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Unfactored DL shear (No settlement)		157.1	-345.5	384.0	-526.5	564.0	-502.5	428.5	-100.7
Unfactored LL shear	+ve	159.5	15.4	213.9	12.9	232.3	26.0	203.7	63.1
	-ve	-43.4	-191.7	-36.3	-224.4	-9.9	-229.9	-14.8	-158.7
Effect of unfactored S_{tr} at Abutment 1		-1.2	-1.2	0.9	0.9	-0.3	-0.3	0.2	0.2
Effect of unfactored S_{tr} at Pier 1		5.2	5.2	-3.9	-3.9	1.1	1.1	-0.6	-0.6
Effect of unfactored S_{tr} at Pier 2		-4.2	-4.2	4.5	4.5	-2.4	-2.4	1.2	1.2
Effect of unfactored S_{tr} at Pier 3		0.7	0.7	-1.6	-1.6	2.6	2.6	-3.1	-3.1
Effect of unfactored S_{tr} at Abutment 2		-0.4	-0.4	0.9	0.9	-2.2	-2.2	3.3	3.3
Total unfactored effect of S_{tr} at all supports	+ve	*6*	*6*	*6*	*6*	*4*	*4*	*5*	*5*
	-ve	*-6*	*-6*	*-5*	*-5*	*-5*	*-5*	*-4*	*-4*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*12*	*12*	*13*	*13*	*7*	*7*	*9*	*9*
	-ve	*-12*	*-12*	*-11*	*-11*	*-10*	*-10*	*-8*	*-7*
Total factored effect of settlement using g_{SE} = 1.75 and S_{tr}	+ve	*10*	*10*	*11*	*11*	*6*	*6*	*8*	*8*
	-ve	*-10*	*-10*	*-10*	*-10*	*-9*	*-9*	*-7*	*-7*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*6*	*6*	*6*	*6*	*4*	*4*	*5*	*5*
	-ve	*-6*	*-6*	*-5*	*-5*	*-5*	*-5*	*-4*	*-4*

Table E3(2)-S6 Service I Comparison		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	317	-330	598	-514	796	-476	632	-38
	Min	114	-537	348	-751	554	-732	414	-259
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	328	-318	611	-501	804	-469	642	-28
	Min	102	-549	337	-762	544	-742	406	-267
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	327	-320	609	-503	803	-470	640	-29
	Min	104	-547	338	-761	546	-741	407	-266
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	322	-324	604	-507	800	-473	637	-33
	Min	108	-543	342	-756	549	-737	410	-263
Ratio of Case 3 to Case 1	Max	1.033	0.969	1.018	0.979	1.008	0.986	1.013	0.783
	Min	0.911	1.019	0.972	1.013	0.985	1.012	0.984	1.025
Ratio of Case 3 to Case 2	Max	0.996	1.005	0.997	1.003	0.999	1.002	0.998	1.041
	Min	1.014	0.997	1.004	0.998	1.002	0.998	1.002	0.996
Ratio of Case 2 to Case 1	Max	1.037	0.964	1.021	0.975	1.009	0.985	1.015	0.752
	Min	0.898	1.022	0.968	1.015	0.982	1.013	0.982	1.029
Ratio of Case 3 to Case 4	Max	1.014	0.986	1.008	0.991	1.003	0.994	1.006	0.894
	Min	0.960	1.008	0.988	1.005	0.993	1.005	0.993	1.011
Ratio of Case 4 to Case 2	Max	0.982	1.018	0.990	1.013	0.995	1.008	0.993	1.165
	Min	1.057	0.989	1.016	0.993	1.009	0.993	1.009	0.986

Table E3(2)-S7 Strength I Comparison		Shear (kip)							
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	475	-405	854	-636	1112	-583	892	-16
	Min	120	-767	416	-1051	688	-1030	510	-404
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	487	-393	867	-623	1119	-575	901	-6
	Min	109	-779	406	-1062	678	-1040	502	-411
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	486	-395	865	-625	1118	-576	900	-7
	Min	110	-777	407	-1061	679	-1039	503	-410
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	481	-399	861	-629	1115	-579	897	-11
	Min	115	-773	411	-1056	683	-1035	506	-407
Ratio of Case 3 to Case 1	Max	1.022	0.975	1.013	0.983	1.006	0.989	1.009	0.474
	Min	0.916	1.013	0.977	1.009	0.988	1.008	0.987	1.016
Ratio of Case 3 to Case 2	Max	0.997	1.004	0.998	1.003	0.999	1.002	0.999	1.188
	Min	1.013	0.998	1.003	0.999	1.002	0.999	1.002	0.998
Ratio of Case 2 to Case 1	Max	1.025	0.971	1.015	0.980	1.007	0.987	1.010	0.399
	Min	0.904	1.015	0.974	1.010	0.986	1.010	0.985	1.019
Ratio of Case 3 to Case 4	Max	1.009	0.989	1.005	0.992	1.002	0.995	1.004	0.678
	Min	0.962	1.006	0.990	1.004	0.995	1.004	0.994	1.007
Ratio of Case 4 to Case 2	Max	0.988	1.015	0.993	1.010	0.997	1.006	0.995	1.753
	Min	1.053	0.993	1.014	0.995	1.007	0.995	1.007	0.991

Example E4(1)

Five-Span Bridge, Span Lengths 120 ft, 140 ft, 140 ft, 140 ft, and 120 ft, Girder Spacing 11 ft-2 in.

Moment Comparison

Table E4(1)-M1		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Unfactored DL moment (No Settlement)		2524	-4544	1807	-4213	1967	-4224	1822	-4522	2522
Unfactored LL moment	+ve	2369	432	2186	553	2231	542	2194	420	2357
	-ve	-610	-2629	-694	-2653	-710	-2653	-693	-2612	-591
Unfactored effect of 1 in. settlement at Abutment 1		-368	-920	-330	259	94	-72	-26	20	8
Unfactored effect of 1 in. settlement at Pier 1		585	1462	332	-797	-287	222	80	-62	-25
Unfactored effect of 1 in. settlement at Pier 2		-277	-691	192	1075	194	-687	-248	192	77
Unfactored effect of 1 in. settlement at Pier 3		77	194	-246	-689	195	1077	196	-684	-274
Unfactored effect of 1 in. settlement at Pier 4		-25	-62	82	225	-287	-799	334	1468	587
Unfactored effect of 1 in. settlement at Abutment 2		8	22	-26	-73	94	260	-337	-933	-373

Predicted Unfactored Total Settlements, S_t

Based on an appropriate owner-approved and calibrated method.

Table E4(1)-M2					
Predicted Unfactored Total Settlements, S_t (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.90	1.50	1.80	1.00	2.30	1.40

Estimated Unfactored Relevant Settlements, S_{tr}

Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E4(1)-M3					
Estimated Unfactored Relevant Settlements, S_{tr} (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.45	0.75	0.90	0.50	1.15	0.70

Factored Relevant Settlements, S_f

This example is based on load factor g_{SE} = 1.25

Table E4(1)-M4					
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.56	0.94	1.13	0.63	1.44	0.88

Table E4(1)-M5		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Unfactored DL moment (No Settlement)		2524	-4544	1807	-4213	1967	-4224	1822	-4522	2522
Unfactored LL moment	+ve	2369	432	2186	553	2231	542	2194	420	2357
	-ve	-610	-2629	-694	-2653	-710	-2653	-693	-2612	-591
Effect of unfactored S_{tr} at Abutment 1		-166	-414	-149	117	42	-32	-12	9	4
Effect of unfactored S_{tr} at Pier 1		439	1097	249	-598	-215	167	60	-47	-19
Effect of unfactored S_{tr} at Pier 2		-249	-622	173	968	175	-618	-223	173	69
Effect of unfactored S_{tr} at Pier 3		39	97	-123	-345	98	539	98	-342	-137
Effect of unfactored S_{tr} at Pier 4		-29	-71	94	259	-330	-919	384	1688	675
Effect of unfactored S_{tr} at Abutment 2		6	15	-18	-51	66	182	-236	-653	-261
Total unfactored effect of S_{tr} at all supports	+ve	*483*	*1209*	*516*	*1343*	*380*	*887*	*542*	*1870*	*748*
	-ve	*-444*	*-1107*	*-290*	*-993*	*-545*	*-1570*	*-471*	*-1042*	*-417*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*966*	*2418*	*1032*	*2686*	*760*	*1774*	*1084*	*3740*	*1496*
	-ve	*-887*	*-2214*	*-579*	*-1987*	*-1091*	*-3139*	*-942*	*-2083*	*-834*
Total factored effect of sett using g_{SE} = 1.25 and S_{tr}	+ve	*604*	*1511*	*645*	*1679*	*475*	*1109*	*678*	*2338*	*935*
	-ve	*-555*	*-1384*	*-362*	*-1242*	*-682*	*-1962*	*-589*	*-1302*	*-521*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*483*	*1209*	*516*	*1343*	*380*	*887*	*542*	*1870*	*748*
	-ve	*-444*	*-1107*	*-290*	*-993*	*-545*	*-1570*	*-471*	*-1042*	*-417*

Table E4(1)-M6 Service I Comparison		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	4893	-4112	3993	-3660	4198	-3682	4016	-4102	4879
	Min	1914	-7173	1113	-6866	1257	-6877	1129	-7134	1931
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	5859	-1694	5025	-974	4958	-1908	5100	-362	6375
	Min	1027	-9387	534	-8853	166	-10016	187	-9217	1097
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	5497	-2601	4638	-1982	4673	-2573	4694	-1765	5814
	Min	1359	-8557	751	-8108	575	-8839	541	-8436	1410
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	5376	-2903	4509	-2317	4578	-2795	4558	-2232	5627
	Min	1470	-8280	823	-7859	712	-8447	658	-8176	1514
Ratio of Case 3 to Case 1	Max	1.123	0.633	1.162	0.541	1.113	0.699	1.169	0.430	1.192
	Min	0.710	1.193	0.675	1.181	0.458	1.285	0.479	1.183	0.730
Ratio of Case 3 to Case 2	Max	*0.938*	*1.535*	*0.923*	*2.034*	*0.942*	*1.349*	*0.920*	*4.874*	*0.912*
	Min	*1.324*	*0.912*	*1.407*	*0.916*	*3.458*	*0.882*	*2.884*	*0.915*	*1.285*
Ratio of Case 2 to Case 1	Max	1.197	0.412	1.259	0.266	1.181	0.518	1.270	0.088	1.307
	Min	0.536	1.309	0.479	1.289	0.132	1.456	0.166	1.292	0.568
Ratio of Case 3 to Case 4	Max	1.022	0.896	1.029	0.855	1.021	0.921	1.030	0.791	1.033
	Min	0.925	1.033	0.912	1.032	0.808	1.046	0.821	1.032	0.931
Ratio of Case 4 to Case 2	Max	0.918	1.714	0.897	2.378	0.923	1.465	0.894	6.166	0.883
	Min	1.432	0.882	1.543	0.888	4.277	0.843	3.512	0.887	1.380

Table E4(1)-M7 Strength I Comparison		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	7301	-4924	6084	-4299	6363	-4332	6117	-4918	7277
	Min	2088	-10281	1044	-9909	1216	-9923	1065	-10224	2118
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	8266	-2506	7116	-1613	7123	-2558	7201	-1178	8773
	Min	1200	-12495	465	-11896	126	-13062	123	-12307	1285
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	7904	-3413	6729	-2620	6838	-3223	6795	-2580	8212
	Min	1533	-11665	682	-11151	535	-11885	476	-11526	1597
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	7784	-3715	6600	-2956	6743	-3445	6659	-3048	8025
	Min	1644	-11388	755	-10902	671	-11492	594	-11265	1701
Ratio of Case 3 to Case 1	Max	1.083	0.693	1.106	0.610	1.075	0.744	1.111	0.525	1.128
	Min	0.734	1.135	0.653	1.125	0.440	1.198	0.447	1.127	0.754
Ratio of Case 3 to Case 2	Max	*0.956*	1.362	*0.946*	1.624	*0.960*	1.260	*0.944*	2.191	*0.936*
	Min	1.277	*0.934*	1.467	*0.937*	4.255	*0.910*	3.867	*0.937*	1.243
Ratio of Case 2 to Case 1	Max	1.132	0.509	1.170	0.375	1.120	0.590	1.177	0.239	1.206
	Min	0.575	1.215	0.445	1.200	0.103	1.316	0.116	1.204	0.606
Ratio of Case 3 to Case 4	Max	1.016	0.919	1.020	0.886	1.014	0.936	1.020	0.847	1.023
	Min	0.933	1.024	0.904	1.023	0.797	1.034	0.802	1.023	0.939
Ratio of Case 4 to Case 2	Max	0.942	1.482	0.927	1.833	0.947	1.347	0.925	2.588	0.915
	Min	1.370	0.911	1.623	0.916	5.340	0.880	4.823	0.915	1.325

Example E4(1)
Five-Span Bridge, Span Lengths 120 ft, 140 ft, 140 ft, 140 ft, and 120 ft, Girder Spacing 11 ft-2 in.

Shear Comparison

Table E4(1)-S1		Shear (kip)									
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Unfactored DL shear (No settlement)		112.6	-190.3	180.6	-175.9	178.1	-178.3	175.5	-180.9	189.3	-112.6
Unfactored LL shear	+ve	125.0	4.6	147.1	18.3	149.4	19.0	148.3	17.6	145.4	15.8
	-ve	-16.3	-145.4	-18.0	-146.5	-19.4	-147.6	-18.5	-146.8	-4.5	-124.8
Unfactored effect of 1 in. settlement at Abutment 1		-7.7	-7.7	8.4	8.4	-2.4	-2.4	0.7	0.7	-0.2	-0.2
Unfactored effect of 1 in. settlement at Pier 1		12.2	12.2	-16.1	-16.1	7.3	7.3	-2.0	-2.0	0.5	0.5
Unfactored effect of 1 in. settlement at Pier 2		-5.8	-5.8	12.6	12.6	-12.6	-12.6	6.3	6.3	-1.6	-1.6
Unfactored effect of 1 in. settlement at Pier 3		1.6	1.6	-6.3	-6.3	12.6	12.6	-12.6	-12.6	5.7	5.7
Unfactored effect of 1 in. settlement at Pier 4		-0.5	-0.5	2.1	2.1	-7.3	-7.3	16.2	16.2	-12.2	-12.2
Unfactored effect of 1 in. settlement at Abutment 2		0.2	0.2	-0.7	-0.7	2.4	2.4	-8.5	-8.5	7.8	7.8

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E4(1)-S2					
Predicted Unfactored Total Settlements, S_t (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.90	1.50	1.80	1.00	2.30	1.40

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E4(1)-S3					
Estimated Unfactored Relevant Settlements, S_{tr} (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.45	0.75	0.90	0.50	1.15	0.70

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.25

Table E4(1)-S4					
Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.25					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.56	0.94	1.13	0.63	1.44	0.88

Table E4(1)-S5		Shear (kip)									
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Unfactored DL shear (No settlement)		112.6	-190.3	180.6	-175.9	178.1	-178.3	175.5	-180.9	189.3	-112.6
Unfactored LL shear	+ve	125.0	4.6	147.1	18.3	149.4	19.0	148.3	17.6	145.4	15.8
	-ve	-16.3	-145.4	-18.0	-146.5	-19.4	-147.6	-18.5	-146.8	-4.5	-124.8
Effect of unfactored S_{tr} at Abutment 1		-3.5	-3.5	3.8	3.8	-1.1	-1.1	0.3	0.3	-0.1	-0.1
Effect of unfactored S_{tr} at Pier 1		9.1	9.1	-12.1	-12.1	5.5	5.5	-1.5	-1.5	0.4	0.4
Effect of unfactored S_{tr} at Pier 2		-5.2	-5.2	11.4	11.4	-11.3	-11.3	5.7	5.7	-1.4	-1.4
Effect of unfactored S_{tr} at Pier 3		0.8	0.8	-3.1	-3.1	6.3	6.3	-6.3	-6.3	2.8	2.9
Effect of unfactored S_{tr} at Pier 4		-0.6	-0.6	2.4	2.4	-8.4	-8.4	18.6	18.6	-14.1	-14.1
Effect of unfactored S_{tr} at Abutment 2		0.1	0.1	-0.5	-0.5	1.7	1.7	-6.0	-6.0	5.4	5.4
Total unfactored effect of S_{tr} *at all supports*	+ve	*10*	*10*	*18*	*18*	*13*	*13*	*25*	*25*	*9*	*9*
	-ve	*-9*	*-9*	*-16*	*-16*	*-21*	*-21*	*-14*	*-14*	*-16*	*-16*
Total factored effect of settlement using g_{SE} = 1.00 and S_t	+ve	*20*	*20*	*35*	*35*	*27*	*27*	*49*	*49*	*17*	*17*
	-ve	*-19*	*-19*	*-31*	*-31*	*-42*	*-42*	*-28*	*-28*	*-31*	*-31*
Total factored effect of settlement using g_{SE} = 1.25 and S_{tr}	+ve	*13*	*13*	*22*	*22*	*17*	*17*	*31*	*31*	*11*	*11*
	-ve	*-12*	*-12*	*-20*	*-20*	*-26*	*-26*	*-17*	*-17*	*-19*	*-19*
Total factored effect of settlement using g_{SE} = 1.00 and S_{tr}	+ve	*10*	*10*	*18*	*18*	*13*	*13*	*25*	*25*	*9*	*9*
	-ve	*-9*	*-9*	*-16*	*-16*	*-21*	*-21*	*-14*	*-14*	*-16*	*-16*

Table E4(1)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	238	-186	328	-158	328	-159	324	-163	335	-97
	Min	96	-336	163	-322	159	-326	157	-328	185	-237
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	258	-166	363	-123	354	-132	373	-114	352	-79
	Min	78	-354	131	-354	117	-367	129	-355	154	-269
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	250	-173	350	-136	344	-142	354	-133	346	-86
	Min	85	-347	143	-342	133	-352	140	-345	165	-257
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	248	-176	345	-140	341	-146	348	-139	343	-88
	Min	87	-345	147	-338	138	-347	143	-341	169	-253
Ratio of Case 3 to Case 1	Max	1.053	0.932	1.067	0.861	1.051	0.895	1.095	0.812	1.032	0.888
	Min	0.880	1.034	0.879	1.061	0.836	1.080	0.890	1.053	0.895	1.082
Ratio of Case 3 to Case 2	Max	0.971	1.046	0.964	1.107	0.972	1.076	0.951	1.161	0.981	1.082
	Min	1.089	0.980	1.090	0.967	1.133	0.958	1.080	0.971	1.076	0.957
Ratio of Case 2 to Case 1	Max	1.085	0.892	1.107	0.778	1.082	0.831	1.152	0.699	1.052	0.821
	Min	0.808	1.055	0.807	1.097	0.738	1.128	0.825	1.084	0.832	1.131
Ratio of Case 3 to Case 4	Max	1.010	0.986	1.013	0.969	1.010	0.977	1.018	0.956	1.006	0.975
	Min	0.973	1.007	0.973	1.012	0.962	1.015	0.976	1.010	0.977	1.015
Ratio of Case 4 to Case 2	Max	0.961	1.061	0.952	1.143	0.962	1.101	0.934	1.215	0.975	1.109
	Min	1.119	0.974	1.120	0.956	1.178	0.943	1.106	0.961	1.101	0.942

Table E4(1)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	360	-230	483	-188	484	-190	479	-195	491	-113
	Min	112	-492	194	-476	189	-481	187	-483	229	-359
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	380	-210	518	-153	511	-163	528	-146	508	-96
	Min	94	-511	163	-508	147	-523	159	-511	198	-390
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.25 and S_{tr})	Max	372	-217	505	-166	501	-173	510	-165	502	-102
	Min	101	-504	175	-496	163	-507	170	-500	209	-379
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	370	-220	501	-170	497	-176	503	-171	500	-104
	Min	103	-502	178	-492	168	-502	173	-497	213	-375
Ratio of Case 3 to Case 1	Max	1.035	0.945	1.045	0.883	1.035	0.911	1.064	0.843	1.022	0.904
	Min	0.897	1.023	0.899	1.041	0.862	1.054	0.908	1.036	0.915	1.054
Ratio of Case 3 to Case 2	Max	0.980	1.036	0.975	1.086	0.980	1.062	0.965	1.126	0.987	1.068
	Min	1.074	0.986	1.072	0.977	1.106	0.970	1.065	0.980	1.059	0.970
Ratio of Case 2 to Case 1	Max	1.056	0.912	1.073	0.813	1.055	0.858	1.103	0.748	1.035	0.846
	Min	0.835	1.038	0.838	1.066	0.780	1.086	0.853	1.057	0.864	1.087
Ratio of Case 3 to Case 4	Max	1.007	0.989	1.009	0.974	1.007	0.981	1.012	0.964	1.004	0.979
	Min	0.978	1.005	0.978	1.008	0.969	1.010	0.980	1.007	0.982	1.010
Ratio of Case 4 to Case 2	Max	0.974	1.048	0.966	1.115	0.974	1.083	0.953	1.168	0.983	1.091
	Min	1.099	0.982	1.097	0.969	1.141	0.960	1.086	0.973	1.079	0.960

Example E4(2)
Five-Span Bridge, Span Lengths 120 ft, 140 ft, 140 ft, 140 ft, and 120 ft, Girder Spacing 11 ft-2 in.

Moment Comparison

Table E4(2)-M1		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Unfactored DL moment (No Settlement)		2524	-4544	1807	-4213	1967	-4224	1822	-4522	2522
Unfactored LL moment	+ve	2369	432	2186	553	2231	542	2194	420	2357
	-ve	-610	-2629	-694	-2653	-710	-2653	-693	-2612	-591
Unfactored effect of 1 in. settlement at Abutment 1		-368	-920	-330	259	94	-72	-26	20	8
Unfactored effect of 1 in. settlement at Pier 1		585	1462	332	-797	-287	222	80	-62	-25
Unfactored effect of 1 in. settlement at Pier 2		-277	-691	192	1075	194	-687	-248	192	77
Unfactored effect of 1 in. settlement at Pier 3		77	194	-246	-689	195	1077	196	-684	-274
Unfactored effect of 1 in. settlement at Pier 4		-25	-62	82	225	-287	-799	334	1468	587
Unfactored effect of 1 in. settlement at Abutment 2		8	22	-26	-73	94	260	-337	-933	-373

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E4(2)-M2

Predicted Unfactored Total Settlements, S_t (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.90	1.50	1.80	1.00	2.30	1.40

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E4(2)-M3

Estimated Unfactored Relevant Settlements, S_{tr} (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.45	0.75	0.90	0.50	1.15	0.70

Factored Relevant Settlements, S_f
This example is based on load factor g_{SE} = 1.75

Table E4(2)-M4

Factored Relevant Settlements, S_f (in.) using g_{SE} = 1.75					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.79	1.31	1.58	0.88	2.01	1.23

Table E4(2)-M5		Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Unfactored DL moment (No Settlement)		2524	-4544	1807	-4213	1967	-4224	1822	-4522	2522
Unfactored LL moment	+ve	2369	432	2186	553	2231	542	2194	420	2357
	-ve	-610	-2629	-694	-2653	-710	-2653	-693	-2612	-591
Effect of unfactored S_{tr} at Abutment 1		-166	-414	-149	117	42	-32	-12	9	4
Effect of unfactored S_{tr} at Pier 1		439	1097	249	-598	-215	167	60	-47	-19
Effect of unfactored S_{tr} at Pier 2		-249	-622	173	968	175	-618	-223	173	69
Effect of unfactored S_{tr} at Pier 3		39	97	-123	-345	98	539	98	-342	-137
Effect of unfactored S_{tr} at Pier 4		-29	-71	94	259	-330	-919	384	1688	675
Effect of unfactored S_{tr} at Abutment 2		6	15	-18	-51	66	182	-236	-653	-261
Total unfactored effect of S_{tr} at all supports	+ve	*483*	*1209*	*516*	*1343*	*380*	*887*	*542*	*1870*	*748*
	-ve	*-444*	*-1107*	*-290*	*-993*	*-545*	*-1570*	*-471*	*-1042*	*-417*
Total factored effect of sett using g_{SE} = 1.00 and S_t	+ve	*966*	*2418*	*1032*	*2686*	*760*	*1774*	*1084*	*3740*	*1496*
	-ve	*-887*	*-2214*	*-579*	*-1987*	*-1091*	*-3139*	*-942*	*-2083*	*-834*
Total factored effect of sett using g_{SE} = 1.75 and S_{tr}	+ve	*845*	*2116*	*903*	*2350*	*665*	*1552*	*949*	*3273*	*1309*
	-ve	*-776*	*-1938*	*-507*	*-1738*	*-954*	*-2747*	*-824*	*-1823*	*-729*
Total factored effect of sett using g_{SE} = 1.00 and S_{tr}	+ve	*483*	*1209*	*516*	*1343*	*380*	*887*	*542*	*1870*	*748*
	-ve	*-444*	*-1107*	*-290*	*-993*	*-545*	*-1570*	*-471*	*-1042*	*-417*

Table E4(2)-M6 Service I Comparison		Moment (kip-ft) Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Case 1: 1.0 DL + 1.0 LL without SE	Max	4893	-4112	3993	-3660	4198	-3682	4016	-4102	4879
	Min	1914	-7173	1113	-6866	1257	-6877	1129	-7134	1931
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	5859	-1694	5025	-974	4958	-1908	5100	-362	6375
	Min	1027	-9387	534	-8853	166	-10016	187	-9217	1097
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	5738	-1996	4896	-1310	4863	-2130	4965	-830	6188
	Min	1138	-9111	606	-8604	303	-9624	305	-8957	1202
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	5376	-2903	4509	-2317	4578	-2795	4558	-2232	5627
	Min	1470	-8280	823	-7859	712	-8447	658	-8176	1514
Ratio of Case 3 to Case 1	Max	1.173	0.486	1.226	0.358	1.158	0.578	1.236	0.202	1.268
	Min	0.594	1.270	0.544	1.253	0.241	1.399	0.270	1.256	0.622
Ratio of Case 3 to Case 2	Max	*0.979*	*1.178*	*0.974*	*1.345*	*0.981*	*1.116*	*0.973*	*2.291*	*0.971*
	Min	*1.108*	*0.971*	*1.136*	*0.972*	*1.819*	*0.961*	*1.628*	*0.972*	*1.095*
Ratio of Case 2 to Case 1	Max	1.197	0.412	1.259	0.266	1.181	0.518	1.270	0.088	1.307
	Min	0.536	1.309	0.479	1.289	0.132	1.456	0.166	1.292	0.568
Ratio of Case 3 to Case 4	Max	1.067	0.688	1.086	0.565	1.062	0.762	1.089	0.372	1.100
	Min	0.774	1.100	0.736	1.095	0.425	1.139	0.464	1.096	0.794
Ratio of Case 4 to Case 2	Max	0.918	1.714	0.897	2.378	0.923	1.465	0.894	6.166	0.883
	Min	1.432	0.882	1.543	0.888	4.277	0.843	3.512	0.887	1.380

Table E4(2)-M7 Strength I Comparison		Moment (kip-ft) Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.5L	Pier 4	Span 5 - 0.6L
Case 1: 1.25 DL + 1.75 LL without SE	Max	7301	-4924	6084	-4299	6363	-4332	6117	-4918	7277
	Min	2088	-10281	1044	-9909	1216	-9923	1065	-10224	2118
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	8266	-2506	7116	-1613	7123	-2558	7201	-1178	8773
	Min	1200	-12495	465	-11896	126	-13062	123	-12307	1285
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	8146	-2808	6987	-1949	7028	-2779	7066	-1645	8586
	Min	1311	-12218	537	-11647	262	-12669	241	-12046	1389
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	7784	-3715	6600	-2956	6743	-3445	6659	-3048	8025
	Min	1644	-11388	755	-10902	671	-11492	594	-11265	1701
Ratio of Case 3 to Case 1	Max	1.116	0.570	1.148	0.453	1.105	0.642	1.155	0.335	1.180
	Min	0.628	1.188	0.515	1.175	0.215	1.277	0.226	1.178	0.656
Ratio of Case 3 to Case 2	Max	*0.985*	1.121	*0.982*	1.208	*0.987*	1.087	*0.981*	1.397	*0.979*
	Min	1.092	*0.978*	1.156	*0.979*	2.085	*0.970*	1.956	*0.979*	1.081
Ratio of Case 2 to Case 1	Max	1.132	0.509	1.170	0.375	1.120	0.590	1.177	0.239	1.206
	Min	0.575	1.215	0.445	1.200	0.103	1.316	0.116	1.204	0.606
Ratio of Case 3 to Case 4	Max	1.047	0.756	1.059	0.659	1.042	0.807	1.061	0.540	1.070
	Min	0.798	1.073	0.712	1.068	0.390	1.102	0.406	1.069	0.816
Ratio of Case 4 to Case 2	Max	0.942	1.482	0.927	1.833	0.947	1.347	0.925	2.588	0.915
	Min	1.370	0.911	1.623	0.916	5.340	0.880	4.823	0.915	1.325

Example E4(2)
Five-Span Bridge, Span Lengths 120 ft, 140 ft, 140 ft, 140 ft, and 120 ft, Girder Spacing 11 ft-2 in.

Shear Comparison

Table E4(2)-S1		Shear (kip)									
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Unfactored DL shear (No settlement)		112.6	-190.3	180.6	-175.9	178.1	-178.3	175.5	-180.9	189.3	-112.6
Unfactored LL shear	+ve	125.0	4.6	147.1	18.3	149.4	19.0	148.3	17.6	145.4	15.8
	-ve	-16.3	-145.4	-18.0	-146.5	-19.4	-147.6	-18.5	-146.8	-4.5	-124.8
Unfactored effect of 1 in. settlement at Abutment 1		-7.7	-7.7	8.4	8.4	-2.4	-2.4	0.7	0.7	-0.2	-0.2
Unfactored effect of 1 in. settlement at Pier 1		12.2	12.2	-16.1	-16.1	7.3	7.3	-2.0	-2.0	0.5	0.5
Unfactored effect of 1 in. settlement at Pier 2		-5.8	-5.8	12.6	12.6	-12.6	-12.6	6.3	6.3	-1.6	-1.6
Unfactored effect of 1 in. settlement at Pier 3		1.6	1.6	-6.3	-6.3	12.6	12.6	-12.6	-12.6	5.7	5.7
Unfactored effect of 1 in. settlement at Pier 4		-0.5	-0.5	2.1	2.1	-7.3	-7.3	16.2	16.2	-12.2	-12.2
Unfactored effect of 1 in. settlement at Abutment 2		0.2	0.2	-0.7	-0.7	2.4	2.4	-8.5	-8.5	7.8	7.8

Predicted Unfactored Total Settlements, S_t
Based on an appropriate owner-approved and calibrated method.

Table E4(2)-S2					
Predicted Unfactored Total Settlements, S_t (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.90	1.50	1.80	1.00	2.30	1.40

Estimated Unfactored Relevant Settlements, S_{tr}
Should be calculated based on the site-specific soil conditions and loads at different stages of the bridge. Assumed as 50% of S_t for this example.

Table E4(2)-S3					
Estimated Unfactored Relevant Settlements, S_{tr} (in.)					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.45	0.75	0.90	0.50	1.15	0.70

Factored Relevant Settlements, S_f
This example is based on load factor γ_{SE} = 1.75

Table E4(2)-S4					
Factored Relevant Settlements, S_f (in.) using γ_{SE} = 1.75					
Abutment 1	Pier 1	Pier 2	Pier 3	Pier 4	Abutment 2
0.79	1.31	1.58	0.88	2.01	1.23

Table E4(2)-S5		Shear (kip)									
		Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Unfactored DL shear (No settlement)		112.6	-190.3	180.6	-175.9	178.1	-178.3	175.5	-180.9	189.3	-112.6
Unfactored LL shear	+ve	125.0	4.6	147.1	18.3	149.4	19.0	148.3	17.6	145.4	15.8
	-ve	-16.3	-145.4	-18.0	-146.5	-19.4	-147.6	-18.5	-146.8	-4.5	-124.8
Effect of unfactored S_{tr} at Abutment 1		-3.5	-3.5	3.8	3.8	-1.1	-1.1	0.3	0.3	-0.1	-0.1
Effect of unfactored S_{tr} at Pier 1		9.1	9.1	-12.1	-12.1	5.5	5.5	-1.5	-1.5	0.4	0.4
Effect of unfactored S_{tr} at Pier 2		-5.2	-5.2	11.4	11.4	-11.3	-11.3	5.7	5.7	-1.4	-1.4
Effect of unfactored S_{tr} at Pier 3		0.8	0.8	-3.1	-3.1	6.3	6.3	-6.3	-6.3	2.8	2.9
Effect of unfactored S_{tr} at Pier 4		-0.6	-0.6	2.4	2.4	-8.4	-8.4	18.6	18.6	-14.1	-14.1
Effect of unfactored S_{tr} at Abutment 2		0.1	0.1	-0.5	-0.5	1.7	1.7	-6.0	-6.0	5.4	5.4
Total unfactored effect of S_{tr} at all supports	+ve	*10*	*10*	*18*	*18*	*13*	*13*	*25*	*25*	*9*	*9*
	-ve	*-9*	*-9*	*-16*	*-16*	*-21*	*-21*	*-14*	*-14*	*-16*	*-16*
Total factored effect of settlement using γ_{SE} = 1.00 and S_t	+ve	*20*	*20*	*35*	*35*	*27*	*27*	*49*	*49*	*17*	*17*
	-ve	*-19*	*-19*	*-31*	*-31*	*-42*	*-42*	*-28*	*-28*	*-31*	*-31*
Total factored effect of settlement using γ_{SE} = 1.75 and S_{tr}	+ve	*18*	*18*	*31*	*31*	*24*	*24*	*43*	*43*	*15*	*15*
	-ve	*-16*	*-16*	*-27*	*-27*	*-36*	*-36*	*-24*	*-24*	*-27*	*-27*
Total factored effect of settlement using γ_{SE} = 1.00 and S_{tr}	+ve	*10*	*10*	*18*	*18*	*13*	*13*	*25*	*25*	*9*	*9*
	-ve	*-9*	*-9*	*-16*	*-16*	*-21*	*-21*	*-14*	*-14*	*-16*	*-16*

Table E4(2)-S6 Service I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Case 1: 1.0 DL + 1.0 LL without SE	Max	238	-186	328	-158	328	-159	324	-163	335	-97
	Min	96	-336	163	-322	159	-326	157	-328	185	-237
Case 2: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	258	-166	363	-123	354	-132	373	-114	352	-79
	Min	78	-354	131	-354	117	-367	129	-355	154	-269
Case 3: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	255	-168	358	-127	351	-136	367	-120	350	-82
	Min	80	-352	135	-350	122	-362	133	-352	158	-265
Case 4: 1.0 DL + 1.0 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	248	-176	345	-140	341	-146	348	-139	343	-88
	Min	87	-345	147	-338	138	-347	143	-341	169	-253
Ratio of Case 3 to Case 1	Max	1.074	0.905	1.094	0.805	1.072	0.852	1.133	0.737	1.045	0.843
	Min	0.832	1.048	0.831	1.085	0.771	1.112	0.846	1.074	0.853	1.115
Ratio of Case 3 to Case 2	Max	0.990	1.015	0.988	1.036	0.991	1.025	0.984	1.054	0.994	1.027
	Min	1.030	0.993	1.030	0.989	1.044	0.986	1.027	0.990	1.025	0.986
Ratio of Case 2 to Case 1	Max	1.085	0.892	1.107	0.778	1.082	0.831	1.152	0.699	1.052	0.821
	Min	0.808	1.055	0.807	1.097	0.738	1.128	0.825	1.084	0.832	1.131
Ratio of Case 3 to Case 4	Max	1.030	0.957	1.038	0.906	1.030	0.931	1.053	0.867	1.019	0.926
	Min	0.920	1.020	0.920	1.035	0.887	1.045	0.928	1.030	0.931	1.046
Ratio of Case 4 to Case 2	Max	0.961	1.061	0.952	1.143	0.962	1.101	0.934	1.215	0.975	1.109
	Min	1.119	0.974	1.120	0.956	1.178	0.943	1.106	0.961	1.101	0.942

Table E4(2)-S7 Strength I Comparison		Shear (kip) Right of Abutment 1	Left of Pier 1	Right of Pier 1	Left of Pier 2	Right of Pier 2	Left of Pier 3	Right of Pier 3	Left of Pier 4	Right of Pier 4	Left of Abutment 2
Case 1: 1.25 DL + 1.75 LL without SE	Max	360	-230	483	-188	484	-190	479	-195	491	-113
	Min	112	-492	194	-476	189	-481	187	-483	229	-359
Case 2: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_t)	Max	380	-210	518	-153	511	-163	528	-146	508	-96
	Min	94	-511	163	-508	147	-523	159	-511	198	-390
Case 3: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.75 and S_{tr})	Max	377	-212	514	-157	508	-166	522	-152	506	-98
	Min	96	-508	167	-504	152	-518	163	-507	201	-386
Case 4: 1.25 DL + 1.75 LL + g_{SE} SE (use g_{SE} = 1.00 and S_{tr})	Max	370	-220	501	-170	497	-176	503	-171	500	-104
	Min	103	-502	178	-492	168	-502	173	-497	213	-375
Ratio of Case 3 to Case 1	Max	1.049	0.923	1.063	0.837	1.049	0.876	1.090	0.780	1.031	0.866
	Min	0.856	1.033	0.858	1.058	0.807	1.076	0.871	1.050	0.881	1.076
Ratio of Case 3 to Case 2	Max	0.993	1.012	0.992	1.029	0.993	1.021	0.988	1.042	0.996	1.023
	Min	1.025	0.995	1.024	0.992	1.035	0.990	1.022	0.993	1.020	0.990
Ratio of Case 2 to Case 1	Max	1.056	0.912	1.073	0.813	1.055	0.858	1.103	0.748	1.035	0.846
	Min	0.835	1.038	0.838	1.066	0.780	1.086	0.853	1.057	0.864	1.087
Ratio of Case 3 to Case 4	Max	1.020	0.966	1.026	0.923	1.020	0.943	1.037	0.892	1.013	0.938
	Min	0.933	1.014	0.934	1.024	0.907	1.031	0.940	1.021	0.945	1.031
Ratio of Case 4 to Case 2	Max	0.974	1.048	0.966	1.115	0.974	1.083	0.953	1.168	0.983	1.091
	Min	1.099	0.982	1.097	0.969	1.141	0.960	1.086	0.973	1.079	0.960

Appendix F
Proposed Modifications to *AASHTO LRFD Bridge Design Specifications* Section 3

Appendix F. Proposed Modifications to *AASHTO LRFD Bridge Design Specifications* Section 3

This appendix contains the original proposed modifications to the *AASHTO LRFD Bridge Design Specifications* Section 3. The proposed modifications were developed as part of the deliverables for the first edition of this report in 2016 and are shown as they would have appeared in the then-current edition (that is, 7th Edition in 2015) of the *AASHTO LRFD* and not the 8th Edition in 2017 as shown in Chapter 3. These proposed modifications were used by AASHTO T-5 and T-15 committees as part of their deliberations during the balloting processes. The reader should consult to the latest version of the *AASHTO LRFD Bridge Design Specifications* for final (actual) modifications.

Proposed Modifications to Section 3

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3.3—NOTATION

3.3.1—General

A = plan area of ice floe (ft^2); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)

A_{df} = factored angular distortion (rad) (Appendix C3)

AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)

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L = perimeter of pier (ft); length of soil reinforcing elements in an MSE wall (ft); length of footing (ft); expansion length (in.) (3.9.5) (3.11.5.8) (3.11.6.3) (3.12.2.3)

L_S = span length (ft) (Appendix C3)

$LLDF$ = live load distribution factor as specified in Table 3.6.1.2.6-1a (3.6.1.2.6b)

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r = radius of pier nose (ft) (C3.9.2.3)

S = Settlement (ft) (Appendix C3)

S_{DS} = horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor (3.10.4.2)

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δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between the far and near corners of a footing measured from the point on the wall under consideration (rad) ; foundation deformation (rad. or in.) (C3.9.5) (3.11.5.3) (3.11.6.2) (Appendix C3)

δ_f = factored deformation (rad. or in.) (Appendix C3)
η_i = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)

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3.4—LOAD FACTORS AND COMBINATIONS

3.4.1—Load Factors and Load Combinations

The total factored force effect shall be taken as:

$$Q = \sum \eta_i \gamma_i Q_i \tag{3.4.1-1}$$

C3.4.1

The background for the load factors specified herein, and the resistance factors specified in other Sections of these Specifications is developed in Nowak (1992).

The evaluation of overall stability of retained fills, as well as earth slopes with or without a shallow or deep foundation unit should be investigated at the service limit state based on the Service I Load Combination and an appropriate resistance factor as specified in Article 11.5.6 and Article 11.6.2.3.

The investigation of foundation settlement shall proceed using the provisions of Article 10.6.2.4 using the load factor, γ_{SE}, specified in Table 3.4.1-4.

For structural plate box structures complying with the provisions of Article 12.9, the live load factor for the vehicular live loads *LL* and *IM* shall be taken as 2.0.

Applying these criteria for the evaluation of the sliding resistance of walls:

- The vertical earth load on the rear of a cantilevered retaining wall would be multiplied by γ_{pmin} (1.00) and the weight of the structure would be multiplied by γ_{pmin} (0.90) because these forces result in an increase in the contact stress (and shear strength) at the base of the wall and foundation.

- The horizontal earth load on a cantilevered retaining wall would be multiplied by γ_{pmax} (1.50) for an active earth pressure distribution because the force results in a more critical sliding force at the base of the wall.

Similarly, the values of γ_{pmax} for structure weight (1.25), vertical earth load (1.35) and horizontal active earth pressure (1.50) would represent the critical load combination for an evaluation of foundation bearing resistance.

Water load and friction are included in all strength load combinations at their respective nominal values.

The load factor for temperature gradient, γ_{TG}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{TG} may be taken as:

- 0.0 at the strength and extreme event limit states,
- 1.0 at the service limit state when live load is not considered, and
- 0.50 at the service limit state when live load is considered.

The effects of the foundation deformation on the bridge superstructure, retaining walls, or other load bearing structures shall be evaluated at applicable strength and service limit states using the provisions of Article 10.5.2.2 and the settlement load factor (γ_{SE}) specified in Table 3.4.1-4. For all bridges, stiffness should be appropriate to the considered limit state. Similarly, the effects of continuity with the substructure should be considered. In assessing the structural implications of foundation deformations of concrete bridges, the determination of the stiffness of the bridge components should consider the effects of cracking, creep, and other inelastic responses.

~~The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{SE}, may be taken as 1.0.~~ Load combinations which include settlement shall also be applied without settlement. As specified in Article 3.12.6, subsets of the settlements shall be considered when determining extreme combinations of force effects.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS \quad (3.4.1\text{-}2)$$

For creep and shrinkage, the specified nominal values should be used. For friction, settlement, and water loads, both minimum and maximum values need to be investigated to produce extreme load combinations.

The load factor for temperature gradient should be determined on the basis of the:

- Type of structure, and
- Limit state being investigated.

Open girder construction and multiple steel box girders have traditionally, but perhaps not necessarily correctly, been designed without consideration of temperature gradient, i.e., $\gamma_{TG} = 0.0$.

The values of γ_{SE} in Table 3.4.1.-4 are based on a target reliability index of 0.50 which assume that the effect of irreversible foundation deformations on the bridge superstructure will be reversed by intervention, e.g., shimming, jacking, etc. If intervention to relieve the superstructure is not practical or desirable for a given bridge type, then larger values of γ_{SE} consistent with target reliability index of 1.00 or larger shall be considered based on Kulicki et al., (2015) and Samtani and Kulicki (2016).

An owner may choose to use a local method that provides better estimation of foundation movement for local geologic conditions compared to methods noted in Section 10. In such cases, the owner will have to calibrate the γ_{SE} value for the local method using the procedures described in Kulicki et al., (2015) and Samtani and Kulicki (2016).

The application of γ_{SE} is illustrated in the flowchart in Appendix C3. The recommended procedure is to factor the deformations and evaluate the effect on the structure using the factored deformations. For example, if a structural analysis of factored deformations is performed, the resulting forces effects are already factored and these results are used directly in the appropriate load combinations in Table 3.4.1-1. The γ_{SE} in Table 3.4.1-1 does not indicate a second application of γ_{SE}. Rather it indicates that the force effects from the factored deformations are to be used in the indicated load combinations.

The value of γ_{SE}=1.00 for consolidation (long-term settlement time-dependent) settlement assumes that the estimation of consolidation settlement is based on appropriate laboratory and field tests to determine parameters (rather than correlations with index properties of soils) in the consolidation settlement equations in Article 10.6.2.4.3.

The value of γ_{SE} for soil-structure interaction methods in Table 3.4.1-4 for estimation of lateral deformations may be increased to larger than 1.0 based on local experience and calibration using procedures described in Kulicki et al., 2015) and Samtani and Kulicki (2016).

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

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

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
using I_g	0.5	0.5
using $I_{effectuve}$	1.0	1.0
Steel Substructures	1.0	1.0

Table 3.4.1-4—Load Factors for Permanent Loads Due to Foundation Deformations, γ_{SE}

Foundation Deformation and Deformation Estimation Method	*SE*
Immediate Settlement	
• Hough method	1.00
• Schmertmann method	1.25
• Local method	*
Consolidation settlement	1.00
Lateral Deformation	
• Soil-structure interaction method (P-y or Strain Wedge)	1.00.
• Local method	*
*To be determined by the owner based on local geologic conditions.	

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3.16—REFERENCES

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APPENDIX C3—CONSIDERATION OF FOUNDATION DEFORMATIONS IN BRIDGE DESIGN

Figure C3-1 shows a flow chart to consider foundation deformation in the bridge design process. The flow chart has two distinct parts, the left and right. The left part provides the outline of the process that a bridge designer may use without explicit consideration of foundation deformations other than what is required in the 7th Edition of *AASHTO LRFD*, i.e. without considering the method-specific load factor, γ_{SE}, the construction-point concept or the δ-0 concept. For convenience this will be called the "legacy loop". The right part provides the recommended procedure to factor the deformations and evaluate the effect on the structure using the factored deformations. The sequence of activities in the deformation loop is based on the discussions in Samtani and Kulicki (2016) which includes the method-specific load factor, γ_{SE}, the construction-point concept or the δ-0 concept. For convenience this will be called the "refined (deformation) loop". The flow chart applies to any type of foundation deformation and hence the symbol δ is used for deformations. If the flow chart is used for settlement, then symbol "*S*" may be substituted for δ.

It is not the intention of the illustrated design process to universally require additional design effort beyond that required by the 7th edition of *AASHTO LRFD*, or approved owner policies that take advantage of well documented past geotechnical practice. For example, if the geomaterials at a site are well understood and past experience shows that a deep foundation is the best option or that a given service bearing pressure results in an acceptable foundation deformations with minimal structural or geometric consequences, then the decision to base a new design on legacy practices is a viable option. If, on the other hand, site conditions are not within past successful practice, there is a desire to consider possible economies of design that alter the experience base, or the structure requires more careful consideration of possible foundation deformations, then the additional provisions embodied in the refined (deformation) loop will result in a more thorough assessment of the implications of foundation deformations and the associated impact on the design and economy of the bridge.

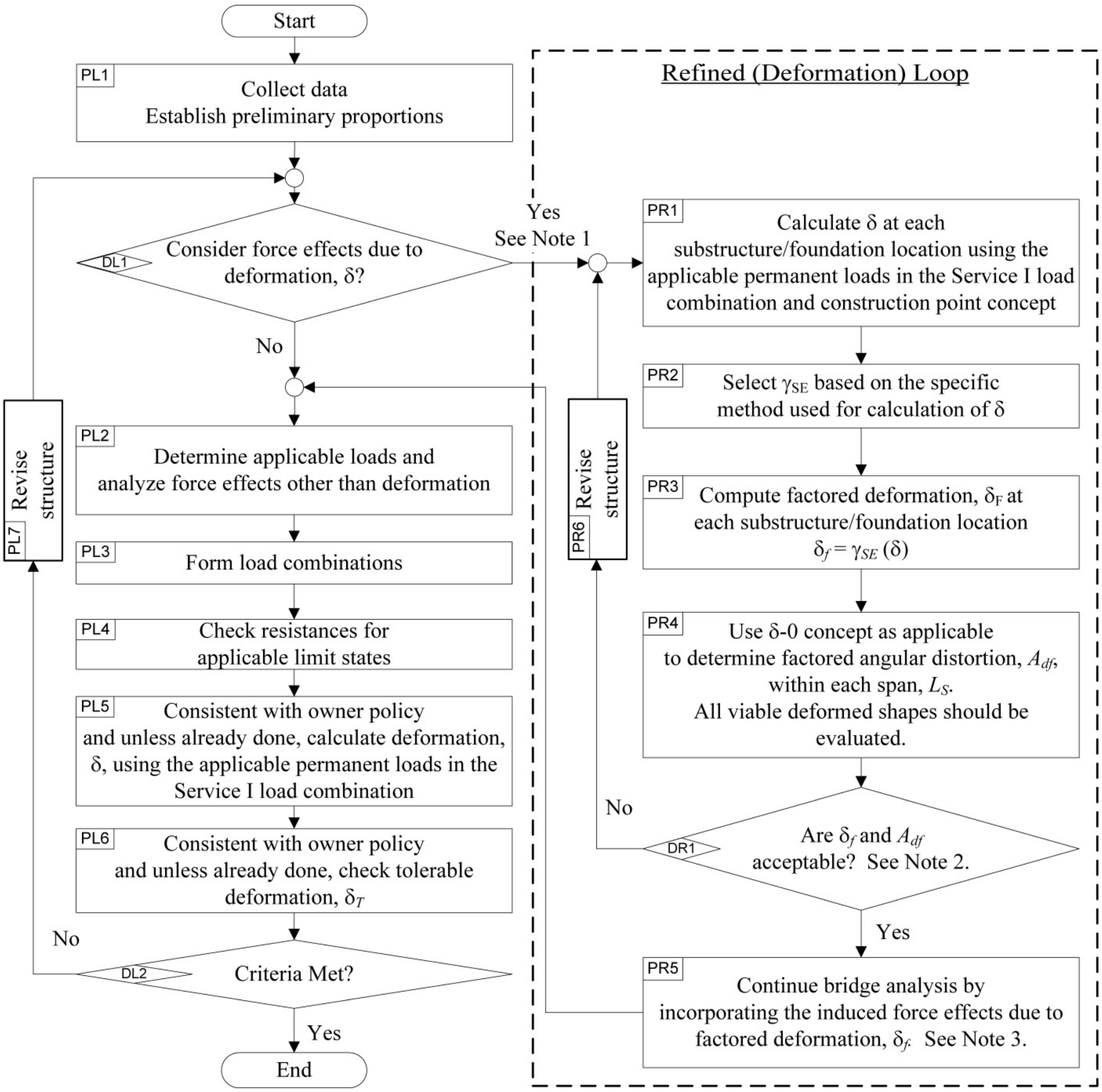
Three notes are provided in the flow chart to include additional guidance for designer.

Some of the key points associated with the flow chart are as follows:

1. The process ("P") related steps are indicated in rectangular boxes (▭). In the left ("L") part there are six process boxes labeled PL1 to PL6. In the right ("R") part there are five process boxes labeled PR1 to PR5.
2. The decision ("D") related steps are indicated in diamond boxes (◇). In the left part there are two decision boxes labeled DL1 and DL2. The right part contains one decision box labeled DR1.
3. The left and right parts are connected at two levels. The first connection is established when a bridge designer decides to proceed with either the legacy or refined (deformation) loop in box DL1. The second connection is established after box PR5 once the designer has determined a favorable resolution of "Yes" to the decision in box DR1.
4. If the resolution to either box DL2 is "No," then the structure is revised and the flow chart is re-entered at box DL1. Likewise, if the resolution at DR1 is "No" the structure is revised and the flowchart is re-entered at box PR1
5. If the answer is "No" at box DL1, then the designer goes through the process provided in boxes PL2 to PL6 using the legacy approach as follows:
   - In box PL2 structural analysis proceeds without use of the construction-point or δ-0 concepts as they are not incorporated into the legacy approach. Consideration of foundation deformations is consistent with the owner's implementation of the 7th edition of *AASHTO LRFD*.
   - Box PL3 indicates use of Table 3.4.1-1 as applicable to the situation at hand. Depending on the owner's policies the values of γ_{SE} will effectively be zero or unity. In this case, the deformation may be evaluated based on past local experience with similar structures.
6. If the answer is "Yes" at box DL1, then the designer goes through the process provided in boxes PR1 to PR5, using the refined (deformation) approach. Note 1 is provided as guidance about entering the right side. The design proceeds as follows:
   - After the calculation of δ for the indicated loads in box PR1 and adjusting them for the construction-point concept they are scaled (factored) as indicated in box PR3 using the method-specific values of $\square_{SE}$ determined in box PR2.
   - These factored deformations, δ_f, are used along with the δ-0 concept to calculate the factored angular distortions, A_{df} in box PR4.
   - In box DR1 the values of δ_f and A_{df} are compared to the applicable criteria. These criteria are geometric, not structural. Note 2 providers additional guidance.

- If the results are not acceptable the structure is revised and the design process returns to box PR1 to evaluate the modified structure.
- If the results at box DR1 are acceptable the structural force effects from the factored deformations, δ_f, are calculated and are carried into the remaining steps of the legacy loop. Note 3 is vital to the correct formulation of load combinations using Table 3.4.1-1 in box PL5.

7. The "Criteria" in box DL2 can include any criteria related to bridge design such as deck grades, joint distress, crack control, moment and shear resistance.
8. In boxes PL5 and PL6, the phrase "unless already done" acknowledges the possibility that the actions in these boxes may already have been performed by a designer who is entering these boxes after completing the right part of the flow chart.
9. If all structural and geometric criteria are satisfied in box DL2 the design is satisfactory; if not, the structure is modified and the design process returns to box DL1.



Note 1: It may be efficient to run some early design iterations without including this loop until the proportions of the bridge are well developed, and then include this loop to consider the force effects from differential deformations.

Note 2: Compare A_{df} to permissible angular distortion criteria and δ_f to permissible values at abutment interfaces and within spans in terms of vertical clearance under bridge. Guidance in Article 10.5.2 may be used to establish permissible values. Owner may establish other permissible values.

Note 3: Note that the γ_{SE} is used to factor the deformations as shown in this flow chart. γ_{SE} also appears in Table 3.4.1-1 (Load Combinations and Load Factors). This does not imply a second application of γ_{SE} in the load combinations but rather it is an acknowledgement that the deformations have already been factored. Use of the factored deformations in a structural analysis program ensures that the output is factored value.

Figure C3-1—Foundation Deformation Procedure Flow Chart (Samtani and Kulicki, 2016)

Appendix G
Proposed Modifications to *AASHTO LRFD Bridge Design Specifications* Section 10

Appendix G. Proposed Modifications to *AASHTO LRFD Bridge Design Specifications* Section 10

This appendix contains the original proposed modifications to the *AASHTO LRFD Bridge Design Specifications* Section 10. The proposed modifications were developed as part of the deliverables for the first edition of this report in 2016 and are shown as they would have appeared in the then-current edition (that is, 7th Edition in 2015) of the *AASHTO LRFD* and not the 8th Edition in 2017 as shown in Chapter 3. These proposed modifications were used by the AASHTO T-15 committee as part of its deliberations during the balloting processes. The reader should consult to the latest version of the *AASHTO LRFD Bridge Design Specifications* for final (actual) modifications.

Proposed Modifications to Section 10

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10.3—NOTATION

A_{ct} = cross-sectional area of steel casing considering reduction for threads (in.^{2}) (10.9.3.10.3a)
A_{df} = factored angular distortion (10.5.2.2.2)
A_g = cross-sectional area of grout within micropile (in.^{2}) (10.9.3.10.3a)
•
•
B = footing width; pile group width; pile diameter (ft) (10.6.1.3) (10.7.2.3.2) (10.7.2.4)
B_f = least width of footing (10.6.2.4.2b)
B' = effective footing width (ft) (10.6.1.3)
C_1 = correction factor to incorporate the effect of strain relief due to embedment (10.6.2.4.2b)
C_2 = correction factor to incorporate time-dependent (creep) increase in settlement for t (years) after construction (10.6.2.4.2b)
C_α = secondary compression index, void ratio definition (dim) (10.4.6.3)
•
•
d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation (dim) (10.6.3.1.2a)
E = modulus of elasticity of pile material (ksi) (10.7.3.8.2); elastic modulus of layer i based on guidance provided in Table C10.4.6.3-1 (10.6.2.4.2b)
E_d = developed hammer energy (ft-lb) (10.7.3.8.5)
•
•
I_w = weak axis moment of inertia for a pile (ft^4) (10.7.3.13.4)
I_z = strain influence factor from Figure 10.6.2.4.2c-1a
i_c, i_q, i_γ = load inclination factors (dim) (10.6.3.1.2a)
•
•
L_b = micropile bonded length (ft) (10.9.3.5.2)
L_f = length of footing (10.6.2.4.2b)
L_i = depth to middle of length interval at the point considered (ft) (10.7.3.8.6g)
L_p = micropile casing plunge length (ft) (10.9.3.10.4)
L_s = bridge span length over which A_{df} is computed (10.5.2.2.2)
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S_e = elastic settlement (ft) (10.6.2.4.1)
S_f = foundation relevant total settlement (ft) (10.5.2.2.2)
S_s = secondary settlement (ft) (10.6.2.4.1)
S_t = total settlement (ft) (10.6.2.4.1)

S_{ta} = total foundation settlement using permanent loads in the Service I load combination (ft) (10.5.2)
S_{tp} = total foundation settlement using permanent loads prior to construction of bridge superstructure in the Service I load combination (ft) (10.5.2.2.2)
S_{tr} = relevant total foundation settlement defined as $S_{ta} - S_{tp}$ (10.5.2.2.2)
S_u = undrained shear strength (ksf) (10.4.6.2.2)
•
•
T = time factor (dim) (10.6.2.4.3)
t = time for a given percentage of one-dimensional consolidation settlement to occur (yr) (10.6.2.4.3); time t from completion of construction to date under consideration for evaluation of C_2 (yrs) (10.6.2.4.2b)
t_1, t_2 = arbitrary time intervals for determination of secondary settlement, S_s (yr) (10.6.2.4.3)
•
•
W_{T1} = vertical movement at the head of the drilled shaft (in.) (C10.8.3.5.4d)
X = width or smallest dimension of pile group (ft) (10.7.3.9); a factor used to determine the value of elastic modulus (10.6.2.4.2b)
Y = length of pile group (ft) (10.7.3.9)
•
•
γ_p = load factor for downdrag (C10.7.3.7)
γ_{SE} = load factor for settlement (10.5.2.2.2)
ΔH_i = elastic settlement of layer i (ft) (10.6.2.4.2)
Δ = differential settlement between two bridge support elements spaced at a distance of L_s (ft) (10.5.2.2)
Δ_f = factored differential settlement (10.5.2.2.2)
Δp = net uniform applied stress (load intensity) at the foundation depth (Figure 10.6.2.4.2c-1b)

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10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General

The limit states shall be as specified in Article 1.3.2; foundation-specific provisions are contained in this Section.

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

10.5.2—Service Limit States

10.5.2.1—General

Foundation design at the service limit state shall include:

- Settlements,
- Horizontal movements,
- Overall stability, and
- Scour at the design flood.

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

Bearing resistance estimated using the presumptive allowable bearing pressure for spread footings, if used, shall be applied only to address the service limit state.

C10.5.2.1

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. Article 2.5.2.3 requires jacking provisions for these bridges.

The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. The Owner may establish more stringent criteria.

The foundation movements should be translated to the deck elevation to evaluate the effect of such movements on the superstructure. In this process, deformations of the substructure, i.e., elements between foundation and superstructure, should be added to foundation deformations as appropriate.

The design flood for scour is defined in Article 2.6.4.4.2, and is specified in Article 3.7.5 as applicable at the service limit state.

Presumptive bearing pressures were developed for use with working stress design. These values may be used for preliminary sizing of foundations, but should generally not be used for final design. If used for final design, presumptive values are only applicable at service limit states.

10.5.2.2—Tolerable Movements and Movement Criteria

10.5.2.2.1—General

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal, and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses, or by consideration of both.

Foundation settlement shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependent consolidation settlements.

All applicable service limit state load combinations in Table 3.4.1-1 shall be used for evaluating horizontal movement and rotation of foundations.

C10.5.2.2.1

Experience has shown that bridges can and often do accommodate more movement and/or rotation than traditionally allowed or anticipated in design. Creep, relaxation, and redistribution of force effects accommodate these movements. Some studies have been made to synthesize apparent response. These studies indicate that angular distortions between adjacent foundations greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be permitted in settlement criteria (Moulton et al., 1985; DiMillio, 1982; Barker et al., 1991; Samtani et al. 2010). Other angular distortion limits may be appropriate after consideration of:

- cost of mitigation through larger foundations, realignment or surcharge,
- rideability,
- vertical clearance,
- tolerable limits of deformation of other structures associated with a bridge, e.g., approach slabs, wingwalls, pavement structures, drainage grades, utilities on the bridge, etc.
- roadway drainage,
- aesthetics, and
- safety.

Rotation movements should be evaluated at the top of the substructure unit in plan location and at the deck elevation.

Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement, with consideration of the column length and stiffness.

Tolerance of the superstructure to lateral movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects.

10.5.2.2.2—Factored Relevant Total Settlement, S_f, and Factored Angular Distortion, A_{df}

In lieu of owner supplied provisions, the following steps should be followed to estimate and use practical values of factored settlement, S_f, and factored angular distortion, A_{df} in the bridge design process as shown in Appendix C3 of Section 3:

1. At each support element, compute factored relevant total foundation settlement for the assumed foundation type (e.g., spread footings, driven piles, drilled shafts, etc.) as follows:
   a. Determine the total foundation settlement, S_{ta}, using all applicable permanent loads in the Service I load combination.

C10.5.2.2.2

Determination of relevant total settlement should include consideration of how and when settlement occurs during construction process and uncertainty of the settlement itself. These two factors are addressed by the construction-point concept and S_f-0 concept in this article, respectively.

Foundation deformations should not be estimated as if a weightless bridge structure is instantaneously set into place and all the loads are applied at the same time. In reality, loads are applied gradually as construction proceeds. Consequently, foundation deformations also occur gradually as construction proceeds. There are several critical construction points or stages during construction that should be evaluated separately by the

b. Determine the total foundation settlement, S_{tp}, prior to construction of bridge superstructure. This settlement would generally be as a result of all applicable substructure loads computed in accordance with permanent loads in the Service I load combination.
c. Determine relevant total settlement, S_{tr} as $S_{tr} = S_{ta} - S_{tp}$.
d. Determine the factored relevant total settlement, S_f, using Eq. 10.5.2.2.2-1

$$S_f = \gamma_{SE}(S_{tr}) \qquad (10.5.2.2.2\text{-}1)$$

where:

γ_{SE} = *SE* load factor value selected from Table 3.4.1-4 based on the method used to estimate the settlement.

designer. Figure C10.5.2.2-1 shows the critical construction stages (W, X, Y, and Z) and their associated load-settlement behavior for the case of a pier and vertical loads. 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 X and Z are the most relevant. Formulation of settlements in a manner shown in Figure C10.5.2.2-1b permits an assessment of settlements up to that point that can affect the bridge superstructure. Although Figure C10.5.2.2-1 illustrates the construction-point concept fort the case of a pier, vertical loads and settlements (vertical deformation), the concepts apply to other elements of bridge structure (e.g., abutments), load types (shears, moments, etc.) and deformation types (lateral movements, rotations, etc.).

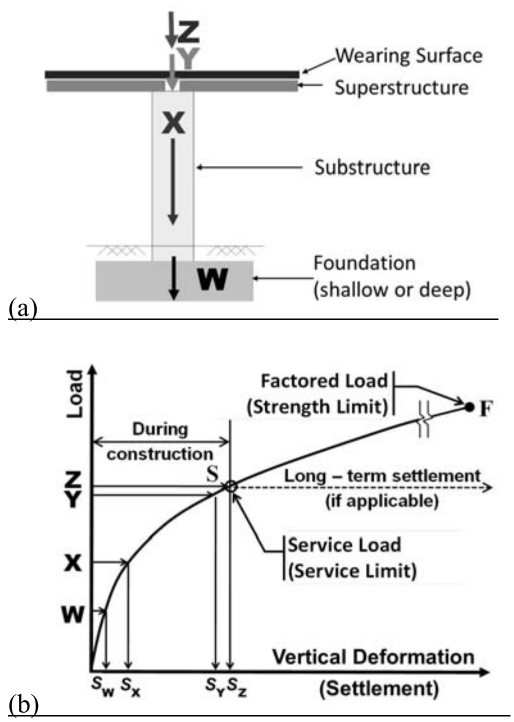


Figure C10.5.2.2-1. Construction-point concept for a bridge pier. (a) Identification of critical construction points, (b) conceptual load-deformation pattern for a given foundation (Kulicki, et. al, 2015; Samtani and Kulicki, 2016).

Long-term settlements as shown by the horizontal dashed line corresponding to the total construction load (Z) in Figure C10.5.2.2-1 shall be included as appropriate.

The contribution of deformations in the substructure columns to the angular distortions at the deck elevation should be considered.

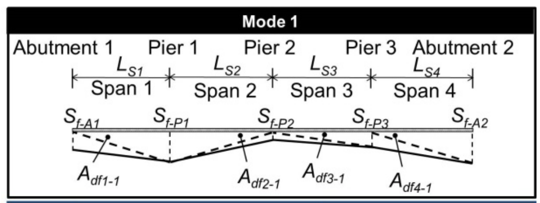
2. Compute the factored angular distortion within each span using the S_f-0 concept. At a given support element assume that the actual settlement could be as large as the factored relevant total settlement calculated by the chosen method, S_f.. At the same time, assume that an adjacent support element does not settle at all. Thus, the factored differential settlement, Δ_f, within a given bridge span is equal to the larger of the factored relevant total settlement at each of two supports of a bridge span. Compute factored angular distortion, A_{df}, as the ratio of the factored differential settlement, Δ_f, to the span length, L_s. Express A_{df} value in radians.

   All viable deformation shapes should be evaluated.

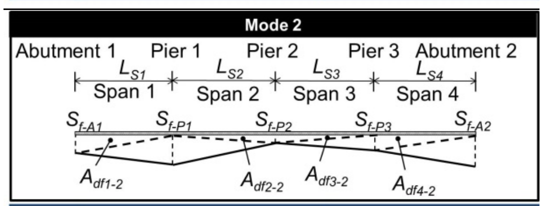
While the angular distortion is generally applied in the longitudinal direction of a bridge, similar analyses should be performed in transverse direction based on consideration of bridge width and stiffness. If the distance between support elements in the transverse direction is less than one-half of the bridge width at that line of support elements then the angular distortion may be computed based on the difference between the factored relevant settlement between the support points rather than the S_f-0 approach.

While all analytical methods for estimating settlements have some degree of uncertainty, 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 that differential settlement, e.g., between an abutment and a pier, or between two adjacent piers. The S-0 concept is used to account for this uncertainty.

A hypothetical 4-span bridge structure with span lengths, L_{s1}, L_{s2}, L_{s3} and L_{s4} is shown in Figure C10.5.2.2-2 to illustrate the application of S_f-0 concept and computation of factored angular distortion. The factored relevant total settlement, S_f, is computed at each support element and the profile of S_f along the bridge is shown by the solid line. In this figure, $S_{f\text{-}A1} < S_{f\text{-}P1} > S_{f\text{-}P2} < S_{f\text{-}P3} < S_{f\text{-}A2}$. As shown, two viable modes of deformation shapes, Mode 1 and Mode 2, are possible. For each of these two modes, the S_f profile assumed for computation of the factored angular distortion, A_{df}, for each span is represented by the dashed lines. The factored angular distortion within each span is computed as shown for each viable mode as shown in Figure C10.5.2.2-2. The symbols are in accordance with $\Delta_{fi\text{-}j}$ and $A_{dfi\text{-}j}$ where i represents the span number (1 to 4) and j represents the mode (1 and 2).



Span	Factored Differential Settlement	Factored Angular Distortion
1	$\Delta_{f1\text{-}1} = S_{f\text{-}P1}$ (assume $S_{f\text{-}A1} = 0$)	$A_{df1\text{-}1} = \Delta_{f1\text{-}1}/L_{S1}$
2	$\Delta_{f2\text{-}1} = S_{f\text{-}P1}$ (assume $S_{f\text{-}P2} = 0$)	$A_{df2\text{-}1} = \Delta_{f2\text{-}1}/L_{S2}$
3	$\Delta_{f3\text{-}1} = S_{f\text{-}P3}$ (assume $S_{f\text{-}P2} = 0$)	$A_{df3\text{-}1} = \Delta_{f3\text{-}1}/L_{S3}$
4	$\Delta_{f4\text{-}1} = S_{f\text{-}A2}$ (assume $S_{f\text{-}P3} = 0$)	$A_{df4\text{-}1} = \Delta_{f4\text{-}1}/L_{S4}$



Span	Factored Differential Settlement	Factored Angular Distortion
1	$\Delta_{f1\text{-}2} = S_{f\text{-}A1}$ (assume $S_{f\text{-}P1} = 0$)	$A_{df1\text{-}2} = \Delta_{f1\text{-}2}/L_{S1}$
2	$\Delta_{f2\text{-}2} = S_{f\text{-}P2}$ (assume $S_{f\text{-}P1} = 0$)	$A_{df2\text{-}2} = \Delta_{f2\text{-}2}/L_{S2}$
3	$\Delta_{f3\text{-}2} = S_{f\text{-}P2}$ (assume $S_{f\text{-}P3} = 0$)	$A_{df3\text{-}2} = \Delta_{f3\text{-}2}/L_{S3}$
4	$\Delta_{f4\text{-}2} = S_{f\text{-}P3}$ (assume $S_{f\text{-}A2} = 0$)	$A_{df4\text{-}2} = \Delta_{f4\text{-}2}/L_{S4}$

Figure C10.5.2.2-2—Computing Factored Angular Distortion, A_{df}, Based on S_f-0 Concept for a hypothetical 4-span Bridge (Samtani and Kulicki, 2016).

If γ_{SE} has already been applied in computation of factored settlement, S_f, as indicated in Step 1d, it should not be applied again during computation of differential settlement or angular distortion.

3. Compare the value of A_{df} within each span and value of S_f at each support element with owner specified total settlement and angular distortion criteria.

   If owner specified angular distortion criteria are not available then use limiting angular distortion criteria noted in C10.5.2.2.1.

The value of S_f should be evaluated with respect to the various factors listed in C10.5.2.2.2.

4. Incorporate S_f and A_{df} in the bridge design process.

The flow chart in Appendix C3 illustrates a typical design process. Note that the flow chart in Appendix C3 uses the symbol δ that is general and applies to any type of deformation. When the flow chart is used for settlement, δ can be substituted with S.

10.5.2.3—Overall Stability

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in Article 11.6.2.3.

10.5.2.4—Abutment Transitions

Vertical and horizontal movements caused by embankment loads behind bridge abutments shall be investigated.

C10.5.2.4

Settlement of foundation soils induced by embankment loads can result in excessive movements of substructure elements. Both short and long term settlement potential should be considered.

Settlement of improperly placed or compacted backfill behind abutments can cause poor rideability and a possibly dangerous bump at the end of the bridge. Guidance for proper detailing and material requirements for abutment backfill is provided in ~~Cheney and Chassie~~ Samtani and Nowatzki (200~~0~~6).

Lateral earth pressure behind and/or lateral squeeze below abutments can also contribute to lateral movement of abutments and should be investigated, if applicable.

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10.6.2.4—Settlement Analyses

10.6.2.4.1—General

Foundation settlements should be estimated using computational methods based on the results of laboratory or insitu testing, or both. The soil parameters used in the computations should be chosen to reflect the loading history of the ground, the construction sequence, and the effects of soil layering.

Both total and differential settlements, including time dependant effects, shall be considered.

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$$S_t = S_e + S_c + S_s \qquad (10.6.2.4.1\text{-}1)$$

where:

S_e = elastic settlement (ft)
S_c = primary consolidation settlement (ft)

S_s = secondary settlement (ft)

C10.6.2.4.1

Elastic, or immediate, settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is estimated as a function of the applied stress beneath a footing or embankment. Elastic settlement is usually small and neglected in design, but where settlement is critical, it is the most important deformation consideration in cohesionless soil deposits and for footings bearing on rock. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked.

In a nearly saturated or saturated cohesive soil, the pore water pressure initially carries the applied stress. As pore water is forced from the voids in the soil by the applied load, the load is transferred to the soil skeleton. Consolidation settlement is the gradual compression of the soil skeleton as the pore water is forced from the voids in the soil. Consolidation settlement is the most important deformation consideration in cohesive soil deposits that possess sufficient strength to safely support a spread footing. While consolidation settlement can occur in saturated cohesionless soils, the consolidation occurs quickly and is normally not distinguishable from the elastic settlement.

Secondary settlement, or creep, occurs as a result of the plastic deformation of the soil skeleton under a constant effective stress. Secondary settlement is of principal concern in highly plastic or organic soil deposits. Such deposits are normally so obviously weak and soft as to preclude consideration of bearing a spread footing on such materials.

The principal deformation component for footings on rock is elastic settlement, unless the rock or included discontinuities exhibit noticeable time-dependent behavior.

To avoid overestimation, relevant settlements should be evaluated using the construction-point concept noted in Samtani and Kulicki (2016). The effect of settlement on superstructure shall be evaluated based on Article 10.5.2.2.

The effects of the zone of stress influence, or vertical stress distribution, beneath a footing shall be considered in estimating the settlement of the footing.

Spread footings bearing on a layered profile consisting of a combination of cohesive soil, cohesionless soil and/or rock shall be evaluated using an appropriate settlement estimation procedure for each layer within the zone of influence of induced stress beneath the footing.

The distribution of vertical stress increase below circular or square and long rectangular footings, i.e., where $L > 5B$, may be estimated using Figure 10.6.2.4.1-1.

For guidance on vertical stress distribution for complex footing geometries, see Poulos and Davis (1974) or Lambe and Whitman (1969).

Some methods used for estimating settlement of footings on sand include an integral method to account for the effects of vertical stress increase variations. For guidance regarding application of these procedures, see Gifford et al. (1987).

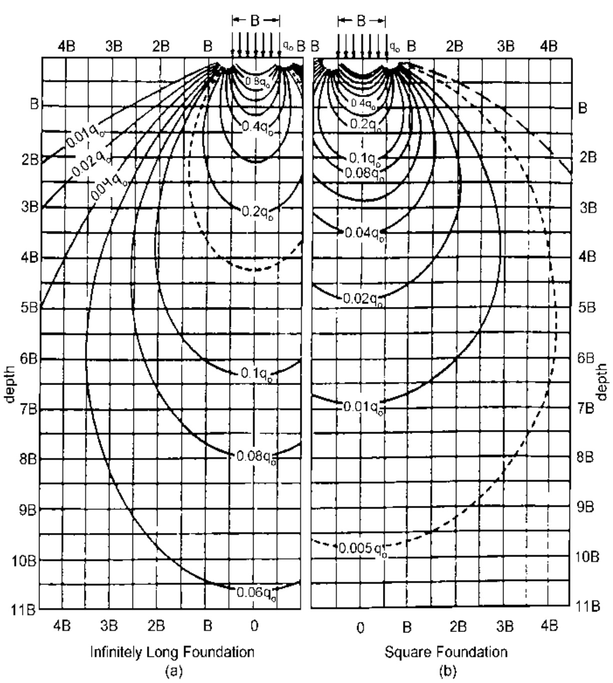


Figure 10.6.2.4.1-1—Boussinesq Vertical Stress Contours for Continuous and Square Footings Modified after Sowers (1979)

10.6.2.4.2—Settlement of Footings on Cohesionless Soils

10.6.2.4.2a—General

The settlement of spread footings bearing on cohesionless soil deposits shall be estimated as a function of effective footing width and shall consider the effects of footing geometry and soil and rock layering with depth.

Settlements of footings on cohesionless soils shall be estimated using elastic theory or empirical procedures.

C10.6.2.4.2a

Although methods are recommended for the determination of settlement of cohesionless soils, experience has indicated that settlements can vary considerably in a construction site, and this variation may not be predicted by conventional calculations.

Settlements of cohesionless soils occur rapidly, essentially as soon as the foundation is loaded. Therefore, the total settlement under the service loads may not be as important as the incremental settlement between intermediate load stages. For example, the total and differential settlement due to loads applied by columns and cross beams is generally less important than the total and differential settlements due to girder placement and casting of continuous concrete decks.

Generally conservative settlement estimates may be obtained using the elastic half-space procedure or the empirical method by Hough. Additional information regarding the accuracy of the methods described herein is provided in Gifford et al. (1987), ~~and~~ Kimmerling (2002) and Samtani and Notwazki (2006). This information, in combination with local experience and engineering judgment, should be used when determining the estimated settlement for a structure foundation, as there may be cases, such as attempting to build a structure grade high to account for the estimated settlement, when overestimating the settlement magnitude could be problematic.

Details of other procedures can be found in textbooks and engineering manuals, including:

- Terzaghi and Peck (1967)
- Sowers (1979)
- U.S. Department of the Navy (1982)
- D'Appolonia (Gifford et al., 1987)—This method includes consideration for over-consolidated sands.
- Tomlinson (1986)
- Gifford et al. (1987)

10.6.2.4.2b—Elastic Half-space Method

The elastic half-space method assumes the footing is flexible and is supported on a homogeneous soil of infinite depth. The elastic settlement of spread footings, in feet, by the elastic half-space method shall be estimated as:

C10.6.2.4.2b

For general guidance regarding the estimation of elastic settlement of footings on sand, see Gifford et al. (1987), ~~and~~ Kimmerling (2002), and Samtani and Notwazki (2006).

The stress distributions used to calculate elastic settlement assume the footing is flexible and supported

$$S_e = \frac{\left[q_o\left(1-\nu^2\right)\sqrt{A'}\right]}{144\ E_s\beta_z} \quad (10.6.2.4.2\underline{b}\text{-}1)$$

where:

q_o = applied vertical stress (ksf)

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

E_s = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of in situ or laboratory tests (ksi)

β_z = shape factor taken as specified in Table 10.6.2.4.2b-1 (dim)

ν = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of ν are not available from the results of in situ or laboratory tests (dim)

Unless E_s varies significantly with depth, E_s should be determined at a depth of about 1/2 to 2/3 of *B* below the footing, where *B* is the footing width. If the soil modulus varies significantly with depth, a weighted average value of E_s should be used.

Table 10.6.2.4.2b-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

on a homogeneous soil of infinite depth. The settlement below a flexible footing varies from a maximum near the center to a minimum at the edge equal to about 50 percent and 64 percent of the maximum for rectangular and circular footings, respectively. The settlement profile for rigid footings is assumed to be uniform across the width of the footing.

Spread footings of the dimensions normally used for bridges are generally assumed to be rigid, although the actual performance will be somewhere between perfectly rigid and perfectly flexible, even for relatively thick concrete footings, due to stress redistribution and concrete creep.

The accuracy of settlement estimates using elastic theory are strongly affected by the selection of soil modulus and the inherent assumptions of infinite elastic half space. Accurate estimates of soil moduli are difficult to obtain because the analyses are based on only a single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent footings.

For footings with eccentric loads, the area, A', should be computed based on reduced footing dimensions as specified in Article 10.6.1.3.

10.6.2.4.2c—Hough Method

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be determined using Eqs. 10.6.2.4.2c-2 and 10.6.2.4.2c-3. *SPT* blow counts shall be corrected as specified in Article 10.4.6.2.4 for depth, i.e. overburden stress, before correlating the *SPT* blow counts to the bearing capacity index, C'.

C10.6.2.4.2c

The Hough method was developed for normally consolidated cohesionless soils.

The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite size.

The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about

$$S_e = \sum_{i=1}^{n} \Delta H_i \qquad (10.6.2.4.2\underline{c}\text{-}1)$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \qquad (10.6.2.4.2\underline{c}\text{-}2)$$

where:

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

ΔH_i = elastic settlement of layer *i* (ft)

H_C = initial height of layer *i* (ft)

C' = bearing capacity index from Figure 10.6.2.4.2c-1 (dim)

σ'_o = initial vertical effective stress at the midpoint of layer *i* (ksf)

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

In Figure 10.6.2.4.2-1, N1 shall be taken as $N1_{60}$, Standard Penetration Resistance, *N* (blows/ft), corrected for overburden pressure as specified in Article 10.4.6.2.4..

three times the footing width. The maximum layer thickness should be about 10 ft.

While ~~Cheney and Chassie (2000), and~~ Hough (1959), did not specifically state that the SPT *N* values should be corrected for hammer energy in addition to overburden pressure, due to the vintage of the original work, hammers that typically have an efficiency of approximately 60 percent were in general used to develop the empirical correlations contained in the method. If using SPT hammers with efficiencies that differ significantly from this 60 percent value, the *N* values should also be corrected for hammer energy, in effect requiring that $N1_{60}$ be used <u>(Samtani and Nowatzki, 2006)</u>.

<u>Studies conducted by Gifford et al. (1987) and Samtani and Nowatzki (2006) indicate that Hough's procedure is conservative. Such conservatism may be acceptable for the evaluation of the settlement of embankments. However, in the case of shallow foundations such conservatism may lead to unnecessary use of costlier deep foundations in cases where shallow foundations may be viable.</u>

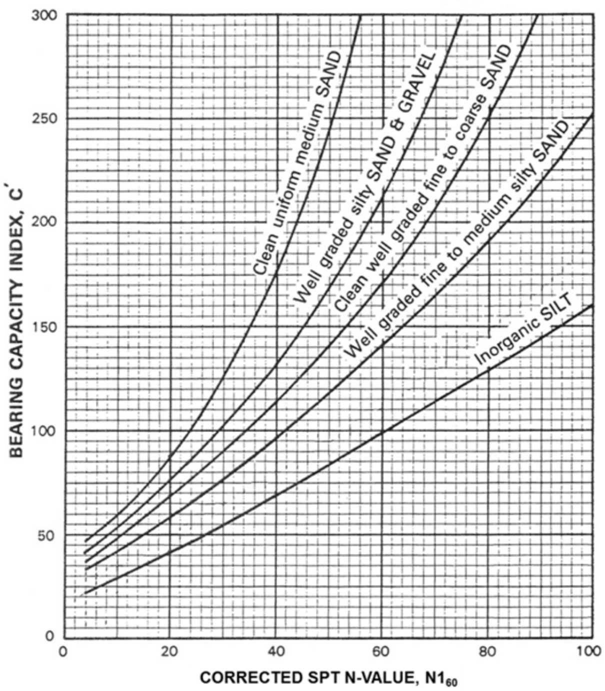


Figure 10.6.2.4.2c-1—Bearing Capacity Index versus Corrected SPT (Samtani and Nowatzki, 2006, after Hough, 1959)

The Hough method is applicable to cohesionless soil deposits. The "Inorganic Silt" curve should generally not be applied to soils that exhibit plasticity because N-values in such soils are unreliable. The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in Article 10.6.2.4.3.

10.6.2.4.2d—Schmertmann Method

Estimation of spread footing immediate settlement, S_i, on cohesionless soils by the empirical Schmertmann, method shall be made using Eq. 10.6.2.4.2d-1.

$$S_i = C_1 C_2 \Delta p \sum_{i=1}^{n} \Delta H_i \qquad (10.6.2.4.2\text{d-}1)$$

in which:

$$\Delta H_i = H_c \left(\frac{I_z}{144XE} \right) \qquad (10.6.2.4.2\text{d-}2)$$

$$C_1 = 1 - 0.5 \left(\frac{p_o}{\Delta p} \right) \geq 0.5 \qquad (10.6.2.4.2\text{d-}3)$$

$$C_2 = 1 + 0.2 \log_{10} \left(\frac{t}{0.1} \right) \qquad (10.6.2.4.2\text{d-}4)$$

where:

C10.6.2.4.2d

Background information for Schmertmann, *et al.* (1978) in the format as presented here can be found in Samtani and Nowatzki (2006).

For C_2 correction factor the time duration, t, in Eq. 10.6.2.4.2d-4 is set to 0.1 years to evaluate the settlement immediately after construction, i.e., C_2 = 1. If long-term creep deformation of the soil is suspected then an appropriate time duration, *t*, should be used in the computation of C_2. Creep deformation is not the same as consolidation settlement. This factor can have an important influence on the reported settlement since it is included in Eq. 10.6.2.4.2d-1 as a multiplier. For example, the C_2 factor for time durations of 0.1 yrs, 1 yr, 10 yrs and 50 yrs are 1.0, 1.2, 1.4 and 1.54, respectively. In cohesionless soils and unsaturated fine-grained cohesive soils with low plasticity, time durations of 0.1 yr and 1 yr, respectively, are generally appropriate and sufficient for cases of static loads.

ΔH_i = elastic settlement of layer i (ft)

H_C = height of compressible soil layer i (ft)

I_z = strain influence factor from Figure 10.6.2.4.2d-1a. The dimension B_f represents the least lateral dimension of the footing after correction for eccentricities, i.e. use least lateral effective footing dimension. The strain influence factor is a function of depth and is obtained from the strain influence diagram. The strain influence diagram is constructed for the axisymmetric case ($L_f/B_f = 1$) and the plane strain case ($L_f/B_f \geq 10$) as shown in Figure 10.6.2.4.2d-1a. The strain influence diagram for intermediate conditions should be determined by simple linear interpolation.

n = number of soil layers within the zone of strain influence (strain influence diagram).

Δp = net uniform applied stress (load intensity) at the foundation depth (see Figure 10.6.2.4.2d-1b) (ksf).

E = elastic modulus of layer i based on guidance provided in Table C10.4.6.3-1 (ksi).

X = a factor used to determine the value of elastic modulus. If the value of elastic modulus is based on correlations with $N1_{60}$-values or q_c from Table C10.4.6.3-1, then values of X shall be taken as follows:

$X = 1.25$ for axisymmetric case ($L_f/B_f = 1$)

$X = 1.75$ for plane strain case ($L_f/B_f \geq 10$)

Use interpolation for footings with values of L_f/B_f between 1 and 10.

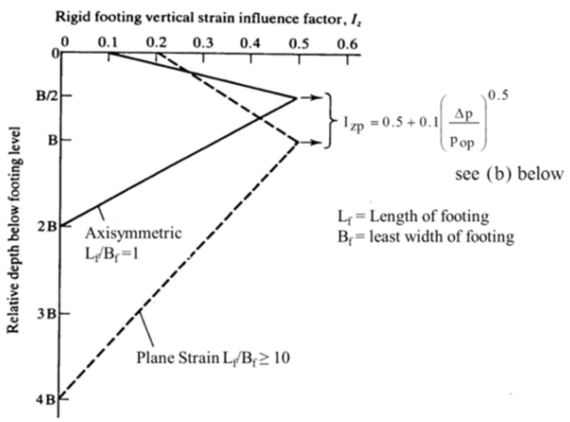
If the value of elastic modulus is estimated based on the range of elastic moduli in Table C10.4.6.3-1 or other sources, use $X = 1.0$.

C_1 = correction factor to incorporate the effect of strain relief due to embedment

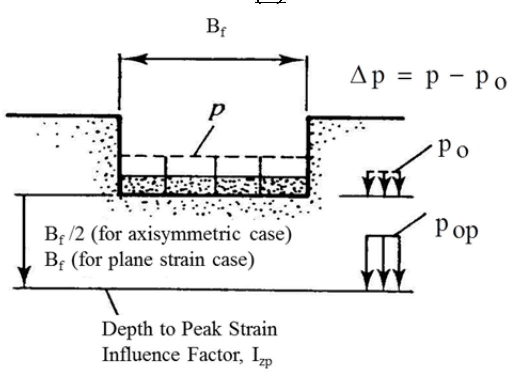
p_o = effective in-situ overburden stress at the foundation depth and Δp is the net foundation pressure as shown in Figure 10.6.2.4.2d-1b (ksf).

C_2 = correction factor to incorporate time-dependent (creep) increase in settlement for time t after construction

t = time t from completion of construction to date under consideration for evaluation of C_2 (yrs)



(a)



(b)

Figure 10.6.2.4.2d-1—(a) Simplified vertical strain influence factor distributions, (b) Explanation of pressure terms in equation for I_{zp} (Samtani and Notatzki, 2006, after Schmertmann, et al., 1978).

The C_2 parameter shall not be used to estimate time-dependent consolidation settlements. Where consolidation settlement can occur within the depth of the strain distribution diagram, the magnitude of the consolidation settlement shall be estimated as per Article 10.6.2.4.3 and added to the immediate settlement of other layers within the strain distribution diagram where consolidation settlement may not occur.

10.6.2.4.2e—Local Method

Use of methods based on local geologic conditions and calibration shall be used subject to approval from the Owner.

C10.6.2.4.2e

Calibration of local methods should be based on processes as described in SHRP 2 R19B program report (Kulicki et al., 2015) and Samtani and Kulicki (2016)

10.10—REFERENCES

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