


SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Service Limit State Design For Bridges (R19B) Training Course

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Eastern Federal Lands
June 27-28, 2017

U.S. Department of Transportation
Federal Highway Administration

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Introduction and Course Objectives

Lecture 1
Introduction and Course Objectives

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Easterm Federal Lands
June 27-28, 2017

U.S. Department of Transportation
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Learning Outcomes

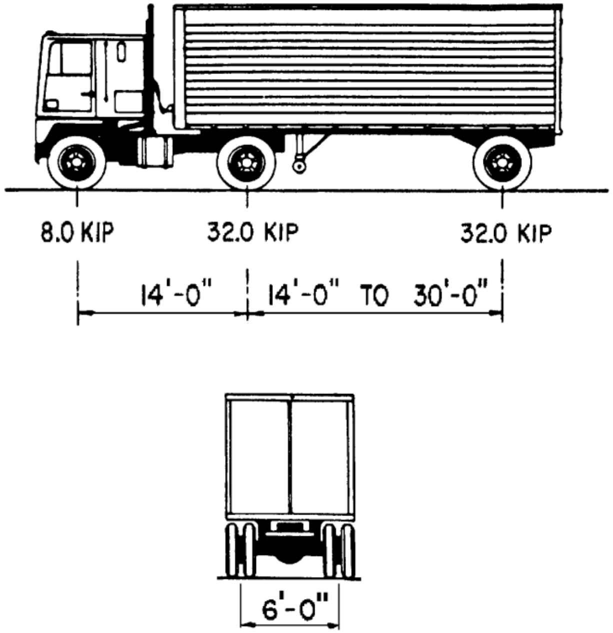
- Brief history of the development of AASHTO design live load
- Overview of the basics of reliability based specifications
- Overview of research on service limit state calibration
- Overview of course learning outcomes

A Brief History of AASHTO LRFD

- 1931 – First printed version of AASHO *Standard Specifications for Highway Bridges and Incidental Structures* using working stress design
- 1970s AASHO becomes AASHTO (1990s AREA becomes AREMA)
- Early 1970s AASHTO adopts LFD
- Late 1970s OMTC starts work on limit-states based OHBDC
- 1986 AASHTO explores need to change

A Brief History of AASHTO LRFD *(continued)*

1944 HS 20 Design Truck Added



Live Load Continued to be Debated

- Late 1960s – H40, HS25 and HS30 discussed
- 1969
  - SCOBS states unanimous opposition to increasing weight of design truck – “wasteful obsolescence” of existing bridges
- 1978 – HS25 proposed again
- 1979 – HS25 proposed once again
  - Commentary: Need for heavier design load seems unavoidable
  - HS25 best present solution
  - 5% cost penalty
  - Motion soundly defeated

LRFD Design Code Objectives

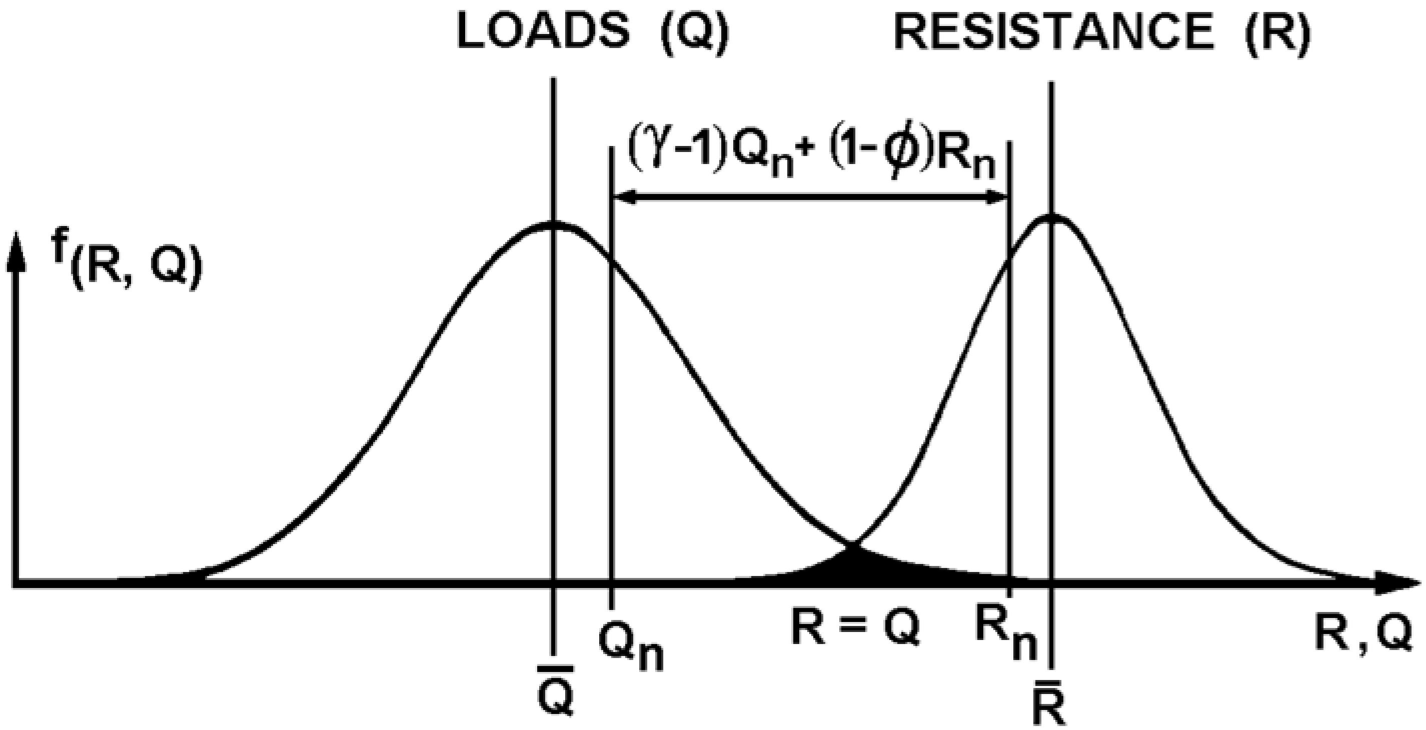
In 1986, work started on AASHTO LRFD, specifications requirements:

- Technically state-of-the-art specification
- Comprehensive as possible
- Readable and easy to use
- Keep specification-type wording – do not develop a textbook
- Encourage a multi-disciplinary approach to bridge design

Major Changes

- A new philosophy of safety - LRFD
- The identification of four limit states
- The relationship of the selected reliability level, the load and resistance factors, and load models through the process of calibration
  - New load factors
  - New resistance factors

LRFD - Basic Design Concept



Load and Resistance Factor Design

$\Sigma\eta_i \gamma_i Q_i \leq \phi \; R_n = R_r$

in which:

- $\eta_i = \eta_D \eta_R \eta_I \geq 0.95$ for loads for max
  $= 1/(\eta_I \eta_D \eta_R) \leq 1.0$ for loads for min

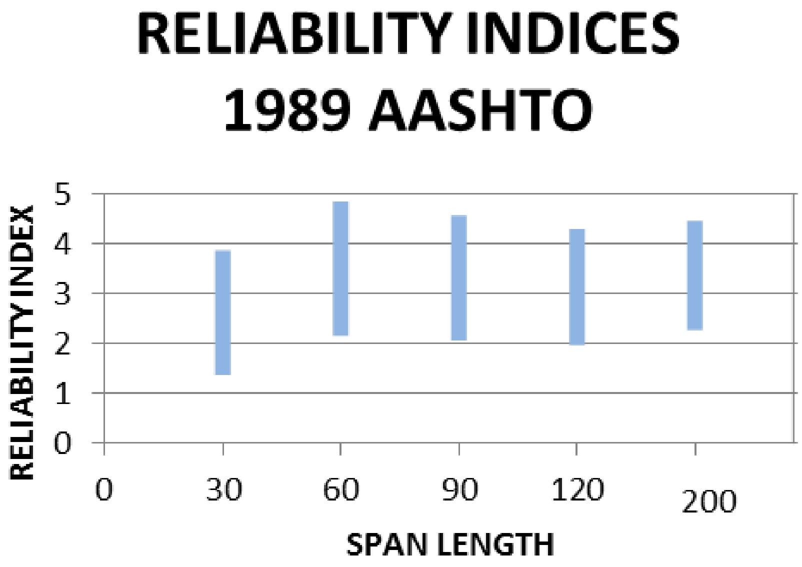
where:

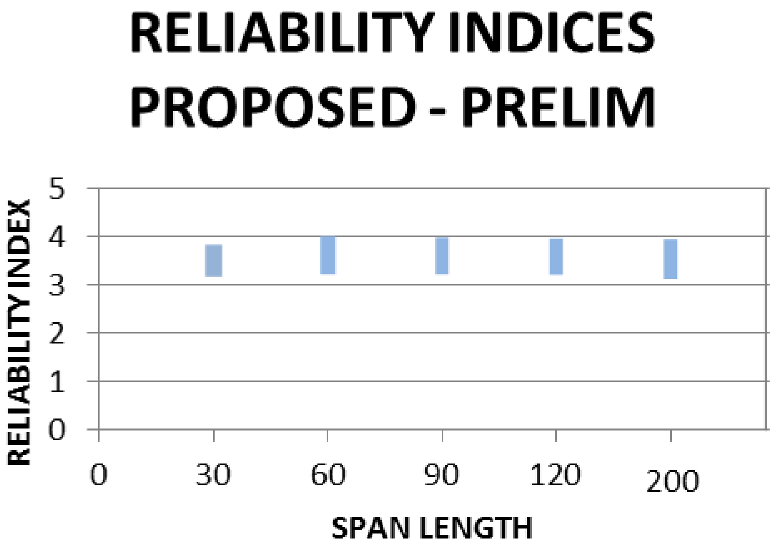
- $\gamma_i =$ load factor: a statistically based multiplier on force effects
- $\phi =$ resistance factor: a statistically based multiplier applied to nominal resistance

LRFD (Continued)

- η_i = load modifier
- η_D = a factor relating to ductility
- η_R = a factor relating to redundancy
- η_I = a factor relating to importance
- Q_i = nominal force effect: a deformation stress, or stress resultant
- R_n = nominal resistance
- R_r = factored resistance: ϕR_n

Reliability of Bridges before and after LRFD (Strength Limit State)

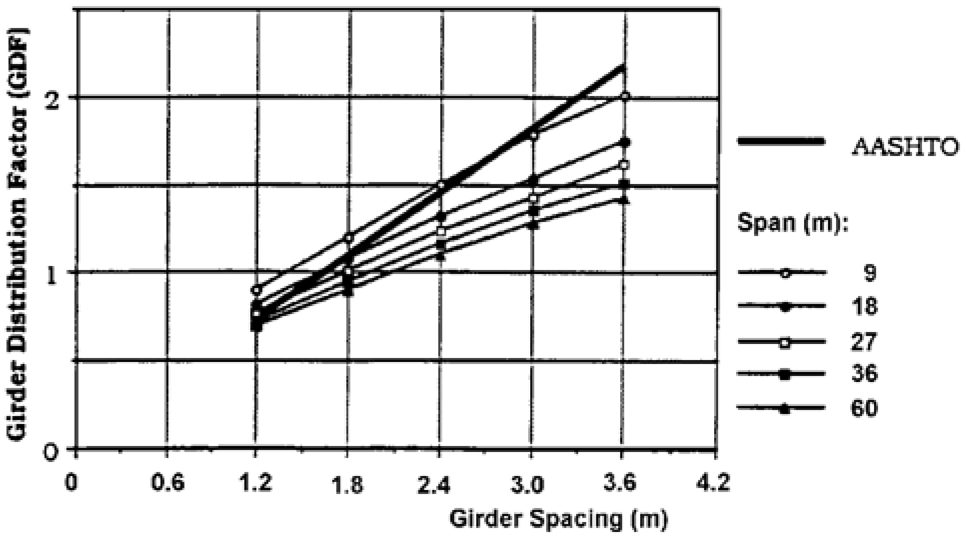




Major Changes

- Revised calculation of load distribution

$$g = 0.075 + \left(\frac{S}{2900}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{Lt_s^3}\right)^{0.1}$$

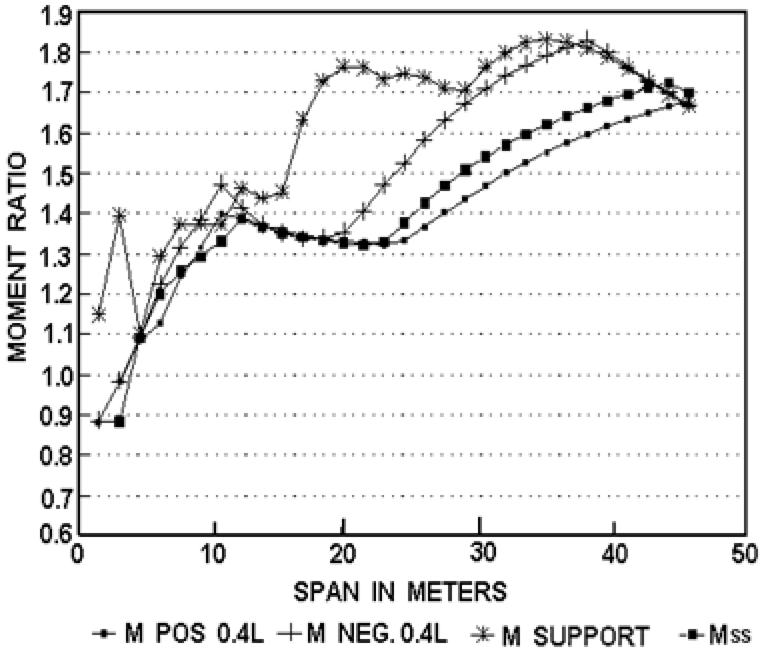


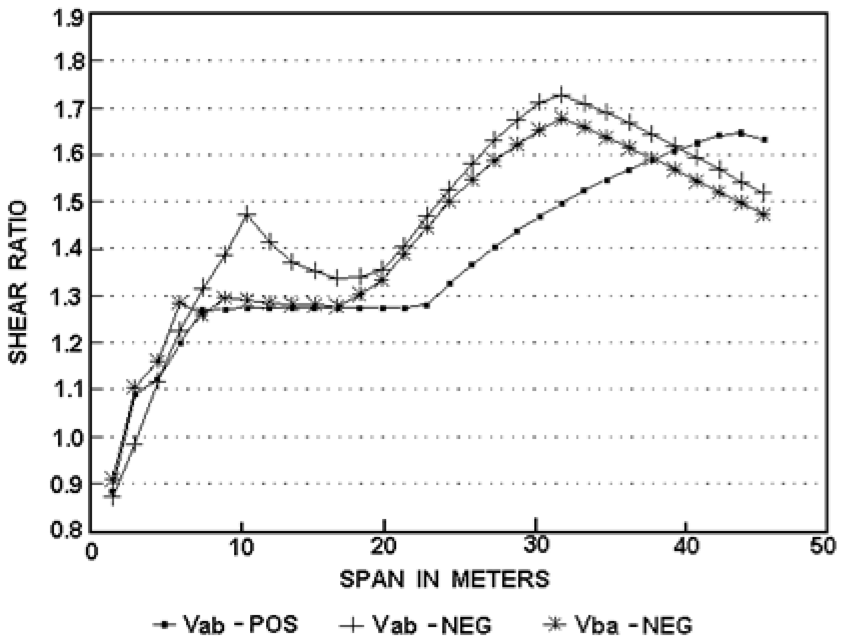
Circa 1990

Major Changes

- New Live Load Model – HL93
- New live load distribution factors
- Revised methods of analysis and design

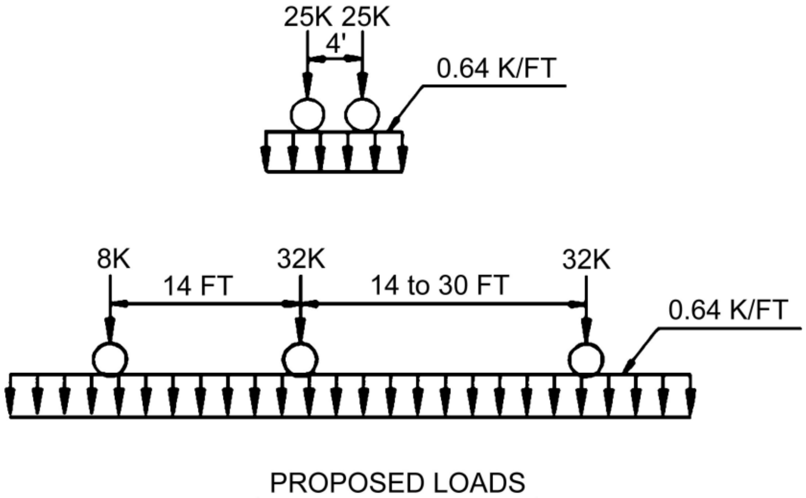
Exclusion/HS20 Truck or Lane or 2 – 25 kips Axles @ 4 ft.



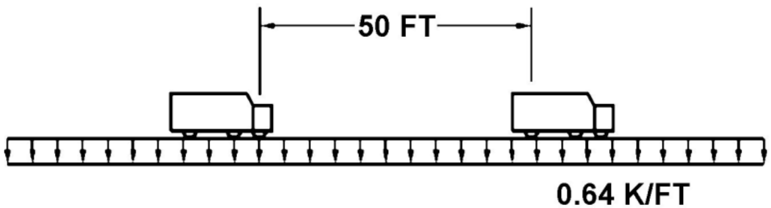


Ratio of Exclusion trucks to HS20 Truck or Lane or 2 – 25 kips Axles @ 4 ft.

Selected Notional Design Load



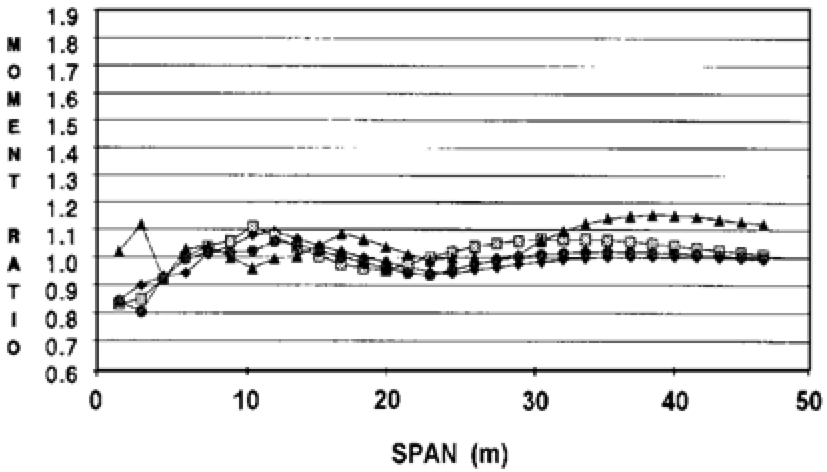
HL-93

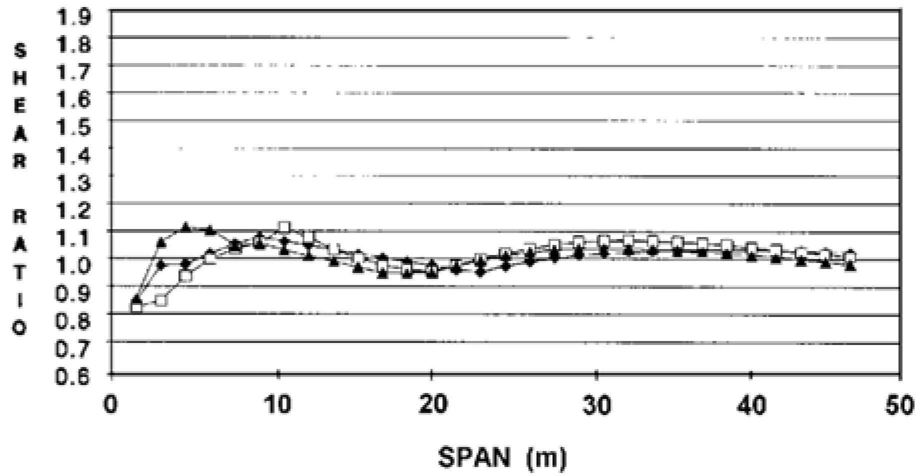


- NEGATIVE MOMENT AND INTERIOR REACTIONS
- ≥ 50 FT
- FIXED WHEELBASE ON TRUCK = 14 FT
- 90%

EXCL/HS20 Truck or Lane or 2 – 25 kips Axles @ 4 ft.

EXCL/HL 93 – Circa 1992





Why New Research Was Needed

- The original AASHTO LRFD live-load study was based on load measurements made in the 1970s in Ontario. How does this relate to today's loads?

- The specifications were calibrated for the strength limit state where the definition of failure is relatively simple: If the factored loads exceed the factored resistance, failure, i.e. severe distress or collapse, will take place.

  What about service limit state and what is failure under service limit states?

Where Did We Go From Here“?

Two projects were initiated to calibrate the service limit state

- SHRP2 R19B, Bridge for Service Life Beyond 100 Years: Service Limit State Design (SLS)
- NCHRP 12-83, *Calibration of Service Limit State for Concrete*

Research Teams

R19B Research Team

Modjeski and Masters, Inc.:	John Kulicki, Ph.D., P.E.
	Wagdy Wassef, Ph.D., P.E.
University of Delaware:	Dennis Mertz, Ph.D., P.E.
University of Nebraska:	Andy Nowak, Ph.D.
NCS Consultants:	Naresh Samtani, Ph.D., P.E.

NCHRP 12-83 Research Team

Modjeski and Masters, Inc.:	Wagdy Wassef, Ph.D., P.E.
	John Kulicki, Ph.D., P.E.
University of Delaware:	Dennis Mertz, Ph.D., P.E.
University of Nebraska:	Andy Nowak, Ph.D.
Rutgers University:	Hani Nasif, Ph.D., P.E.

R19B & NCHRP 12-83 Research Objectives

- Identify service limit states in the then-current specifications.
- Identify new service limit states required to cover aspects of design not currently covered by the design specifications.
- Develop the methodology for service limit state calibration. The process should allow future updates and, where, applicable, user input of region-specific information.
- Where adequate information related to a certain limit state exists, calibrate the limit state.

Training Course Objectives

Introducing the research from R19B and NCHRP 12-83 and including:

- Provide the background of the calibration process
- Introduce the difference between strength and service limit states calibration
- Introduce different types of service limit states (Drivers and reversibility)
- Provide an overview of live load WIM data studies for the calibration
- Provide an overview of the calibration of service limit states in the specifications with emphasis on foundations
- Provided an overview of specifications revisions related to service limit states calibration

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Introduction and Course Objectives

Lecture 2: Basic SLS Calibration Concepts and Calibration Process

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Learning Outcomes

- **Gain knowledge** of the history of AASHTO Bridge Design Specifications
- Understand the following concepts:
  - **Limit states**
  - General **calibration process**
  - **History** of incorporation of calibrated limit states in bridge design specifications
  - **Strength vs. Service** calibration

Learning Outcomes *(continued)*

- Understand the following concepts *(continued):*
  - **Reversible vs. non-reversible** limit states
  - **Load-driven vs. non-load-driven** limit state
- Service Limit States **Calibration Process**

A Brief History of AASHTO LRFD

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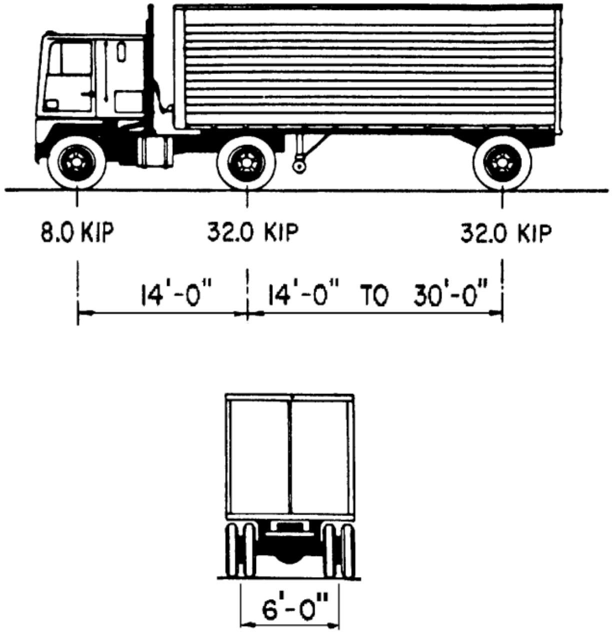
- Early 1970s – AASHTO adopts LFD
- Late 1970s – OMTC starts work on limit-states based OHBDC
- 1979 – First edition of OHBDC
- 1986 – AASHTO explores need to change

A Brief History of AASHTO LRFD *(continued)*

- In 1986, work started on AASHTO LRFD
- First edition published in 1994
- Mainly, the strength limit state was statistically calibrated

Live Load –

1944 HS 20 Design Truck Added



A Brief History of AASHTO LRFD *(continued)*

Live Load continued to be debated:

- Late 60s – H40, HS25 and HS30 discussed
- 1969 – SCOBS states unanimous opposition to increasing weight of design truck – "wasteful obsolescence" of existing bridges
- 1978 – HS25 proposed again
- 1979 – HS25 considered again
  - Commentary:
    - Need for heavier design load seems unavoidable
    - HS25 best present solution
    - 5% cost penalty
    - Motion soundly defeated

Calibration Approaches

- Full probability approach preferred
- Semi-probabilistic partial factor approach
- Deemed to satisfy

So far, full probability approach is limited to Chloride ion penetration.

Major Features of AASHTO LRFD

- A new philosophy of safety - LRFD
- The identification of four limit states
- The relationship of the selected reliability level, the load and resistance factors, and load models through the process of calibration
  - New load factors
  - New resistance factors

Basic Concepts

The following slides will provide the definition of the basic concepts used in specifications calibration, particularly, service limit state calibration.

Limit States

- A **limit state** is a condition of a structure beyond which it no longer fulfills the relevant design criteria.
- The condition may refer to a **degree of loading or other actions** on the structure
- The **criteria** refer to structural integrity, fitness for use, durability or other design requirements.

Limit States *(continued)*

- AASHTO LRFD includes **four basic limit states:**
  - **Strength** (Five load combinations)
  - **Service** (Four load combinations)
  - **Fatigue and Fracture** (Two load combinations)
  - **Extreme event** (Two load combinations)

Each load combination is meant to address the structure under a certain load or other condition.

Limit States *(continued)*

- AASHTO LRFD strength limit state load combinations

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	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—

Service Limit States in AASHTO LRFD

“**General**” Service Limit States **(SLS)**:

- Live load deflections
- Bearings-movements and service forces
- Settlement of foundations and walls

Service Limit States in AASHTO LRFD *(continued)*

Steel SLS

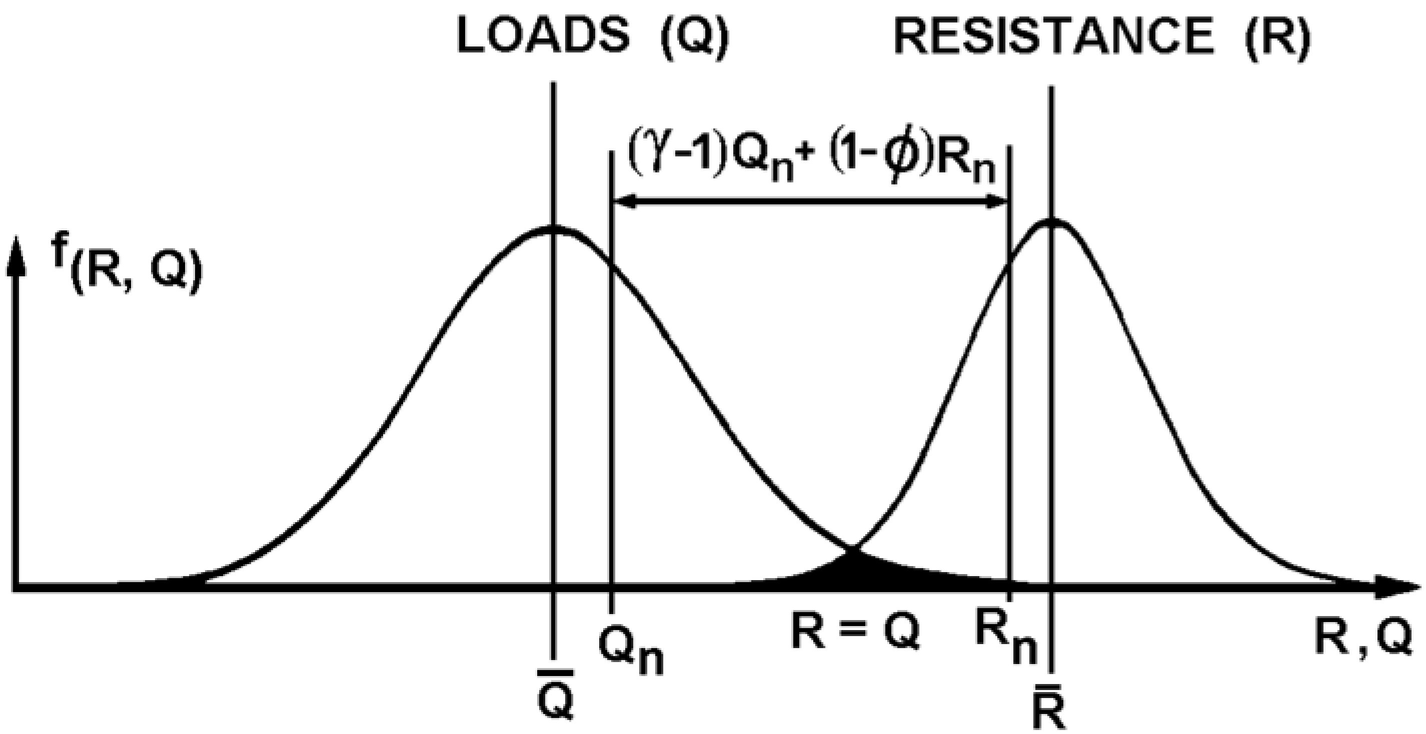
- Permanent deformations in compact steel components
- Fatigue of structural steel, steel reinforcement and concrete (through its own limit state)
- Slip of slip–critical bolted connections

Service Limit States in AASHTO LRFD *(continued)*

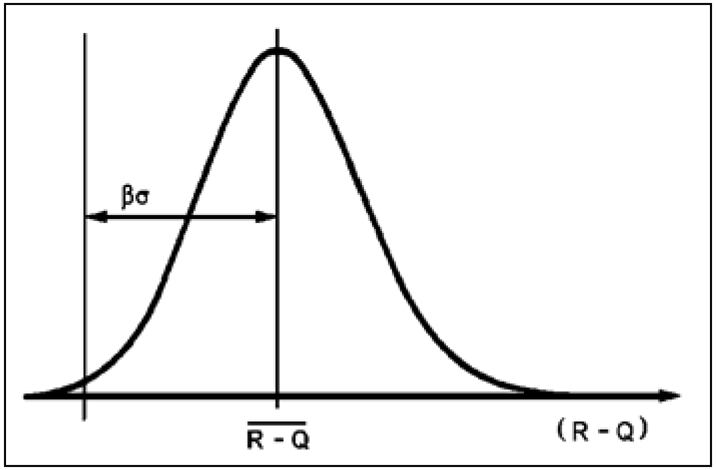
Steel SLS

- Stresses in prestressed concrete under service loads
- Crack control reinforcement
- Shrinkage and temperature reinforcement
- Splitting reinforcement

LRFD - Basic Calibration Concept



Some Algebra



$$\sigma_{(R-Q)} = \sqrt{\sigma_R^2 + \sigma_Q^2}$$

$$\beta = \frac{\overline{R} - \overline{Q}}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

$$\overline{R} = \overline{Q} + \beta \sqrt{\sigma_R^2 + \sigma_Q^2} = \lambda R = \frac{1}{\varphi} \lambda \Sigma \gamma_i x_i$$

$$\phi = \frac{\lambda \Sigma \gamma_i x_i}{\overline{Q} + \beta \sqrt{\sigma_R^2 + \sigma_Q^2}}$$

Load and Resistance Factor Design

$\Sigma \eta_i \, \gamma_i \, Q_i \leq \phi \;\; R_n = R_r$

in which:

- $\eta_i = \eta_D \, \eta_R \, \eta_I \geq 0.95$ for loads for which max value is appropriate
- $\eta_i = 1/(\eta_I \, \eta_D \, \eta_R) \leq 1.0$ for loads for which min value is appropriate

where:

- γ_i = load factor: a statistically-based multiplier on force effects
- ϕ = resistance factor: a statistically-based multiplier applied to nominal resistance

LRFD *(continued)*

- η_i = load modifier
- η_D = a factor relating to ductility
- η_R = a factor relating to redundancy
- η_I = a factor relating to importance
- Q_i = nominal force effect: a deformation stress, or stress resultant
- R_n = nominal resistance
- R_r = factored resistance: ϕR_n

Reliability Calculations for Strength

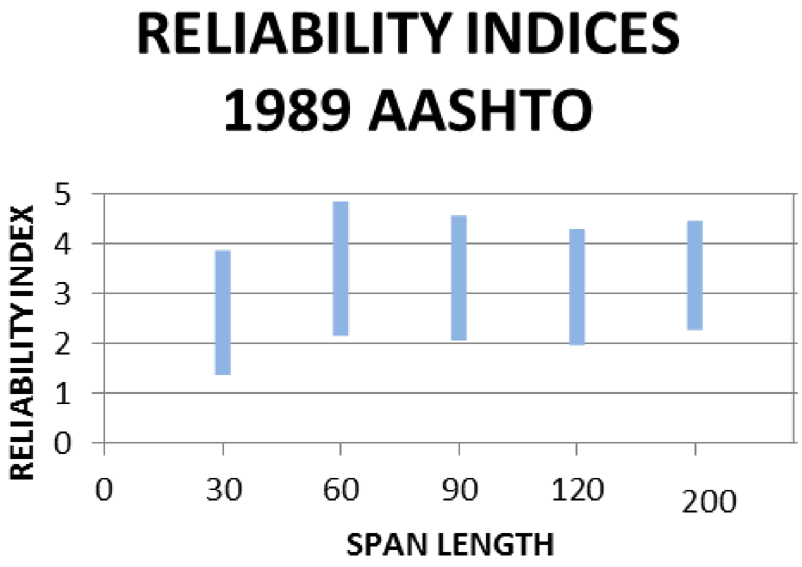
Reliability index analysis for AASHTO LRFD was done for M and V using simulated bridges based on real bridges.

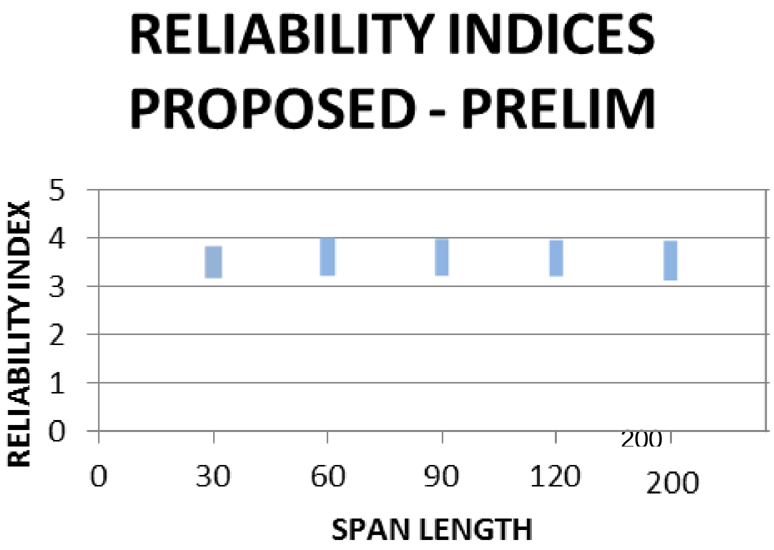
- 25 non-composite steel girder bridge simulations with spans of 30, 60, 90,120,and 200 feet, and spacing of 4, 6, 8, 10,and 12 feet.
- Composite steel girder bridges having the same parameters identified above.
- P/C I-beam bridges with the same parameters identified above.
- R/C T-beam bridges with spans of 30, 60, 90, and 120 feet, with spacing as above.

Reliability Calculations for Strength *(continued)*

- Different combinations for load and resistance factors for the basic strength load combination were attempted. The combination with the most uniform reliability index was selected.
- Other strength load combinations were analyzed to the extent possible.

Reliability of Bridges before and after LRFD (Strength Limit State)





Other Reliability-Based Specifications

- Ontario Highway Bridge Design Code (OHBDC) (Later adopted as Canadian Highway Bridge Design Code, CHBDC)
- Eurocode
- BS 5400
- Japanese Geotechnical

Other Reliability-Based Specifications *(continued)*

- Common characteristics of different specifications
  - Specifications are **similar to LRFD** in that the main limit state calibrated is the strength limit state.
  - **Different specifications** treat basically similar issues.
  - The service limit states-related provisions in the Eurocode seem to have been calibrated using the **Delphi Process** (explained later) and engineering judgment.

Why SLS Calibration is Lagging

- Difficulty to define the “failure” criteria and consequences of failure
- Lack of adequate information on the performance

In some cases, the lack of information requires the use of the “Delphi Process.”

What is a "Delphi Process"?

- Relatively **structured process** to synthesize fragmented knowledge
- Pools knowledge and experience of experts to:
  - **Define** the playing field
  - See if what we have is working – **satisfaction level**
  - Look for **knowledge gaps**

Why Delphi is Needed for Some Service Limit States

- **Compensates** for lack of useful data on SLS
- Subjective features benefit from **consensus** on
  - Significance of the limit state
  - Exceedance rate

Strength Vs. Service Calibration

Criteria	Strength	Service
Consequences of exceedence	The bridge or, more likely, a component of the bridge will collapse or will be severely damaged.	The comfort of the users will be affected and/or the deterioration of the affected components will accelerate.

Strength Vs. Service Calibration *(continued)*

Criteria	Strength	Service
Frequency of exceedence	The possibility of exceeding the limit state during the life span of the bridge should be very low.	Frequency of exceedence varies based on the consequences of exceeding the limit state.

Strength Vs. Service Calibration *(continued)*

Criteria	Strength	Service
What are we trying to stop	Severe damage that may lead to failure or collapse of bridge components that will lead to loss of service and/or loss of life.	Damage that may cause user discomfort, visible distress, and/or accelerate deterioration.

Strength Vs. Service Calibration *(continued)*

Criteria	Strength	Service
Target Reliability	Target reliability is high to prevent loss of structure, its use, and loss of life.	Target reliability needs to be high enough to minimize the effects of exceeding the limit state.

Reversible Vs Non-Reversible

- **Reversible limit states** are those that no residual effects remain once the driver of the limit state is removed.
  Example: Decompression of prestressed girders under Service III limit state.
- **Non-reversible limit states** are those that residual effects remain once the driver of the limit state is removed.
  Example: Yield of steel components under Service II load combination.

Reversible Vs Non-reversible *(continued)*

- Frequency of exceedence for **non-reversible** limit states should be kept **low** while **higher frequency** of exceedence of **reversible** limit states may be acceptable.

Load-Driven Vs. Non-Load-Driven

- Some service limit states are directly related to applied loads.

  Examples:

  - Stresses in prestressed concrete under service loads
  - Crack control reinforcement

- Such limit states are amenable to statistical calibration if adequate information on the statistical variation in the loads and resistance is available.

Load-Driven Vs. Non-Load-Driven *(continued)*

- Some service limit states are not directly related to applied loads.

  Examples:

  - Temperature and shrinkage reinforcement
  - Splitting reinforcement

- Such limit states are not amenable to statistical calibration using available information due to the lack of statistical information on the distribution of associated forces and the resistances.

Overview of SLS Calibration Procedure

- **Step 1:** Formulate the Limit State Function and identify basic variables
  - Identify the load and resistance **parameters**
  - Formulate the **limit state function**
  - Establish the acceptability **criteria**

  In most cases, it was not possible to select a deterministic boundary between what is acceptable and unacceptable.

  Some code-specified limit state functions do not have a physical meaning (e.g. allowable compression stress in concrete).

Overview of SLS Calibration Procedure *(continued)*

- **Step 2:** Identify and select representative structural types and design cases
  - **Select** the representative components and structures to be considered in the development of code provisions for the SLSs.

Overview of SLS Calibration Procedure *(continued)*

- **Step 3:** Determine load and resistance parameters for the selected design cases
  - Identify the **design parameters** based on typical structural types, loads, and locations (climate, exposure).
  - For each considered element and structure, **values** of typical load components must be determined.

Overview of SLS Calibration Procedure *(continued)*

- **Step 4:** Develop statistical models for Load and Resistance
  - **Gather** statistical information about:
    - **Performance** of considered types and models in selected representative locations and traffic.
    - **Information** about quality of workmanship. Ideally, for given location and traffic, including:
      - ✓ General assessment of performance,
      - ✓ Assumed time to initiation of deterioration,
      - ✓ Assumed deterioration rate as a function of time, maintenance, and repair (frequency and extent).

Overview of SLS Calibration Procedure *(continued)*

- **Step 4 *(continued):***
  - Develop statistical **load and resistance models** (as a minimum, determine the bias factors and coefficients of variation).
    Load and resistance parameters should include **magnitude**, as is the case with strength limit states, but also include **frequency of occurrence** (e.g. crack opening) and as a function of time (e.g. corrosion rate, chloride penetration rate).

From the SHRP R19 B final report: "The available statistical parameters were utilized. However, the database is rather limited, and for some serviceability limit states, there is a need to assess, develop, and/or derive the statistical parameters.)"

Overview of SLS Calibration Procedure *(continued)*

- **Step 5:** Develop the Reliability Analysis Procedure
  - The reliability can be calculated using either a **closed-form formula or Monte Carlo method**. Typically, the latter is used.

Overview of SLS Calibration Procedure *(continued)*

- **Step 6:** Calculate the **Reliability Indices** for current design code and current practice
- **Step 7:** Review the results and select the **Target Reliability Index, β_T**
  - Based on the calculated reliability indices, select the target reliability index, $\beta_{T.}$
  - Select the acceptability criteria, i.e., performance parameters, that are acceptable, and performance parameters that are not acceptable.

Overview of SLS Calibration Procedure *(continued)*

- **Step 8:** Select a **set of potential Load and Resistance Factors**
  - Selected design parameters (load and resistance factors) should meet the **acceptability criteria** for the considered design situations (location and traffic).
  - Selected design parameters should provide **reliability** that is consistent, uniform, and conceivably close to the target level.

Overview of SLS Calibration Procedure *(continued)*

- **Step 9:** Calculate Reliability Indices
  - **Calculate the reliability indices** corresponding to the recommended set of load and resistance factors for verification.
  - If the design parameters do not provide consistent safety levels, **modify the parameters and repeat Step 8.**

Overview of SLS Calibration Procedure *(continued)*

- The nine steps above are the **basic steps**.
- Some modifications were applied to accommodate specifics of different limit states.

Key Points

- **Service calibration differs from strength calibration.**
- Level of **reliability varies with consequences**.
- **Sufficient information** needed for SLS calibration **not available for all limit states**.
- A general calibration process has been developed but may need **revisions to fit individual limit states.**

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Introduction and Course Objectives

Lecture 3: Live Load Calibration

Wagdy Wassef, PhD, PE
AECOM

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June 27-28, 2017

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American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

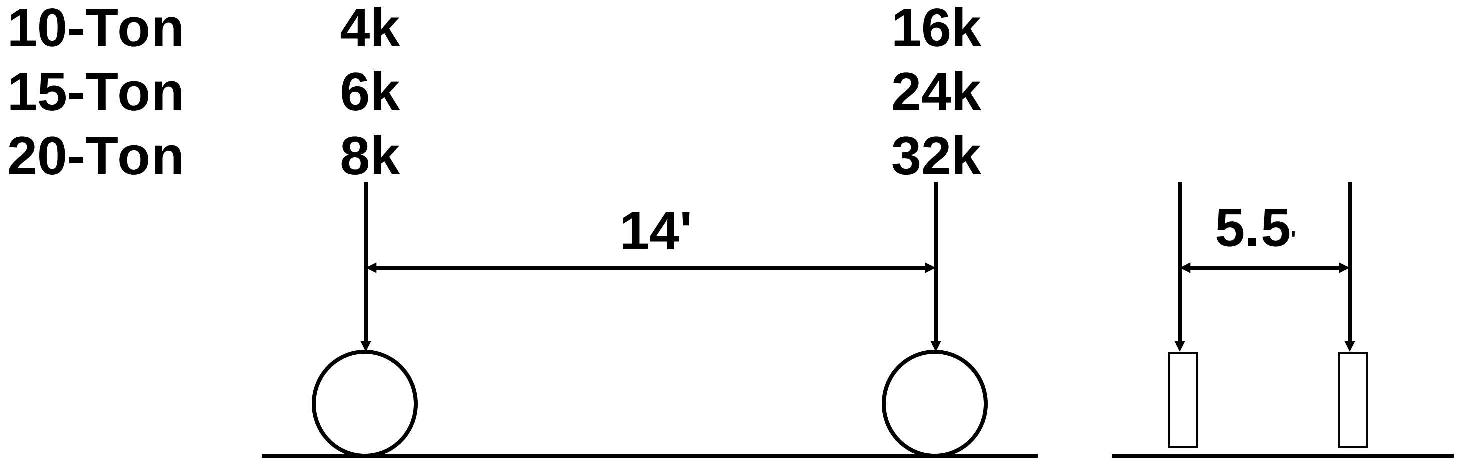
- Knowledge of the history of **AASHTO Live Loads**
- Knowledge of the Live Load used for **AASHTO LRFD calibration** in late 90s
- Knowledge of **Weigh-in-Motion** (WIM) data issues
  - Data collections
  - Quality of data
  - Filtering
  - Use of results

Learning Outcomes *(continued)*

- **Application** of WIM data in determining:
  - Multiple Presence factor
  - Live Load model for strength and service limit states
  - Live Load for fatigue limit states

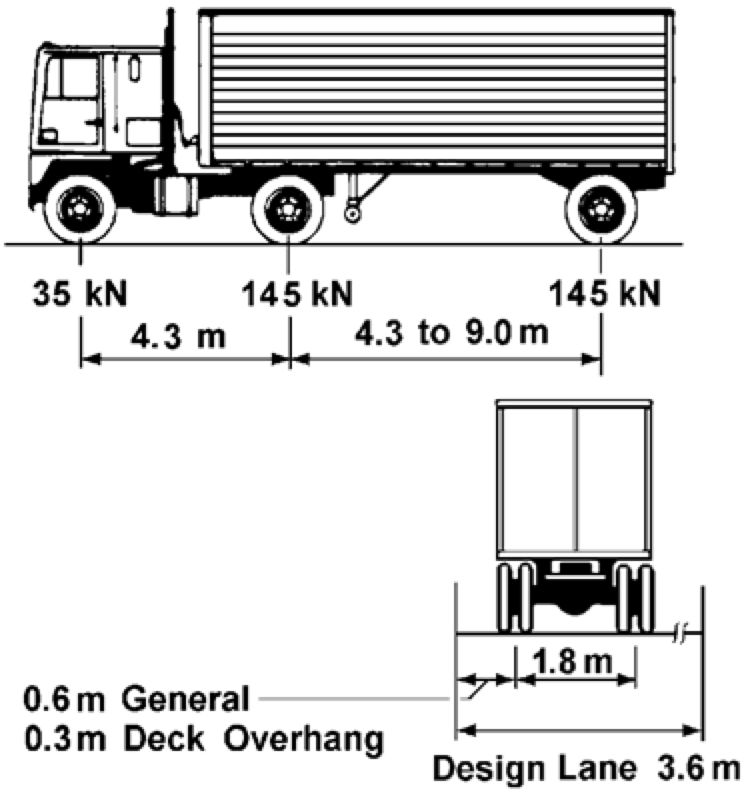
History of AASHTO Live Loads

- **1923 AREA Specification**



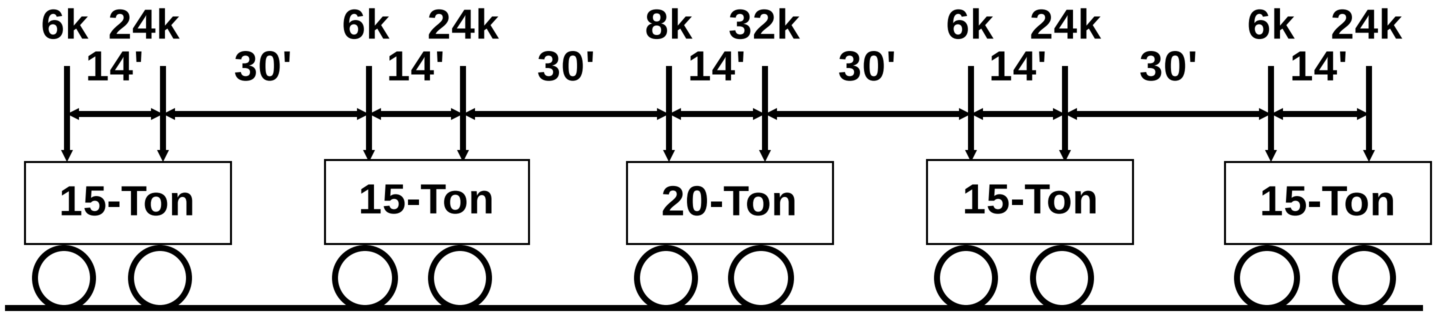
History of AASHTO Live Loads (continued)

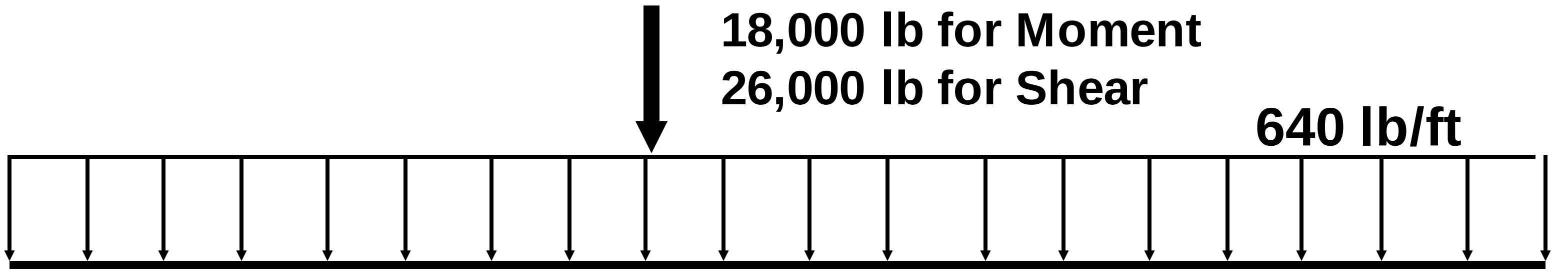
- **1944 HS 20 Design Truck Added**



History of AASHTO Live Loads (continued)

- **1928-1929 Conference Specification**



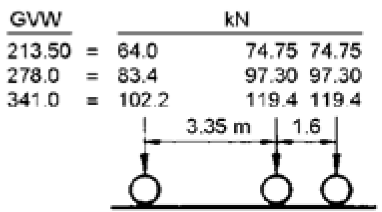


History of AASHTO Live Loads (continued)

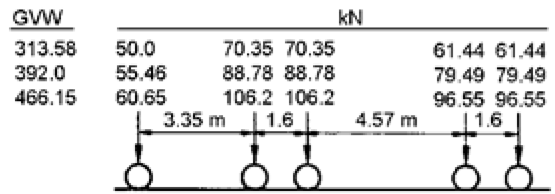
- Late 60s – H40, HS25 and HS30 discussed
- 1969 – SCOBS states unanimous opposition to increasing weight of design truck – "wasteful obsolescence" of existing bridges
- 1978 – HS25 proposed again
- 1979 – HS25 again – commentary":
  - Need for heavier design load seems unavoidable
  - HS25 best present solution
  - 5% cost penalty
- Motion defeated

History of AASHTO Live Loads (continued)

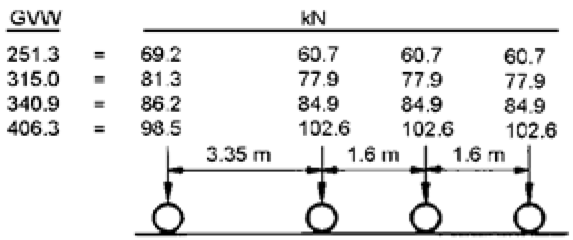
- "Exclusion Loads" – Based on TRB Special Report 225, 1990



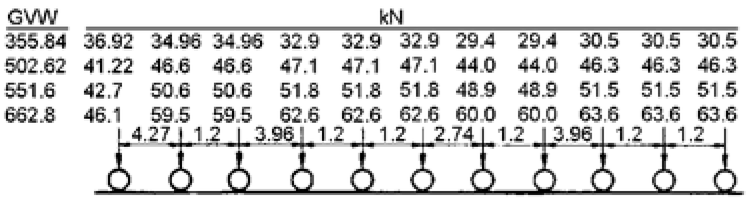
EX 3A (WB 4.95)



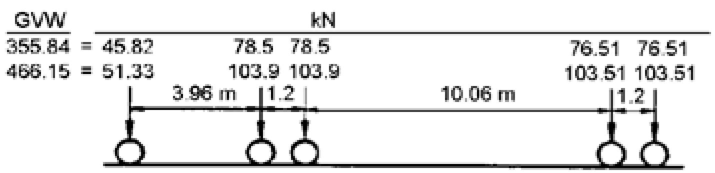
EX 3 - S2 (WB 11.12)



EX 4A (WB 6.55)



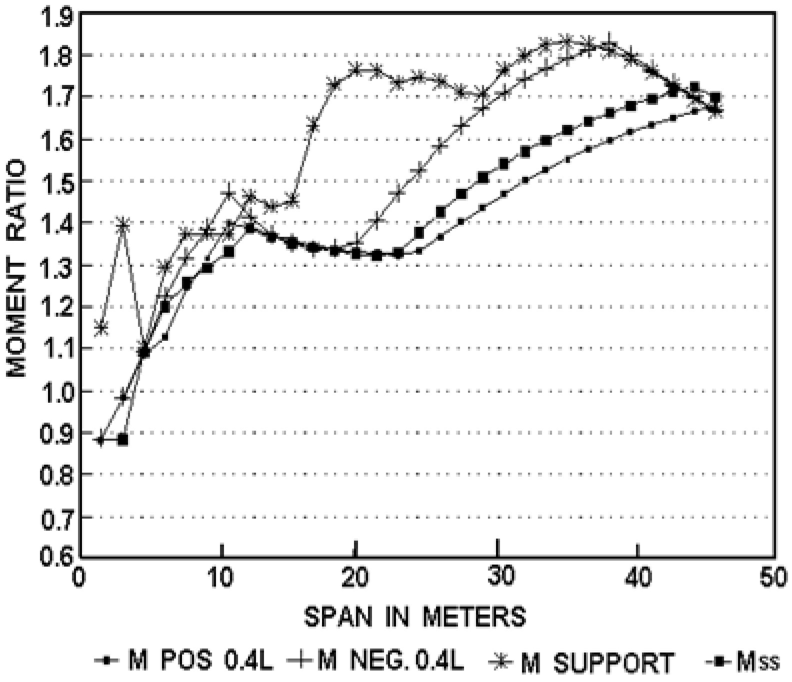
EX 3 - S3 - 5 (WB 22.13)

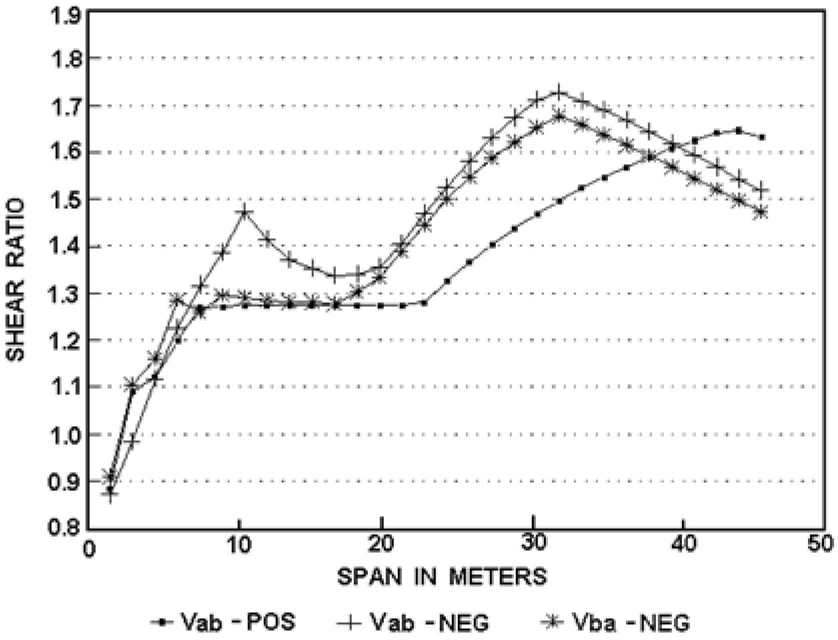


EX 3 - S2 (WB16.42)

History of AASHTO Live Loads (continued)

Ratio of Exclusion Loads/HS20 Load (truck, lane or 24 kips axles @4')





What is Weigh-in-Motion Data?

- Shows stations located on highway
- Measures axle weight, spacing, and speed
- Records total truck weight and length
- Records the lane the truck used
- Determines what FHWA vehicle class
- Shows time stamp for each record
- Does not require stopping trucks and as a result, are not avoided by heavy trucks

Weigh-in-Motion Data Used

- Truck WIM data were obtained from the FHWA and NCHRP Project 12-76.
- More than 60 million records were available.
- Strength Live Load model (HL 93) is adaptable as national notional SLS Live Load model.

Review of WIM data indicated that site/region specific live load should be accommodated in the site-specific calibration for unusual traffic volumes.

Quality of WIM Data

- WIM data need to be thoroughly reviewed to eliminate:
  - Records that are obviously wrong
  - Records that are probably special permit vehicles (large number of axles, unusual axle weight distribution, unusual configuration, …)
  - Records that do not matter
  - Data sets that do not follow typical trends

Quality of WIM Data *(continued)*

- Ensuring quality of WIM Data is achieved through the “filtering” process.
- Total number of records about 65 million, about 35 million used in the calibration
  - 10 million failed filters – obviously bad data
  - 13 million incompatible format
  - 7 million from a state with unique mix of heavy vehicles

Initial Filtering Criteria For Non-Fatigue SLS (FHWA, Unless Noted)

- **Excluded Vehicles** (mostly NCHRP 12-76 filters)
  - 2 kips>Individual axle weight > 70kips -
  - First axle spacing <5 ft
  - Individual axle spacing < 3.4ft
  - GVW +/- the sum of the axle weights by more than 10%.
  - Sum of axle spacing - total length > +/- 1 ft.
  - GVW < 10
  - Steering axles < 6 kips
  - 7 >Total length >200 ft
  - 10 > Speed > 100 mph
  - FHWA Classes 3 – 14

Additional Filtering

Filter #1

1 - Truck length > 120 ft

2 - Sum of axle spacing > length of truck.

3 - GVW +/- sum of the axle weights by more than 7%

Filter #2

1 - Total # of axles < 3 **AND** GVW >50 kips

2 - Steering axle > 35 k

3 - Individual axle weight > 45 kips

Filter #3

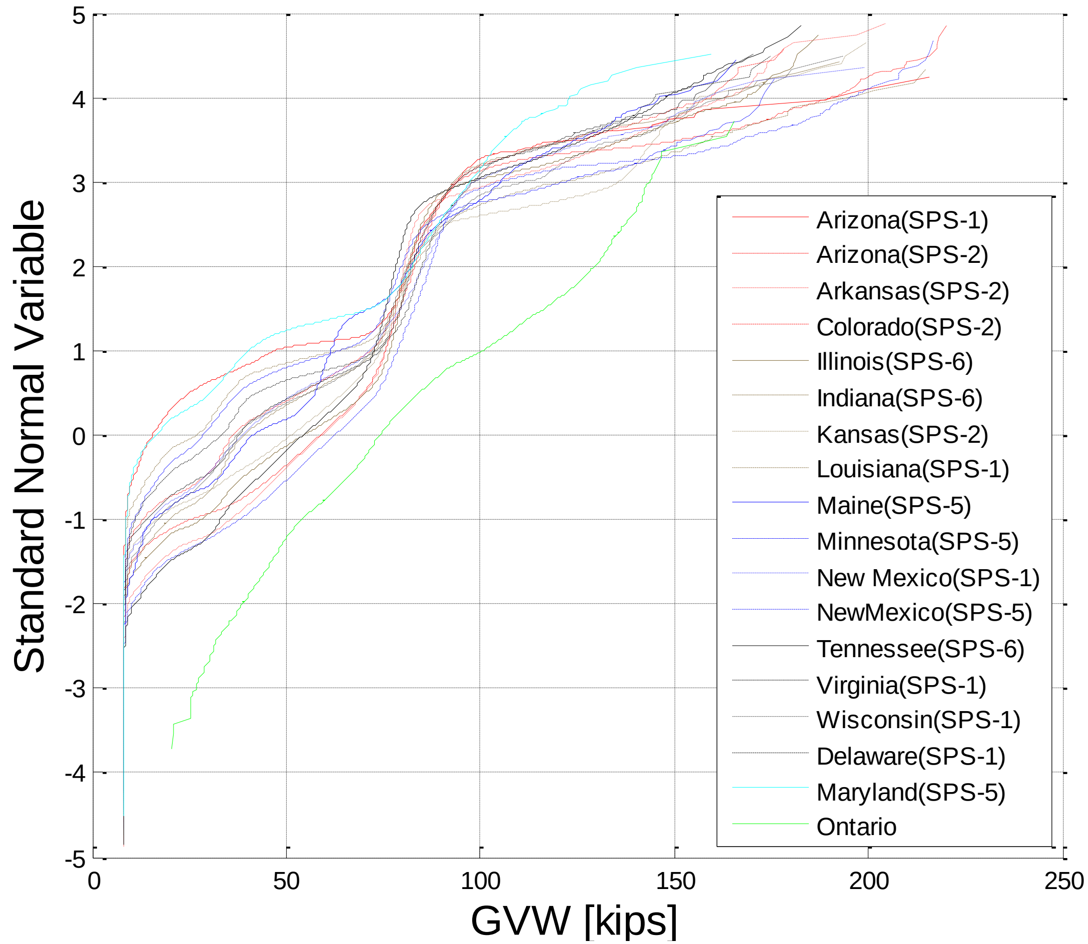
Vehicles with GVW <20 Kips

Filtering By Limit State

- Vehicles passing **Filters #1** & **#2** were used for calibration of all limit states **except for fatigue and the limit state for permit vehicles.**
- Vehicles filtered by **Filter #2** were considered permit vehicles.
- Vehicles passing **all three filters** were used for the fatigue limit state.

WIM Data - FHWA

- **14 sites** – Representing 1 year of traffic
- **Indiana** site: 6 months of traffic
- **New Mexico** sites: 8 months of traffic
- The **maximum recorded GVW** is 220 kips
- Mean values range from **20 to 65 kips**



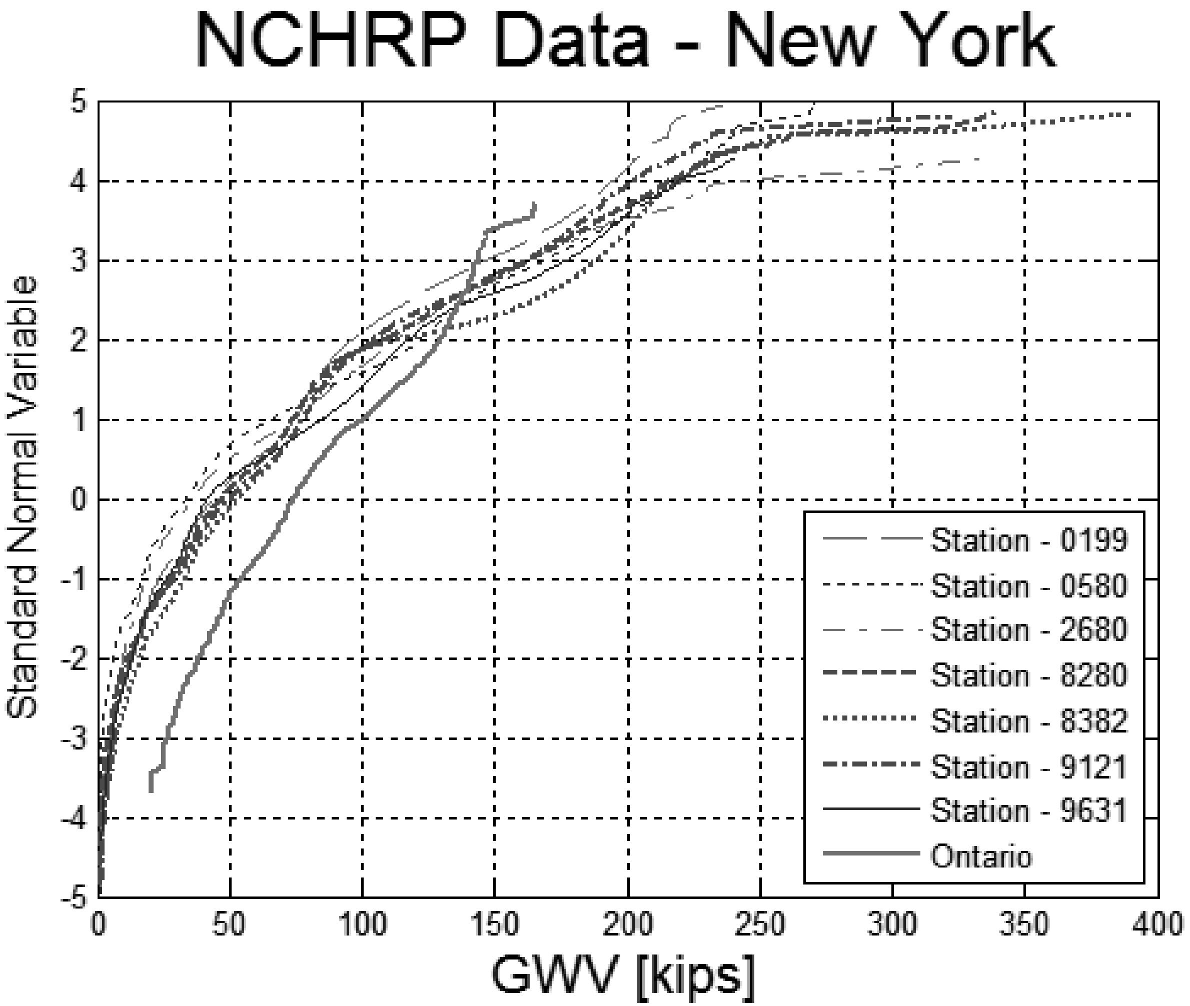
Analysis of the WIM Data

- Live Load effect – maximum moment and shear
- Simple spans with span lengths of 30, 60, 90, 120 and 200 feet
- Trucks causing moments or shears < 0.15 (HL93) were removed due to their insignificance

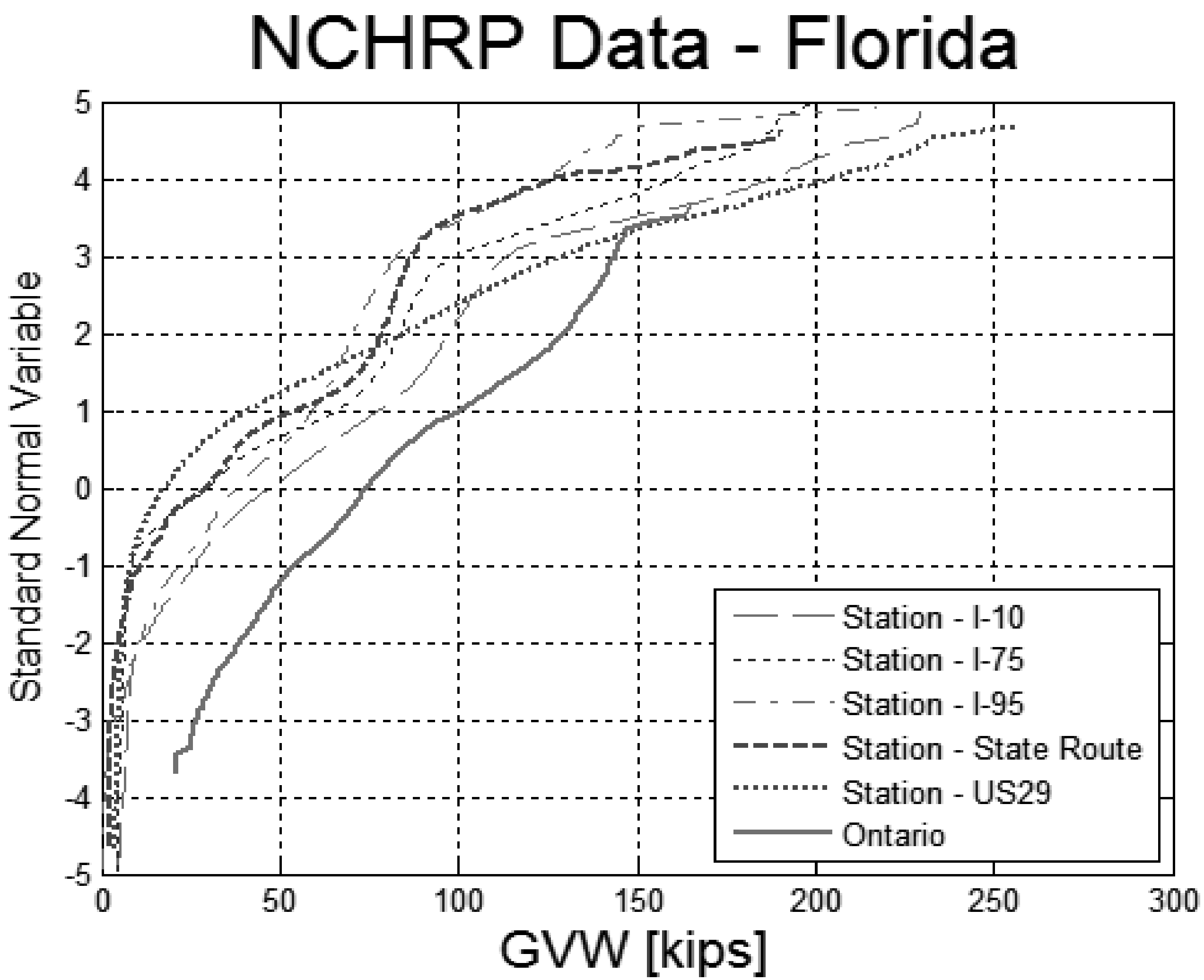
NCHRP 12-76 GVW Data

State	Number of Sites	Months of Data	Maximum GVW (kips)	Mean-Value Range (kips)
Oregon	4	4	200	43 - 52
Florida	5	12	250	20 - 50
Indiana	5	12	250	25 - 57
Mississippi	5	12	260	38 - 57
California	2	8.7	250	40 - 50
	1	7		
New York	7	12	380	35 - 50

Moment and Return Periods



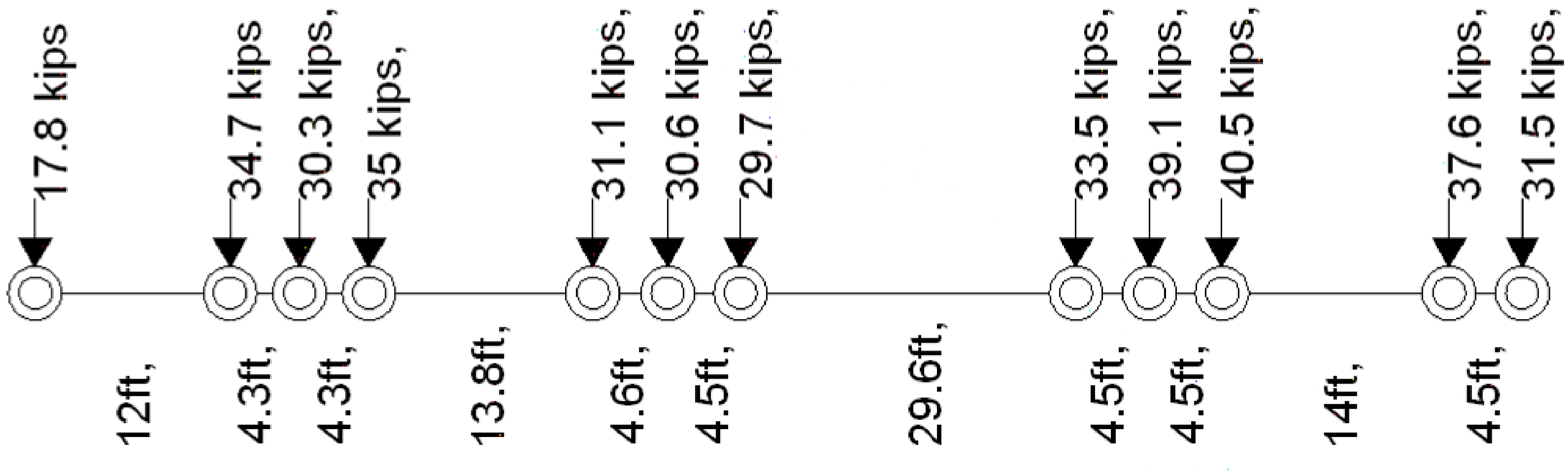
Moment and Return Periods



Analysis of the WIM Data

- Trends were **similar** for most sites (similar curve shape and cumulative distribution function, CDF).
- Some sites in New York included **large number of heavy vehicles.**
- Inclusion of these sites would **distort the calibration** for strength for the entire country. Not as important for service.
- A decision was made **not to include the data from New York** in the calibration.

Example of a Heavy Vehicle



- The total length of the truck is 100.6ft.
- GVW is 391.4 kips.
- Vehicle should be categorized as a special permit vehicle.

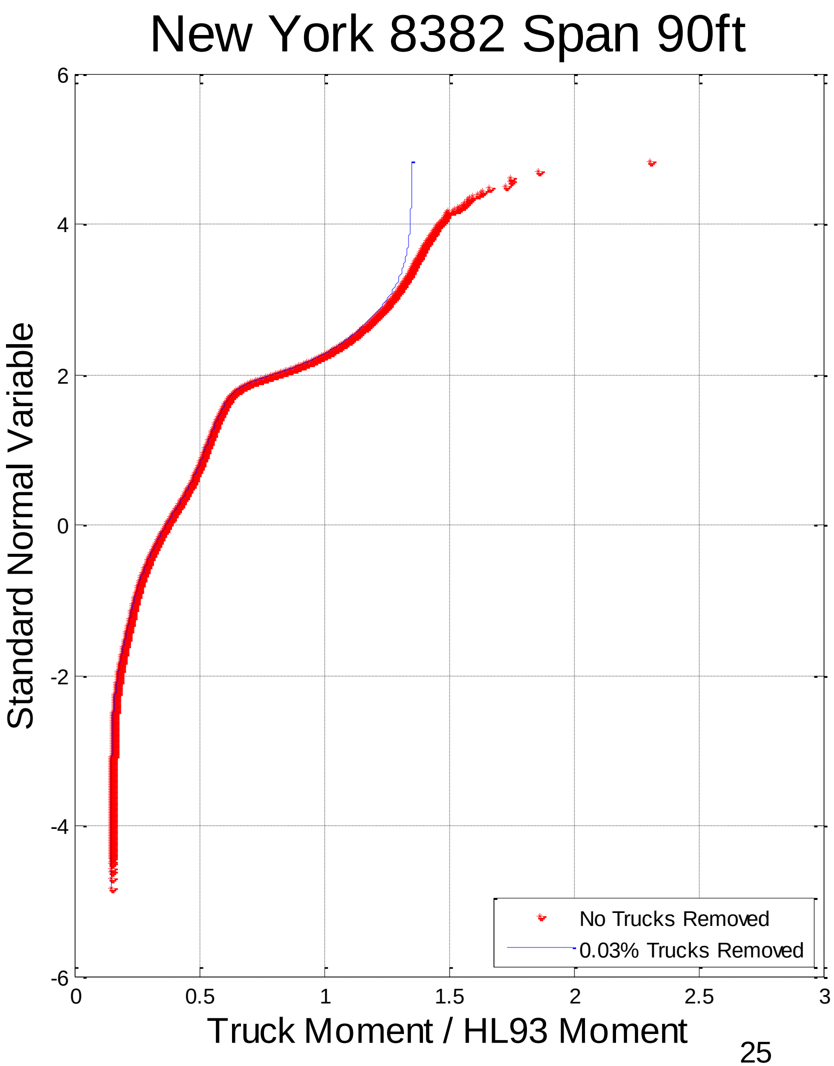
Removal of Heavy Vehicles

State	Location	Number of trucks before filtering	Number of trucks after filtering	Number of removed trucks	Percent of removed trucks
NY	0580	2,474,407	2,468,952	5455	0.22%
NY	2680	89,286	89,250	36	0.04%
NY	8280	1,717,972	1,717,428	544	0.03%
NY	8382	1,551,454	1,550,914	540	0.03%
NY	9121	1,235,963	1,235,886	77	0.01%
MS	I-10	2,103,302	2,103,300	2	0.00%

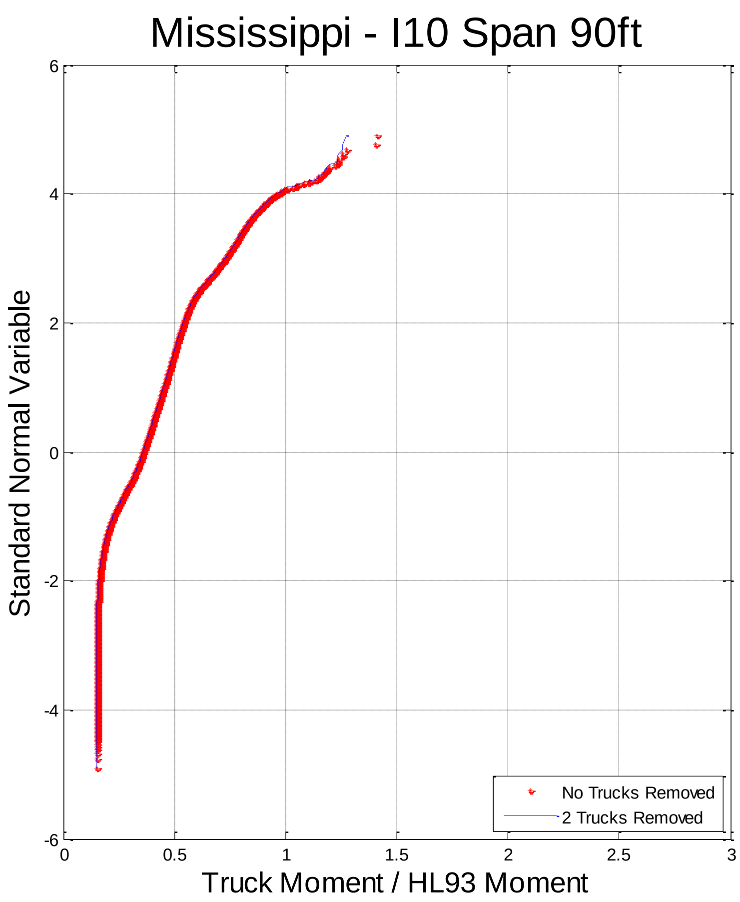
Removal of Heavy Vehicles

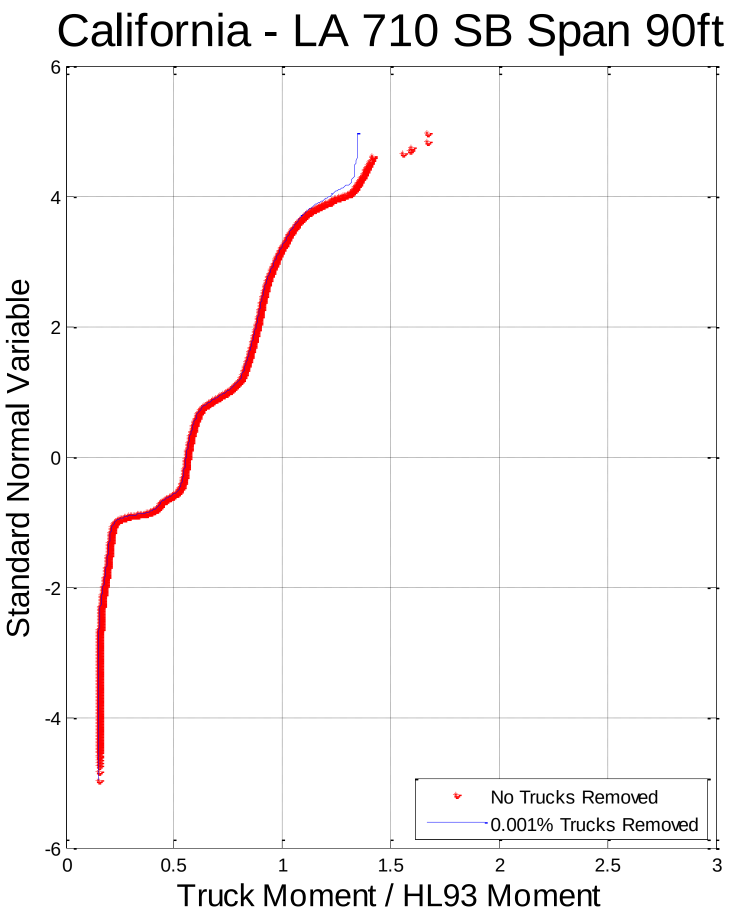
- **Filter** – trucks causing moments or shears more than 1.35 (HL93 live load effect) were removed
- Number of trucks **before filtering** – 1,551,454
- Number of trucks **after filtering** – 1,550,914
- **Number** of removed trucks – 540
- **Percent** of removed trucks – 0.03%

(Note that if six heaviest trucks were removed, the bias would drop from 2.35 to 1.65)

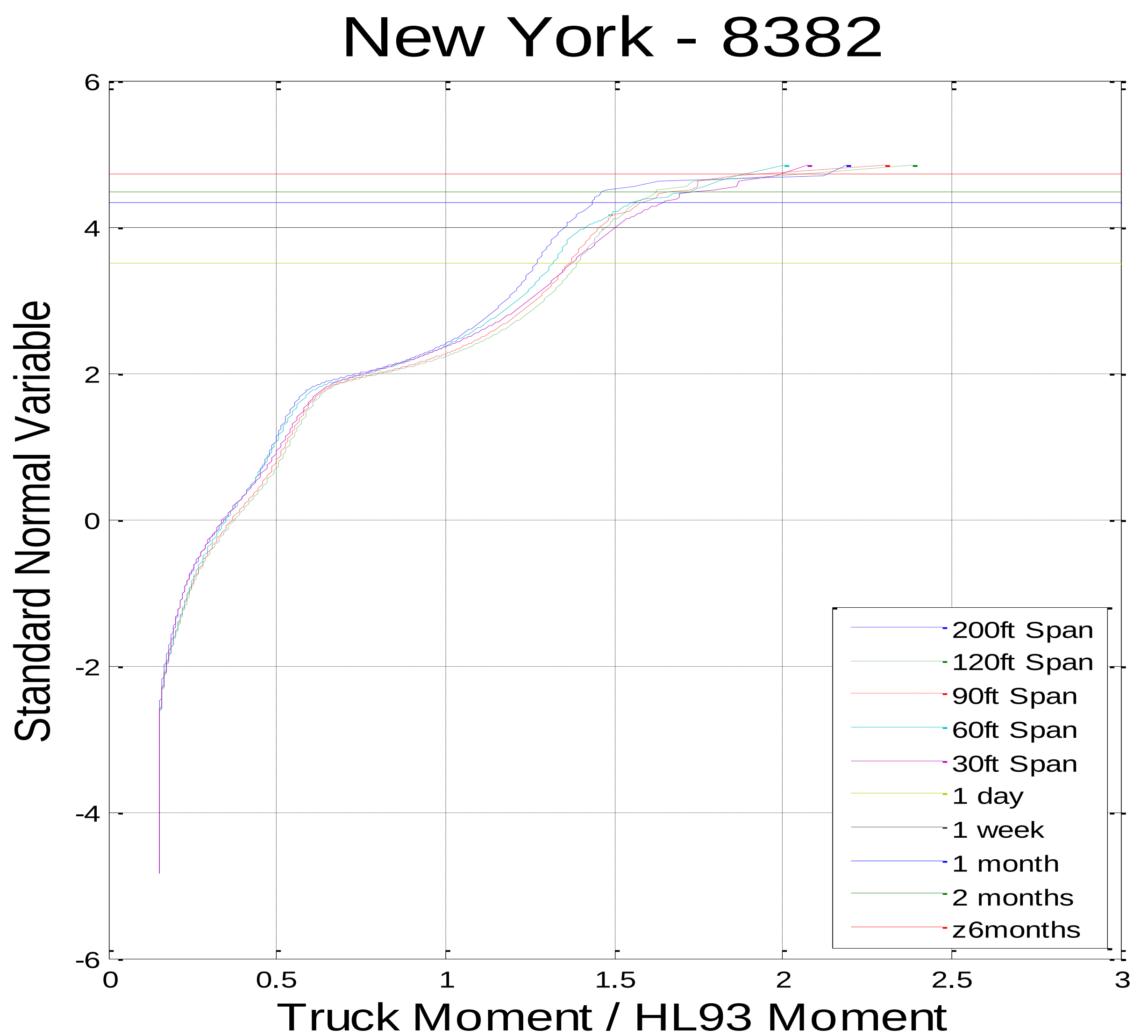


Other Examples of Removal





Moment and Return Periods



WIM Input for Service II for Steel

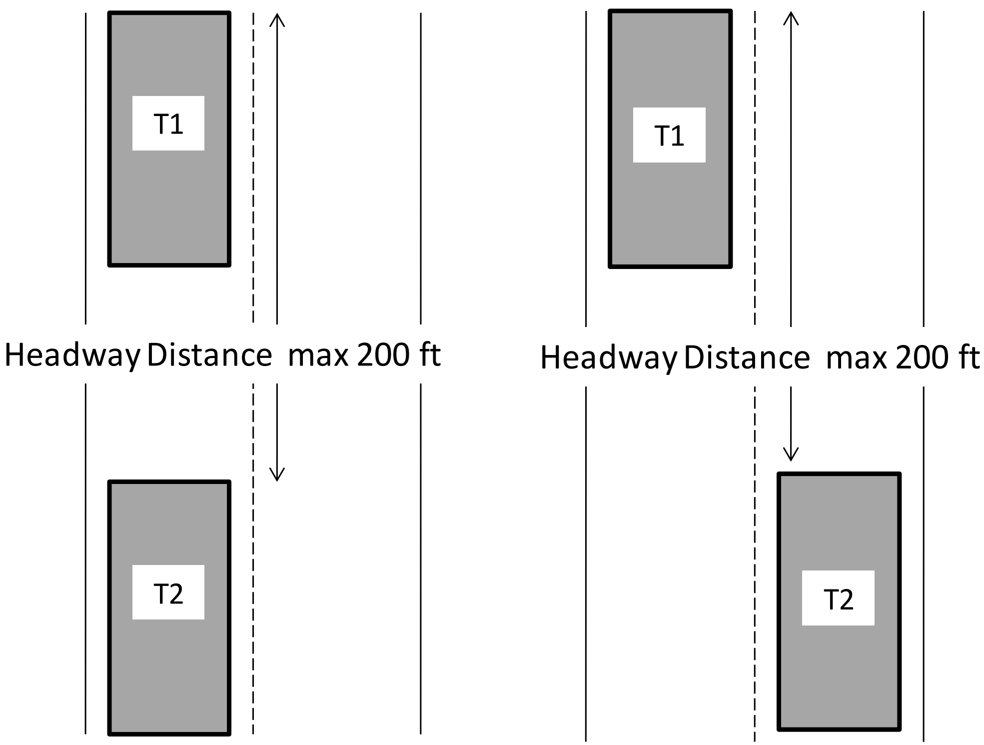
- After Filter #1 – includes "Permit Loads"
- Prior elimination of several sites
- 80% of "Permit Loads" came from one site – excluded it
- Ratios to HL93 (current = 1.30)
  - Ratio =1.1 yields average = ~1 per mo
  - Ratio = 1.2 yields average = <1 per 2 mo
  - Ratio = 1.3 yields average =<1 per 6 mo
- "DEEMED to Satisfy"
- Single Lane??

Multiple Presence

- WIM data analyzed to identify cases of:
  - Two trucks side-by-side
  - Two trucks in the same lane or adjacent lanes at less than 200 ft. headway
- Only cases of “correlated” trucks were considered:
  - Both trucks have the same number of axles
  - GVWs of the trucks are within +/- 5%
  - All corresponding spacings between axles are within +/- 10%
- Correlated trucks were used to match assumptions in original calibration

Multiple Presence Cases

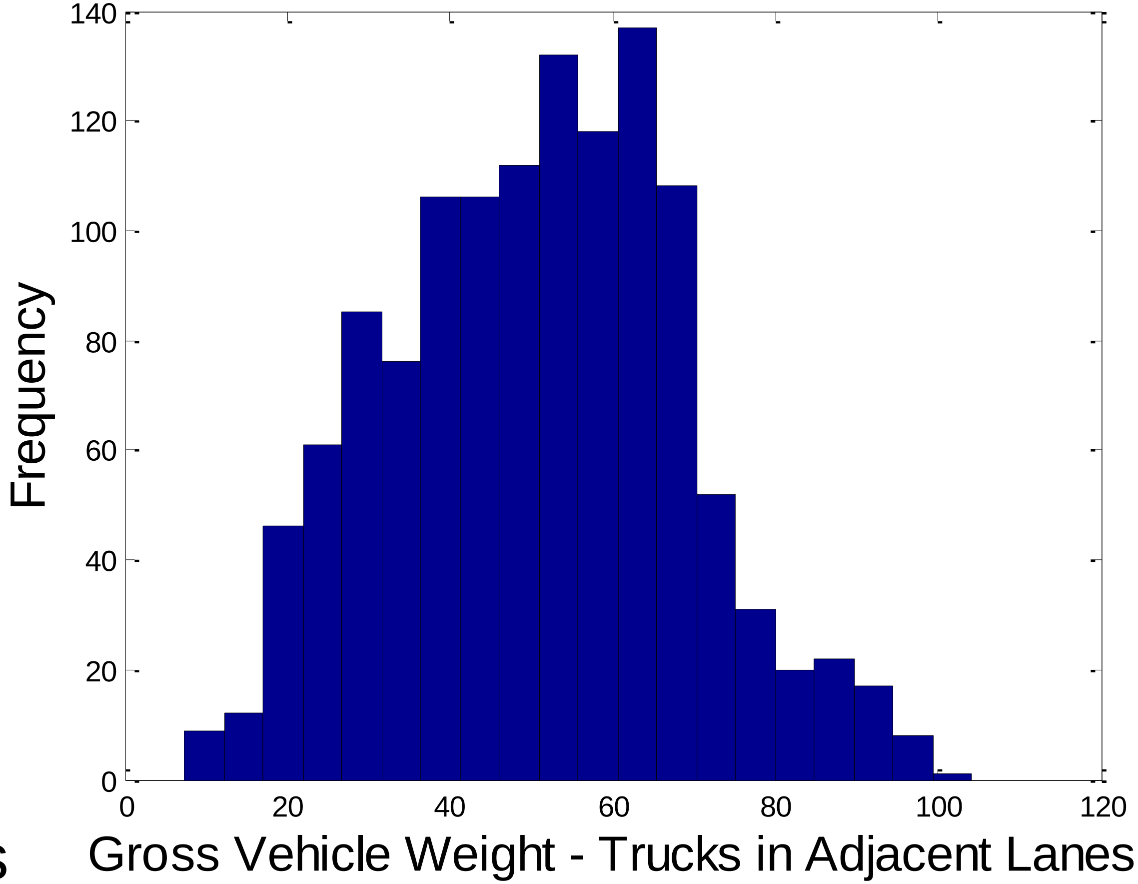
- Simultaneous occurrence of trucks on the bridge
- Filter based on time of a record and speed of the truck
- Distance from the first axis of first truck to the first axis of the second truck maximum 200 ft.



Two cases of the simultaneous occurrence

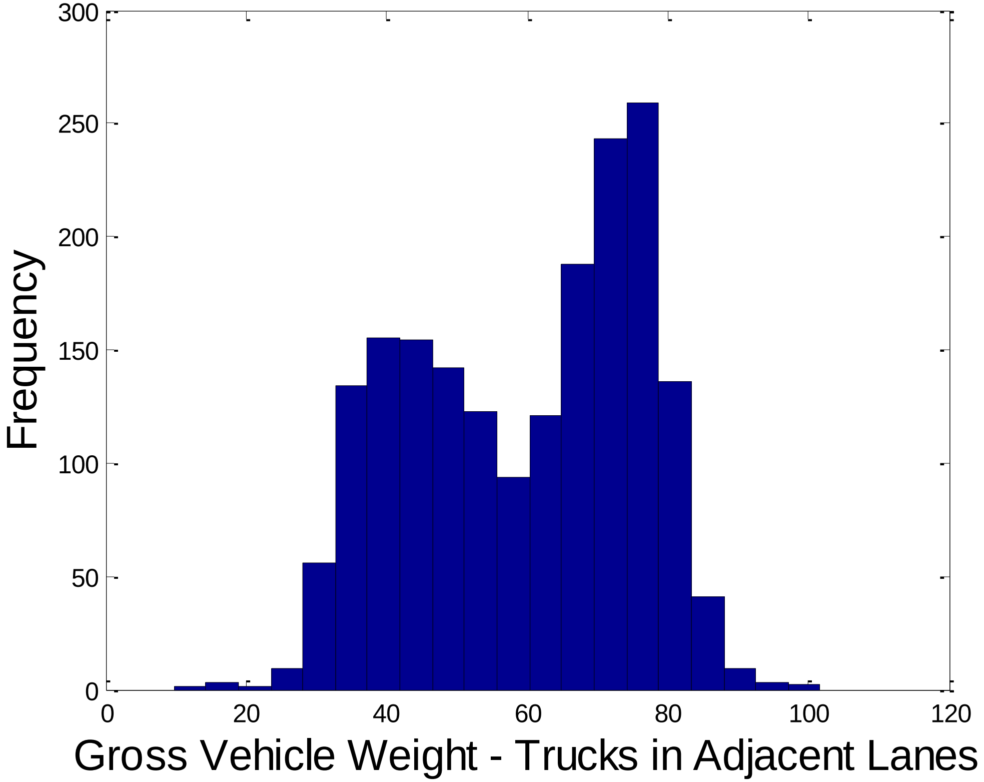
Adjacent Lanes - Florida

- Florida I-10 – Time record accuracy 1 second
- Number of Trucks : 1,654,004
- Number of Fully Correlated Trucks: 2,518
- Max GVW = 102 kips



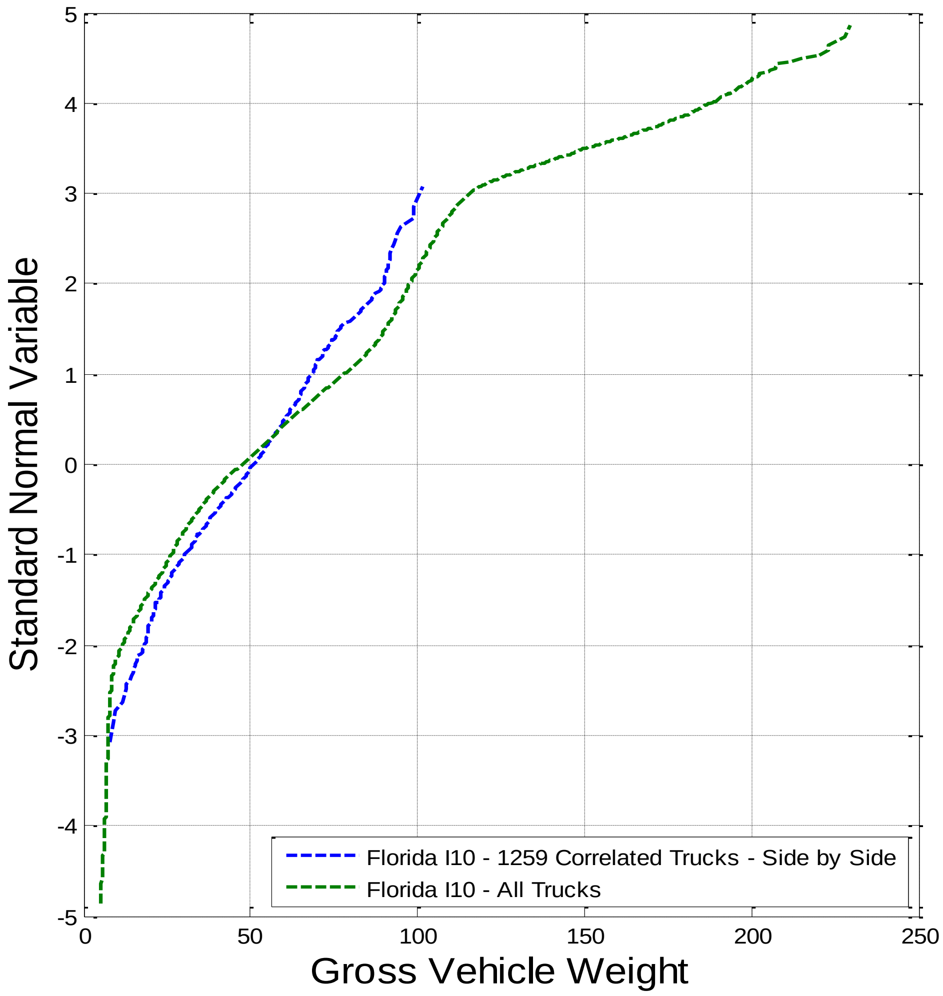
Adjacent Lanes – New York

- New York Weigh Station #8382
- Number of Trucks: 1,550,914
- Number of Fully Correlated Trucks: 3,748
- Max GVW = 102 kips



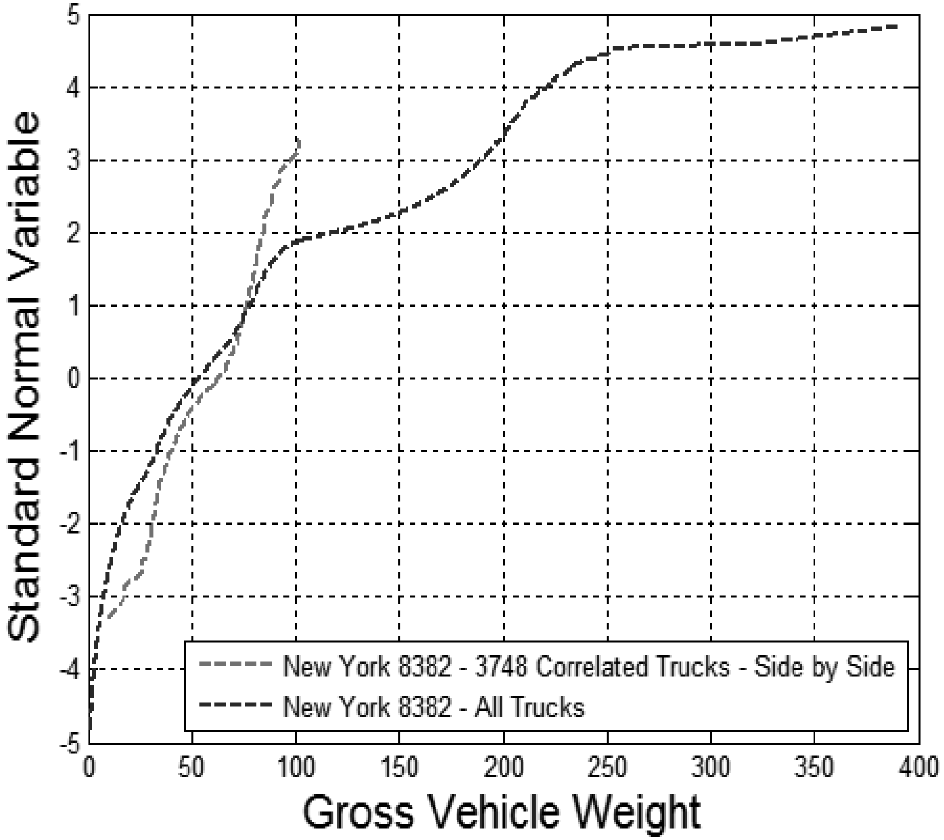
Adjacent Lanes – Florida I-10

2,518 trucks out of 1,654,000



One Lane – New York Weigh Station 8382

3,748 trucks out of 1,550,914



Conclusions for Multiple Presence

- Vehicles representing the extreme tails of the CDFs **need not be considered** to occur simultaneously in multiple lanes.
- For the **SLS, only a single-lane live-load model need be considered.**

Conclusion For Non-Fatigue SLS

- **Not necessary to envelop all trucks** – SLS expected to be exceeded occasionally
- Scaled HL- 93 looks reasonable
- Some **states with less weight enforcement** may have to have additional consideration

Example of Live Load Parameters

For general case (non-fatigue limit states/no permit vehicle):

- Parameters vary with span length , ADTT and period
- For example, for:

  120 ft span, 1 year

  and 5000 ADTT:

  - Bias: 1.36
  - COV: 0.09

Bias Table for ADTT 5000

Span (ft)	1 year	5 years	50 years	75 years
30 ft	1.35	1.39	1.41	1.42
60 ft	1.38	1.4	1.44	1.45
90 ft	1.38	1.4	1.44	1.45
120 ft	1.36	1.41	1.46	1.46
200 ft	1.31	1.34	1.39	1.4
300 ft	1.25	1.28	1.3	1.31

Statistics of Non-Fatigue SLS Live Load

- Based on 95% limit:
  - ADTT = l,000, Bias on HL 93 = 1.4
  - ADTT = 5,000, Bias on HL 93 = 1.45
- COV = 12%
- Based on 100 years – Bias varies with time interval, which will be reflected in calibrated load factor
- Bias not strongly influenced by span length

Fatigue II: Miner's Law

$$M_{eff} = \sqrt[3]{\sum_{i=1}^{n}\left(p_i * m_i^3\right)}$$

- M_{eff} = the equivalent moment cycle load
- m_i = the incremental moment cycle
- p_i = the probability of occurrence of m_i

Miner's Law

$$M_{eq} = \sqrt[3]{\frac{1}{N}\sum\left(n * m_i^3\right)}$$

- M_{eq} = the equivalent constant amplitude moment range
- m_i = a particular recurring moment range
- n = number of recurrences of m_i
- N = total number of cycles in the data period

Miner’s Law

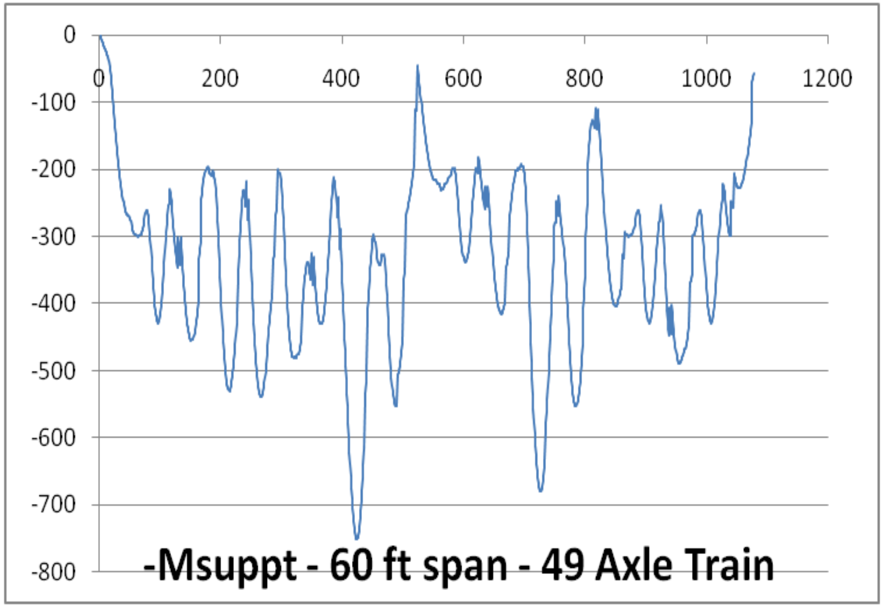
- For constant amplitude moment cycles, Miner’s Law can be used to **determine a different magnitude** of the moment and the **associated number of cycles** that will give the same M_{eff}

Rainflow Method

- Rainflow method is **used to convert** a random stress diagram into a series of stress cycles; each has equal positive and negative peaks

Live Load For Fatigue II (Finite Fatigue Life) *(continued)*

- Using WIM data for axle loads, spacing, speed and time, all axles in the WIM data for each site were placed in one continuous axle train.
- For each site, the axle train was run on spans of different lengths.



Live Load For Fatigue II (Finite Fatigue Life) *(continued)*

- Moment value vs. time were developed.
- Rainflow counting used to convert data to full cycles of different magnitudes.
- Miner's Law yields one effective moment per span with the number of cycles from the rainflow.
- Miner's Law used to determine the number of cycles from the design truck that yields same effective moment and the associated load factor.

Live Load For Fatigue II (Finite Fatigue Life) *(continued)*

- Variety of spans and locations yields Mean, bias and COV

Examples Using FHWA WIM Data At Three Sites

$$M_{eq} = \sqrt[3]{\sum_{i=1}^{n} \left(p_i * m_i^3 \right)}$$

M_{eff} [kip-ft] for 3 sites				
30 ft (-184)*	60 ft (-360)*	90 ft (-530)*	120 ft (-762)*	200 ft (-1342)*
-83	-204	-269	-408	-845
-90	-215	-300	-452	-896
-86	-217	-291	-439	-916

* Values in parentheses = then-current AASHTO fatigue moment

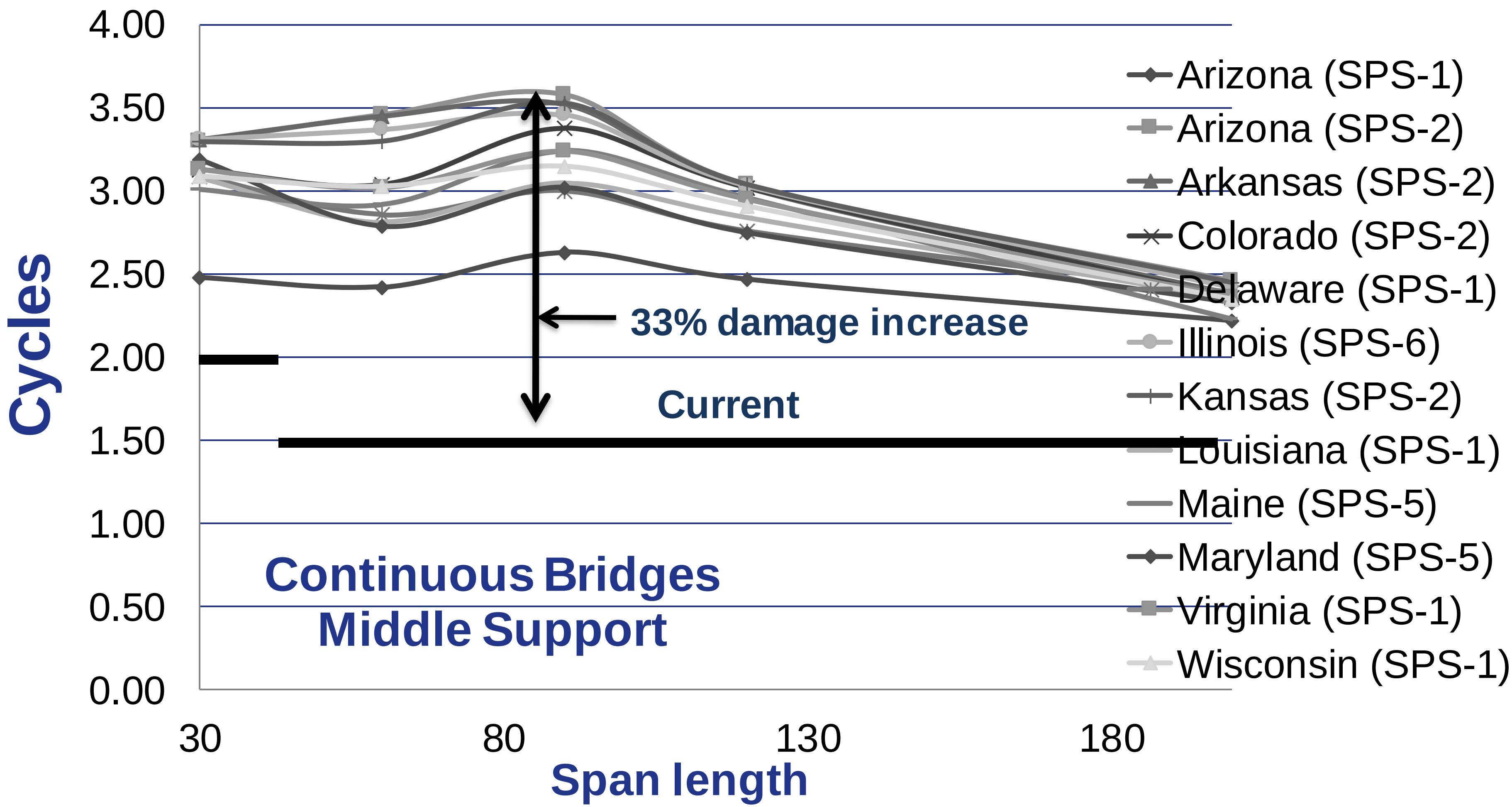
Example Using FHWA WIM Data At Three Sites *(continued)*

$$M_{eq} / M_{Fat-Trk}$$

Fatigue II Load Factors for 3 sites				
30 ft	**60 ft**	**90 ft**	**120 ft**	**200 ft**
0.45	0.56	0.51	0.54	0.63
0.49	0.60	0.57	0.59	0.67
0.47	0.60	0.55	0.58	0.68

Cycles per passage will be incorporated and the load factors associated with the number of cycles will be compared.

Cycles Per Passage



Rainflow Cycles - n_{rc}

Continuous Spans				
30 ft	**60 ft**	**90 ft**	**120 ft**	**200 ft**
3.13	3.03	3.38	3.02	2.36
3.09	2.85	3.00	2.76	2.38
3.30	3.30	3.52	3.04	2.44

Damage Factor Compared to Then-Current Damage Factor

$$\left(M_{eq} / M_{Fat-Trk}\right)\sqrt[3]{\frac{n_{rc}}{n_{AASHTO}}}$$

Then-Current = 0.75				
30 ft	**60 ft**	**90 ft**	**120 ft**	**200 ft**
0.52	0.71	0.66	0.68	0.73
0.57	0.74	0.71	.73	0.78
0.55	0.78	0.73	0.73	0.80

High for all sites = 0.87 or 116% of current

Design Cycles Per Truck

Then-Current ➡

Longitudinal Members		Span Length > 40 ft	Span Length ≤ 40 ft
Simple Span Girders		1.0	2.0
Continuous Girders	near interior support	1.5	2.0
	elsewhere	1.0	2.0

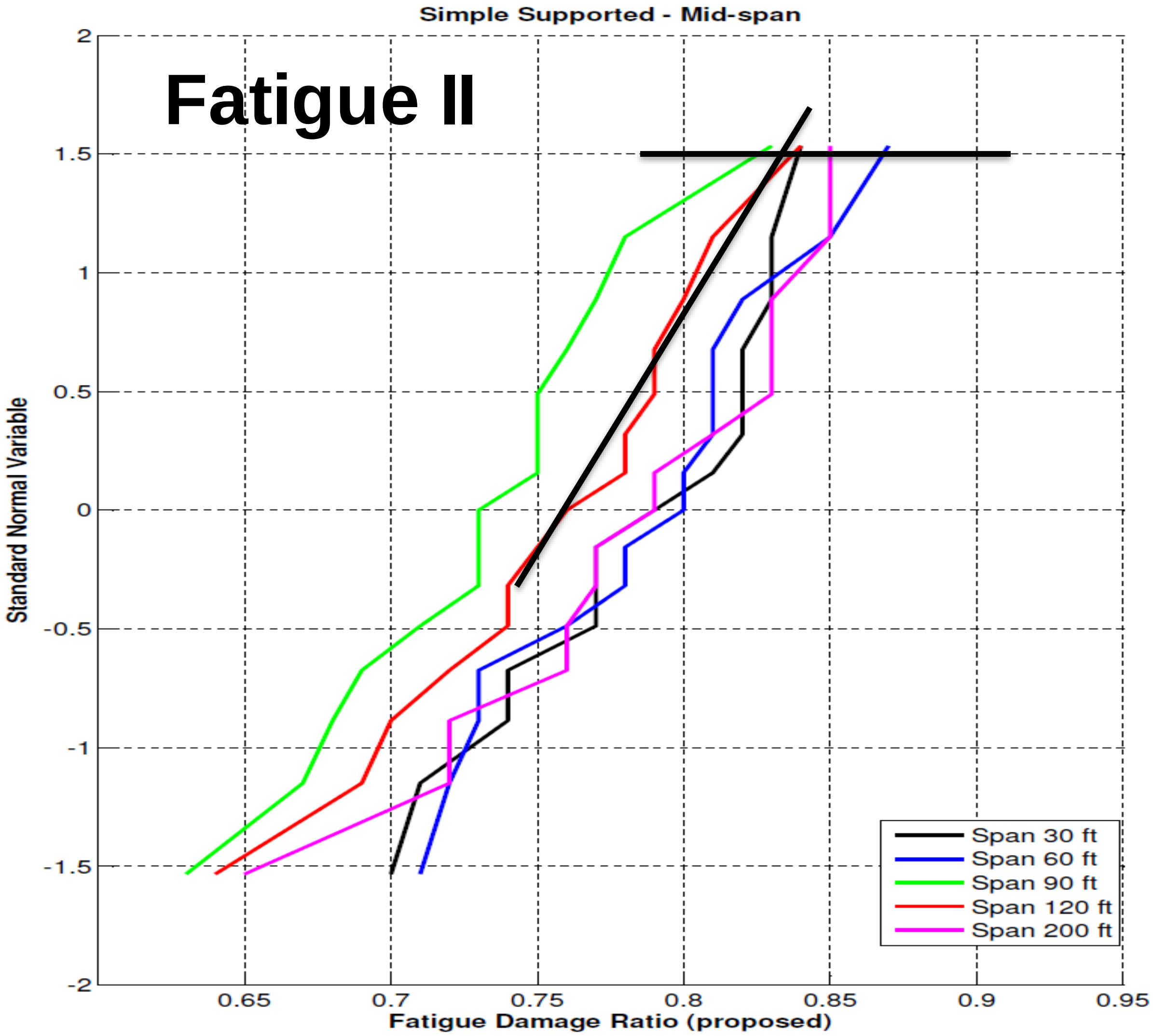
New ➡

Longitudinal Members		*n*
Simple Span Girders		1.0
Continuous Girders	near interior support	1.5
	elsewhere	1.0

Improved Damage Ratios

Simple Support – mid-span	Fatigue Damage Ratio (proposed)				
	30	60	90	120	200
Arizona (SPS-1)	0.81	0.87	0.83	0.84	0.85
Arizona (SPS-2)	0.83	0.81	0.77	0.81	0.85
Arkansas (SPS-2)	0.82	0.81	0.76	0.80	0.83
Colorado (SPS-2)	0.74	0.73	0.69	0.72	0.76
Delaware (SPS-1)	0.83	0.85	0.78	0.78	0.79
Illinois (SPS-6)	0.82	0.81	0.75	0.79	0.83
Kansas (SPS-2)	0.79	0.80	0.75	0.79	0.83
Louisiana (SPS-1)	0.77	0.78	0.73	0.74	0.76
Maine (SPS-5)	0.71	0.72	0.67	0.69	0.72
Maryland (SPS-5)	0.70	0.71	0.63	0.64	0.65
Minnesota (SPS-5)	0.74	0.73	0.68	0.70	0.72
Penn (SPS-6)	0.84	0.82	0.75	0.78	0.81
Tennessee (SPS-6)	0.82	0.78	0.73	0.76	0.79
Virginia (SPS-1)	0.77	0.76	0.71	0.74	0.77
Wisconsin (SPS-1)	0.77	0.80	0.73	0.75	0.77

Calculate COV and Mean + 1.5 Std Dev



Continuous Spans Results Similar

Fatigue Damage Ratio

Fatigue Damage Ratio (proposed) for Fatigue II LS				
	Span	Mean	Mean+1.5 σ	COV
Simply Supported Mid-span	30 ft	0.785	0.87	0.07
	60 ft	0.78	0.86	0.06
	90 ft	0.73	0.81	0.07
	120 ft	0.76	0.84	0.07
	200 ft	0.78	0.86	0.07
Continuous Middle Sup.	30 ft	0.59	0.65	0.07
	60 ft	0.74	0.82	0.07
	90 ft	0.69	0.77	0.07
	120 ft	0.71	0.78	0.06
	200 ft	0.785	0.87	0.07
Continuous 0.4 L	30 ft	0.73	0.81	0.07
	60 ft	0.72	0.80	0.07
	90 ft	0.68	0.75	0.07
	120 ft	0.72	0.79	0.06
	200 ft	0.76	0.84	0.07

Independent Check of WIM Data Processing

- Actual traffic was run on the simulated bridges:
  - Traffic simulation: All filtered trucks at a site were positioned relative to each other using the time stamps and speed in the WIM data.
  - Not individual trucks one at a time.
- Test axle train evaluated by two groups:
  - 8 hypothetical trucks
  - 49 axles
  - 963 ft
  - 843,000 lbs

Independent Check of WIM Data Processing *(continued)*

- Test cobbled together existing pieces:
  - Used rainflow counting algorithm based on ASTM E 1049 – 85 previously developed to process instrumentation data for repair of in-service bridge to calculate cycles per truck; and
  - Miner's Law to calculate M_{eff}.

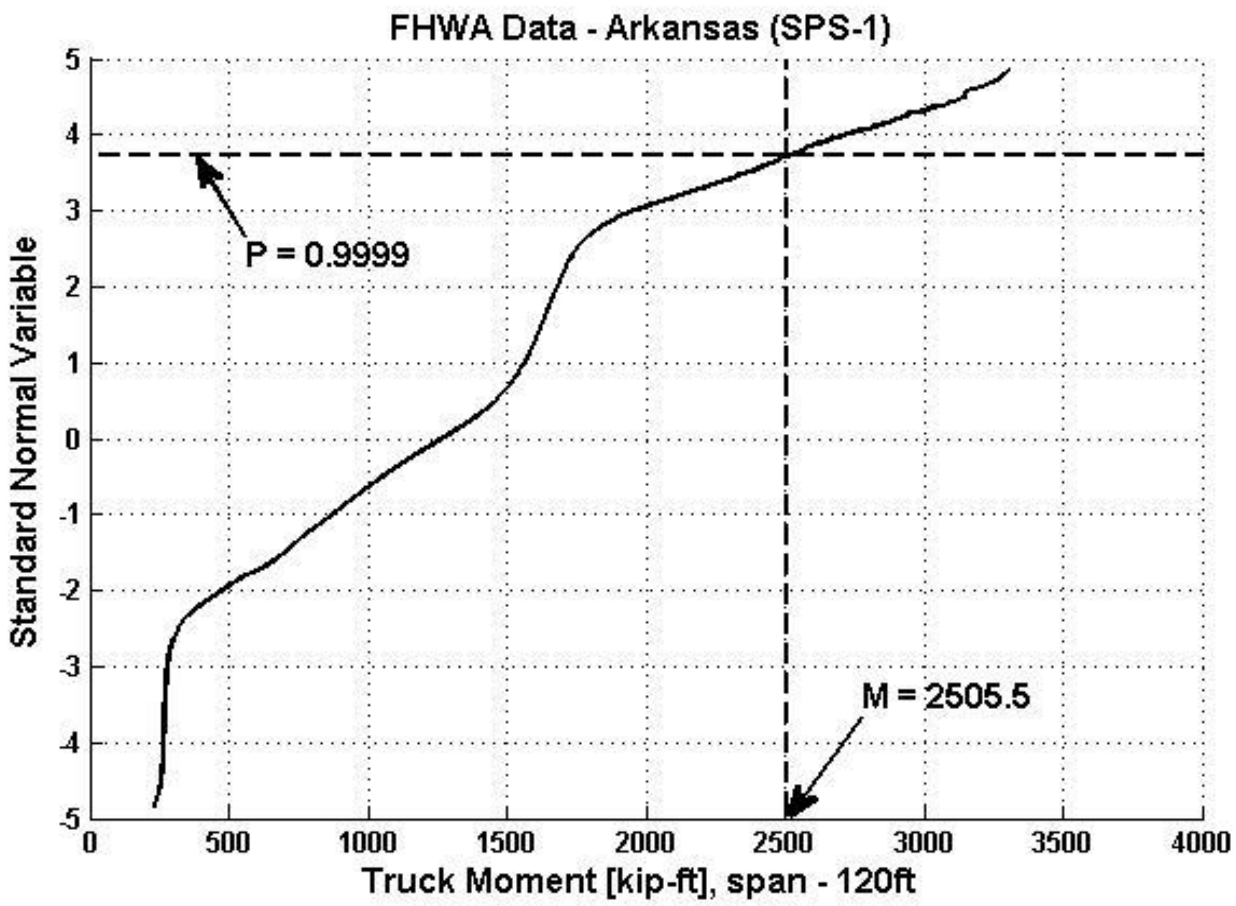
Independent Check of WIM Data Processing *(continued)*

- **Results:**
  - Only a few issues
  - Final results – damage factors – same for simple span, very close for negative moment at pier of continuous
  - Sometimes intermediate results varied – seemed to depend on maximum magnitude of small cycles (noise) that was ignored
- Generally, **test worked well.**

Fatigue I (Infinite Fatigue Life)

- Usually assumed that CAFL can be exceeded by 1/10,000 of the stress cycles
- 99.99% inclusion of normal random variables requires mean plus 3.8 standard deviations

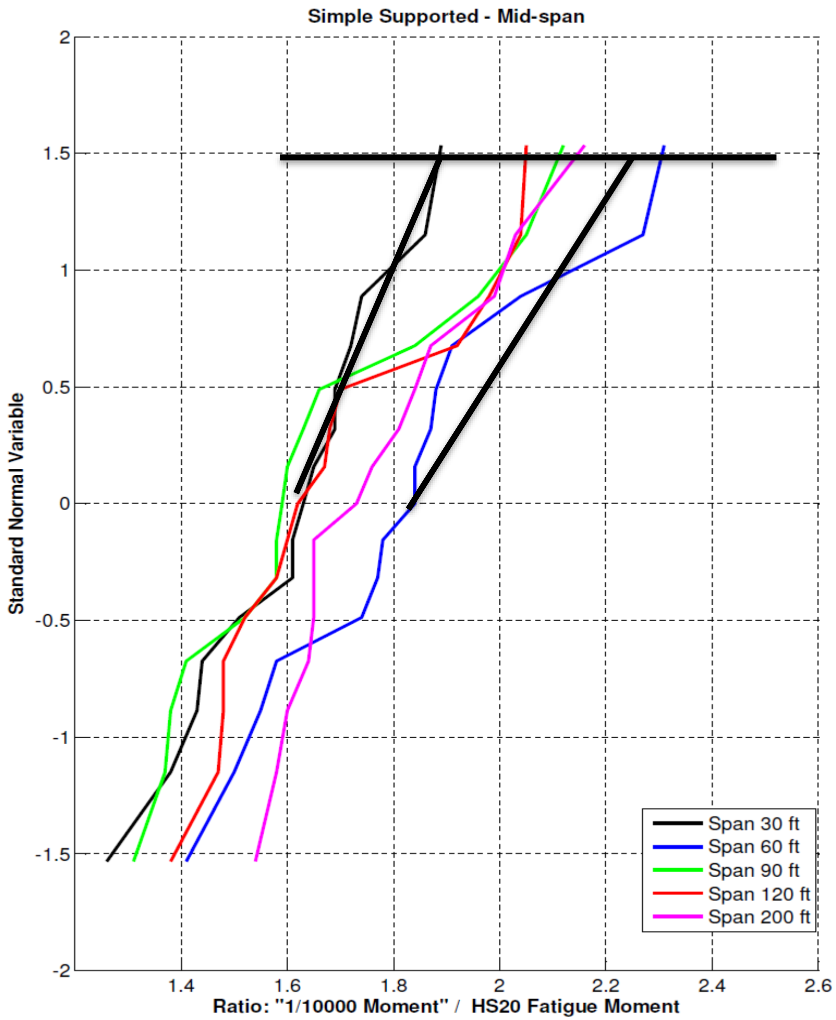
Find Corresponding Point in WIM Data



Site Moments Normalized to HS20

Simple Support - mid-span	"1/10000 Moment" / HS20 Fatigue Moment				
	30	60	90	120	200
Arizona (SPS-1)	1.74	1.84	1.63	1.70	1.84
Arizona (SPS-2)	1.26	1.41	1.31	1.38	1.54
Arkansas (SPS-2)	1.44	1.58	1.41	1.52	1.65
Colorado (SPS-2)	1.38	1.50	1.38	1.48	1.58
Delaware (SPS-1)	1.86	2.31	2.12	1.98	1.87
Illinois (SPS-6)	1.43	1.55	1.37	1.48	1.64
Kansas (SPS-2)	1.69	1.87	1.84	1.92	1.99
Louisiana (SPS-1)	1.89	2.27	1.96	2.05	2.16
Maine (SPS-5)	1.63	1.77	1.59	1.68	1.81
Maryland (SPS-5)	1.69	1.91	1.66	1.60	1.65
Minnesota (SPS-5)	1.61	2.04	2.05	2.04	2.03
Pennsylvania (SPS-6)	1.65	1.84	1.60	1.62	1.73
Tennessee (SPS-6)	1.72	1.88	1.52	1.47	1.60
Virginia (SPS-1)	1.51	1.74	1.58	1.58	1.65
Wisconsin (SPS-1)	1.61	1.78	1.58	1.67	1.76

Same Process



Continuous Spans Results Similar

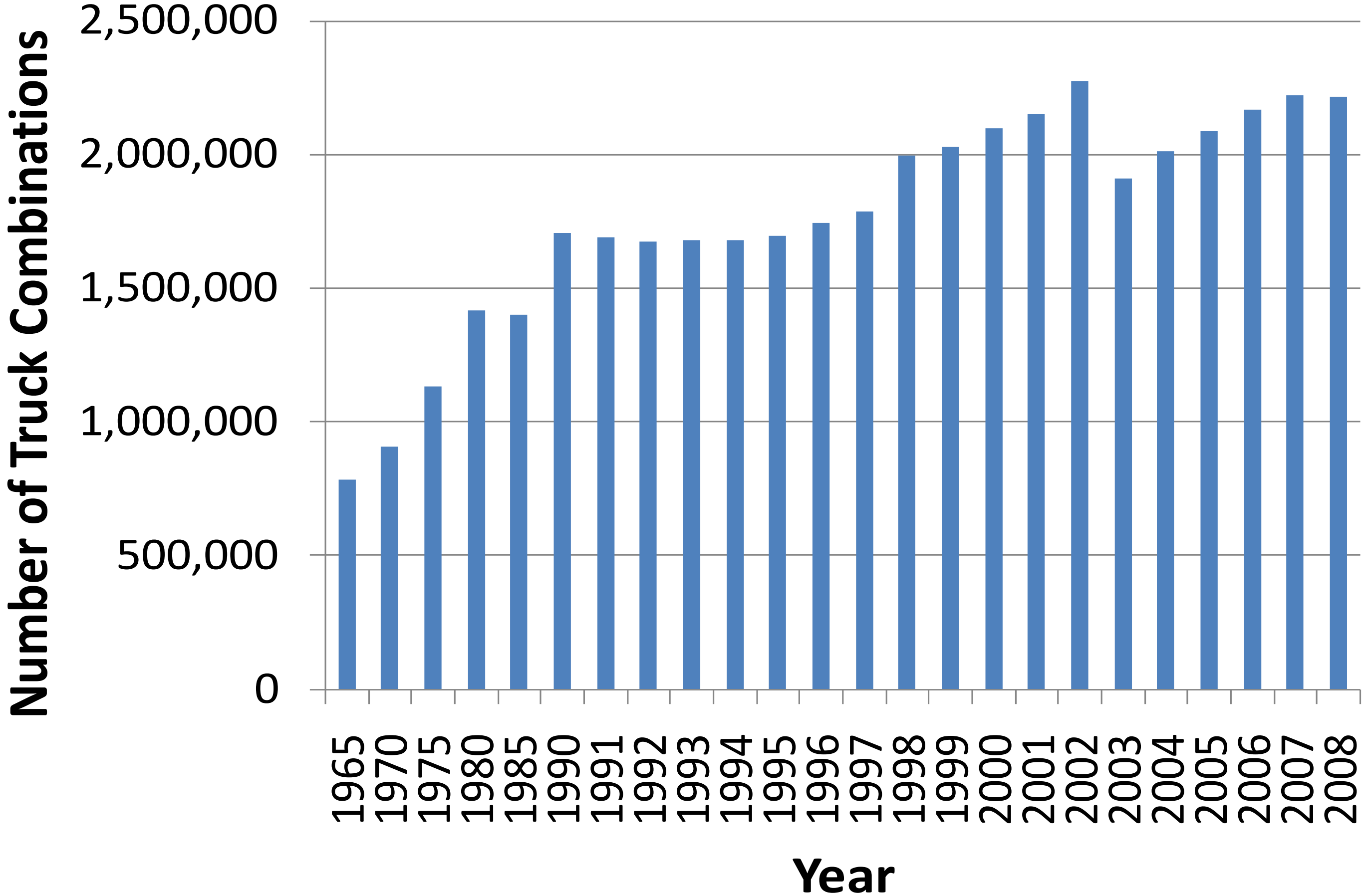
Maximum Moment Range Ratio for Fatigue I LS

The Maximum Moment Range Ratio for Fatigue I LS				
	Span	Mean	Mean+1.5 σ	COV
Simple Supported Mid-span	30 ft	1.6	1.90	0.13
	60 ft	1.83	2.24	0.15
	90 ft	1.6	1.96	0.15
	120 ft	1.64	1.88	0.10
	200 ft	1.7	2.15	0.18
Continuous Middle Sup.	30 ft	1.35	1.61	0.13
	60 ft	1.81	2.13	0.12
	90 ft	1.92	2.18	0.09
	120 ft	1.97	2.17	0.07
	200 ft	2.27	2.47	0.06
Continuous 0.4 L	30 ft	1.54	1.86	0.14
	60 ft	1.67	2.06	0.16
	90 ft	1.6	1.92	0.13
	120 ft	1.65	1.97	0.13
	200 ft	1.72	2.11	0.15

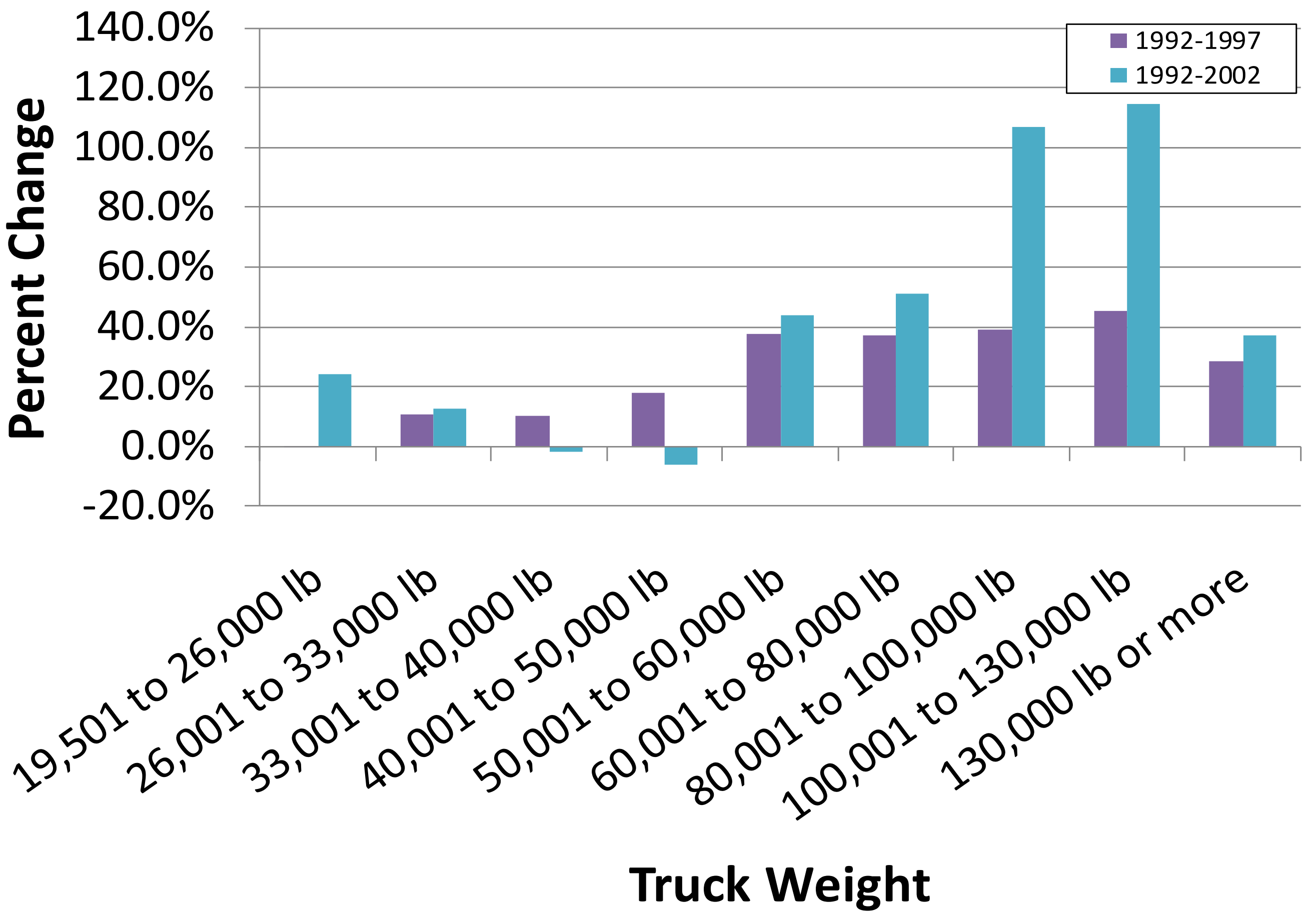
Service Limit State Design-Proposed Fatigue Load Factors

- Fatigue I: 2.0 (instead of the then-current 1.5)
- Fatigue II: 0.8 (instead of the then-current 0.75)

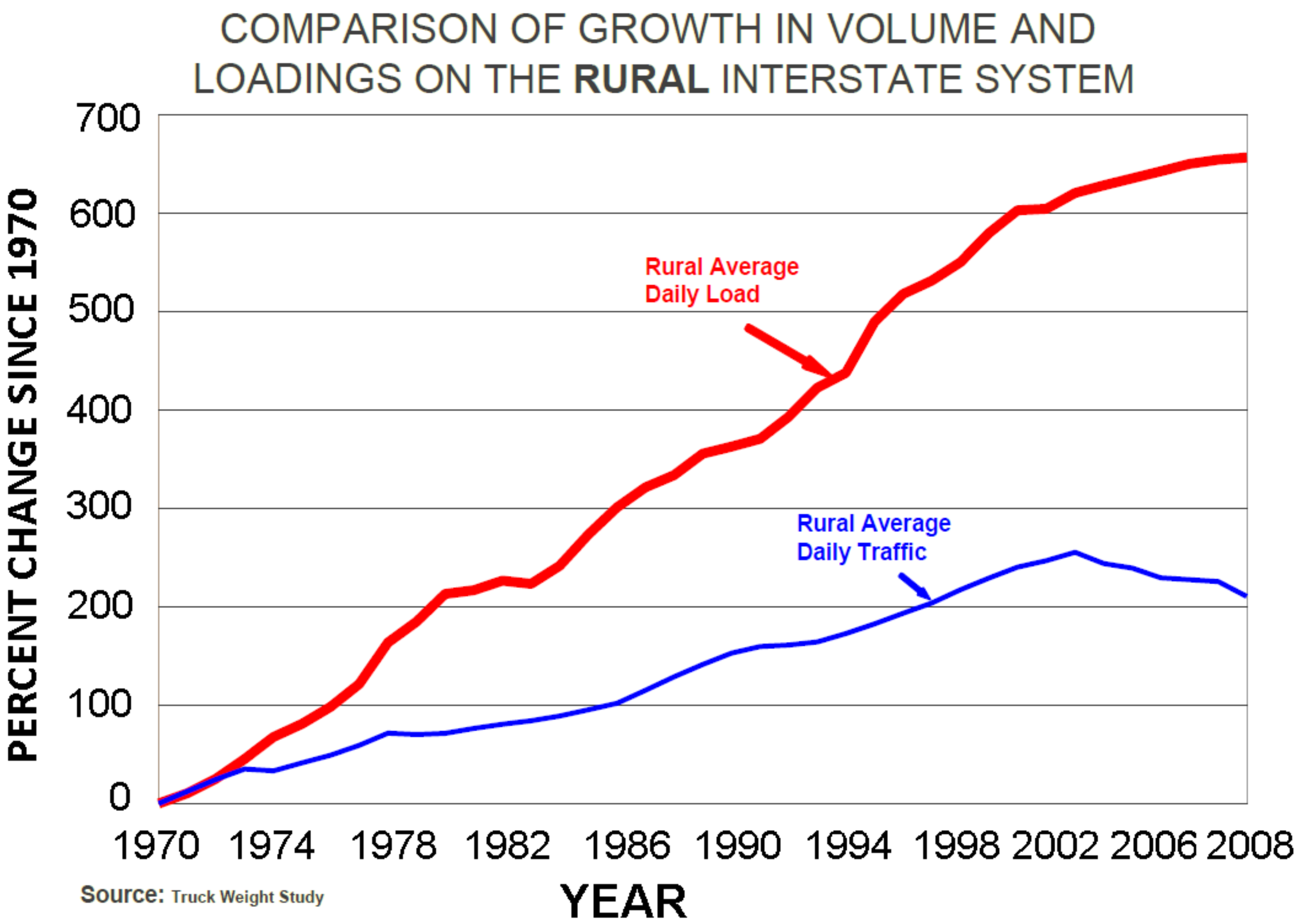
Does This Increase Make Sense?



Does This Increase Make Sense? *(continued)*



Does This Increase Make Sense? *(continued)*



Does This Increase Make Sense? *(continued)*

- **Total number** of trucks increased
- The **most percentage increase** is in the heavy trucks category
- The total **load moved by trucks is increasing significantly faster than the number of trucks,** indicating that trucks are less likely to travel unloaded

Does This Increase Make Sense? *(continued)*

- **Changes in traffic patterns** indicate that current traffic produces higher fatigue damage and calls for a higher load factor for fatigue, which was confirmed by this part of the study.
- However, **the 2.0 load factor for Fatigue I can change many details from infinite life to finite life.** The fatigue life of many of these details may appear to have been consumed even though no fatigue cracking have been observed.

Does This Increase Make Sense? *(continued)*

- Further statistical studies performed after the completion of *Service Limit State Design* (R19B) confirmed that the number of WIM sites included in the study warrants the reduction of the degree of conservatism included in the study.
- The additional studies yielded a lower fatigue load factor for Fatigue I (1.75).
- The fatigue load factor proposed by R19B for Fatigue II (0.8) did not change.

Live Load Calibration for Service II

- Service II is applicable to steel structures only.
- It is meant to prevent yielding of steel components under service loads.

Live Load Calibration for Service II *(continued)*

- Service II live load factor = 1.3
- WIM moments analyzed to determine the frequency of the HL93 moments are exceeded
- One site has disproportionately high frequency (FL 29)
- FL DOT indicated that other highways in the vicinity were closed and traffic was diverted to this route
- This station was excluded from the analysis

Live Load Calibration for Service II (continued)

	MOMENT														
	Ratio Truck/HL-93 >= 1.1					Ratio Truck/HL-93 >= 1.2					Ratio Truck/HL-93 >= 1.3				
Site	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft
AZ SPS-1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
AZ SPS-2	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
AR SPS-2	2	7	3	0	0	0	3	0	0	0	0	0	0	0	0
CO SPS-2	0	2	5	4	0	0	0	2	0	0	0	0	0	0	0
DE SPS-1	36	33	22	11	0	10	22	10	1	0	1	11	1	0	0
IL SPS-6	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
IN SPS-6	3	11	11	10	2	2	4	5	4	0	0	0	1	0	0
KS SPS-2	16	33	35	31	2	7	16	17	7	0	6	7	6	0	0
LA SPS-1	44	6	12	14	7	26	6	7	7	0	6	6	5	4	0
ME SPS-5	4	4	5	2	0	0	4	2	0	0	0	2	0	0	0
MD SPS-5	5	6	2	2	0	0	1	1	0	0	0	1	0	0	0
MN SPS-5	7	5	6	5	0	4	2	2	1	0	2	1	1	0	0
NM SPS-1	0	1	1	1	0	0	0	0	0	0	0	0	0	0	0
NM SPS-5	3	1	1	2	0	2	0	0	0	0	0	0	0	0	0
PA SPS-6	32	22	17	14	1	13	17	13	1	0	3	13	2	0	0
TN SPS-6	53	4	4	0	0	5	1	0	0	0	1	0	0	0	0
VA SPS-	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
WI SPS-1	1	0	3	3	1	0	0	1	1	0	0	0	0	0	0
CA Antelope EB	0	1	0	0	5	0	0	0	0	0	0	0	0	0	0
CA Antelope WB	0	5	4	13	28	0	0	0	1	9	0	0	0	0	1
CA Bowman	0	0	0	1	1	0	0	0	0	1	0	0	0	0	0
CA LA-710 NB	1	31	50	51	15	0	6	24	19	0	0	0	4	1	0
CA LA-710 SB	1	17	45	48	14	0	3	18	19	0	0	0	1	1	0
CA Lodi	0	4	16	46	140	0	0	1	2	32	0	0	0	0	2
FL I-10	79	40	46	75	37	22	16	14	17	5	10	5	4	5	2
FL I-95	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
FL US-29	653	495	322	245	106	360	266	174	119	51	177	160	82	59	21
MS I-10	24	22	31	33	22	7	2	10	19	2	2	2	2	2	1
MS I-55UI	0	0	0	1	2	0	0	0	0	0	0	0	0	0	0
MS I-55R	19	30	48	58	32	7	8	16	21	19	2	3	5	8	9
MS US-49	0	0	2	1	0	0	0	0	0	0	0	0	0	0	0
MS US-61	0	0	1	2	1	0	0	1	1	0	0	0	0	0	0
	Ratio Truck/HL-93 >= 1.1					Ratio Truck/HL-93 >= 1.2					Ratio Truck/HL-93 >= 1.3				
	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft
Total W/O FL 29	331	285	373	430	310	105	111	144	121	68	33	51	32	21	15
Average per site per yr	10.7	9.2	12.0	13.9	10.0	3.4	3.6	4.6	3.9	2.2	1.1	1.6	1.0	0.7	0.5

Live Load Calibration for Service II *(continued)*

- Low frequency of WIM moment exceeding factored Service II moment (LL factor = 1.3)
- The frequency is higher for shorted spans (30 and 60 ft.) and decreased for longer spans.

Key Points

- WIM Data is an essential tool for modern load studies.
- Quality of data and the ability to check work are important factors in ensuring good results.
- HL93 Load Model is adequate to model current traffic for strength.
- For calibration of service limit state, a one lane load is sufficient (for design, multiple lanes should be used)
- Higher fatigue load factors are required.




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Introduction and Course Objectives

Lecture 4: Calibration of Service Limit States in AASHTO LRFD (1)
Service III Limit State

Wagdy Wassef, PhD, PE
AECOM

Eastern Federal Lands
June 27-28, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

- **Gain knowledge** of the background of the live load for tension in concrete in AASHTO LRFD
- **Gain knowledge** of the history of the prestressing losses in AASHTO LRFD
- **Recognize** the variation in reliability of existing structures and the need for calibration
- **Understand** the background of the selection of stress limits and load factor for calibration

Load Factor for Tension in Prestressed Concrete (Service III)

- Limits on tensile stresses in prestressed concrete components, when applied in conjunction with the LRFD loads and load factors, give answers similar to those determined using AASHTO standard specifications in effect at the time the AASHTO LRFD was developed.

Load Factor for Tension in Prestressed Concrete (Service III) *(continued)*

- Service limit states was not statistically calibrated.
- The consequences of the loads exceeding the resistance are not detrimental or well defined. (The effect of exceeding stress limit in PSC does not cause immediate failure; i.e., the limit state may be exceeded but the acceptable frequency of exceedance is not known.)

Load Factor for Tension in Prestressed Concrete (Service III) *(continued)*

- During the development of AASHTO LRFD using:
  - The typical load factor of 1.0 for service
  - AASHTO LRFD live load model and load distribution
  - Same method for determining prestressed losses
  - Same stress limits used in earlier specifications ($f_t = 0.0948\sqrt{f_c'}$ and $f_t = 0.19\sqrt{f_c'}$ depending on the environment)
- Resulting in requiring a larger number of strands compared to those required by AASHTO standard specifications

Load Factor for Tension in Prestressed Concrete (Service III) *(continued)*

- Requiring a larger number of strands would indicate that bridges designed earlier would have high tensile stresses and would suggest that they should show signed of cracking.
- This was not supported by field observations.
- To get the same number of strands (on average), the load factor for live load for Service III limit state was reduced to 0.8 to require, on average, the same number of strands..

Prestress Losses in AASHTO LRFD

- Pre-2005, the method used to determine the prestress losses in AASHTO LRFD was identical to the method used by AASHTO standard specifications.
- In 2005, the new prestress loss method was introduced in AASHTO LRFD.
- The new method, **as specified**, resulted in lower losses, mainly due to the introduction of the elastic gain.
- Using the load factor of 0.8 with the new loss method resulted in fewer strands being required by AASHTO standard specifications (the majority of existing bridges) and the pre-2005 AASHTO LRFD.

Prestress Losses in AASHTO LRFD *(continued)*

- **Elastic Gain**
  - Strands change in length when the concrete of the prestressed component change in length.
  - When the concrete experiences compressive strain; i.e., shortening, the prestressing steel gets shorter, resulting in prestress loss and vise versa.
  - Traditionally, the effect of the concrete elongation when subjected to tensile strain, and the associated increase in prestressing force, was ignored.
  - In 2005, the new prestress loss method took this effect into account and called it the “elastic gain.”

Prestress Losses in AASHTO LRFD *(continued)*

- **Elastic Gain *(continued)***
  - To show the significance of including the elastic gain:
    - The elastic shortening at transfer causes prestress loss equal to the compression in the concrete immediately after transfer, multiplied by the initial modular ratio.
    - The elastic gain is equal to the sum of the tensile stresses in concrete at the centroid of the prestressing due to weight of the deck, weight of composite DL and LL multiplied by the final modular ratio.
    - Considering that the design is based on allowing some tension in the concrete under all loads, the elastic shortening loss and the elastic gain are similar in magnitude.

Prestress Losses in AASHTO LRFD *(continued)*

- **Elastic Gain *(continued)***
  - Reducing the total prestress loss by the value of the elastic gain results in higher final stress in the prestressing steel after losses.
  - The higher final stress results in fewer needed strands.
  - At the time the new prestress loss method was developed, the research scope was to determine the losses and did not include investigating the load factor for live load.
  - The new prestress losses method was incorporated in AASHTO LRFD and the 0.8 load factor remained unchanged even though its development was associated with a specific prestress losses method.

Purpose of Tensile Stress Limits (Service III)

- Tensile Stress Limits: what are we trying to prevent?
- Prestressed beams are designed for some tension ($f_t = 0.0948\sqrt{f_c'}$ or $f_t = 0.19\sqrt{f_c'}$) for severe, and not worse than moderate corrosive conditions, respectively.
- Considering that the modulus of rupture is $f_r = 0.24\sqrt{f_c'}$, are we trying to prevent cracking?
- For strength limit state, we design for the heaviest vehicles. What happens to the tension in prestressed concrete when these vehicles use the bridge?

Purpose of Tensile Stress Limits (Service III) *(continued)*

- For a new bridge, the concrete will have no tension cracks and routine live loads may cause tension in the prestressed concrete without causing cracking.
- When a heavy load crosses the bridge, the stress may exceed the modulus of rupture and the concrete may crack.
- After the formation of the crack, every time the bridge is exposed to load effects that overcome the compression in the concrete (i.e., decompression) the crack will open
- Every time the crack opens, contaminants may penetrate the crack and cause strand deterioration.

Purpose of Tensile Stress Limits (Service III) *(continued)*

- Limiting the tensile stresses in prestressed concrete controls the frequency of the crack opening, and therefore controls the deterioration of the strands.
- What are the possible criteria that can be used in the calibration to control the frequency of crack opening?
  - Decompression: i.e., failure when stress is tension
  - Tensile stress limit: i.e., failure when stress exceeds $f_t = 0.0948\sqrt{f_c'}$, $f_t = 0.19\sqrt{f_c'}$ or $f_t = 0.25\sqrt{f_c'}$ (for final calibration $f_t = 0.19\sqrt{f_c'}$ was used)
  - Crack width: i.e., failure when the crack width reaches a prescribed value. Widths of 0.008, 0.012, and 0.016 inches were initially considered, most work used 0.016.

Live Load for Calibration for Tension in Prestressed Concrete (Service III)

- For design, the design is based on the heavier of single or multiple lanes loaded.
- Based on earlier conclusions that the probability of heavy vehicles in multiple lanes is very low, the load used for calibration is single-lane loaded with no multiple presence factor.
- The dynamic load allowance used in the original calibration of the strength limit state (10%) was used.
- One year return period was used to correspond to the one year of WIM data used.
- ADTT of 5,000 was used (only 3 out of 32 sites had ADTT>5,000 and only one of them was > 8,000).

Method of Analysis of Existing Study Bridges for Service III Calibration

- For bridges designed or analyzed using the post-2005 prestressing loss method:
  - For time-dependent losses: the refined estimates of time-dependent losses in *AASHTO LRFD* (2012);
  - The section properties used in the analysis are based on the gross section of the concrete; and,
  - The effects of the “elastic gain” were considered.

Regardless of the method of design used in designing an existing girder, the stresses in the girder used as part of the reliability index calculations were determined by analyzing the girder using the above assumptions.

Method of Analysis of Study Bridges for Service III Calibration *(continued)*

- For bridges designed using the pre-2005 prestressing loss method:
  - For time-dependent losses: the refined estimates of time-dependent losses in *AASHTO LRFD* editions prior to 2005;
  - The section properties used in the analysis are based on the gross section of the concrete; and,
  - The calculations neglect the effects of the "elastic gain."

Target Reliability Index for Service III Calibration

- Due to the lack of clear consequences of failure and the lack of past calibration that can be used as a guide, the reliability indices for existing bridges were determined and used as a guide.
- Due to the difference in methods of determining prestress losses, bridges designed using both methods were analyzed.
- For each girder studied, the design was performed using the applicable specifications and then the reliability index for each of the three limit state functions discussed earlier was determined.

Target Reliability Index for Service III Calibration *(continued)*

- Due to the difference in the load that causes each of the limit state functions to be exceeded, the reliability index varied for different limit state functions.

Limiting Criteria	Live Load required to violate the limiting criterion	Frequency of exceeding the limiting criterion	Reliability Index
Decompression	**Lowest**	**Highest**	**Lowest**
Maximum allowable tensile stress limit	**Middle**	**Middle**	**Middle**
Maximum allowable crack width limit state	**Highest**	**Lowest**	**Highest**

- With the target reliability index dependent on the definition of the limit state function, which one to use?

Target Reliability Index for Service III Calibration *(continued)*

- What limit state function to use?
- **Answer:** The one that provides more uniform reliability across a wide range of bridge geometrical characteristics.

Random Variables for Service III Calibration

Random variables:

- A_s = area of non-prestressing steel, in^2
- A_{ps} = area of prestressing steel in tension zone, in^2
- b = prestressed beam top flange width, in.
- b_0 = deck width transformed to the beam material, in.
- b_1 = prestressed beam bottom flange width, in.
- b_w = web thickness, in.
- c = depth of neutral axis from the extreme compression fiber, in
- C_{fci} = f_{ci} / f'_c
- d_p = distance from extreme compression fiber to centroid of prestressing steel, in.

Random Variables for Service III Calibration *(continued)*

Random variables:

- d_s = distance from extreme compression fiber to centroid of non-prestressing steel, in.
- e_1 = eccentricity of the prestressing force with respect to the centroid of the section at mid-span, in.
- E_{ps} = modulus of elasticity of prestressing steel, psi
- E_s = modulus of elasticity of non-prestressing steel, psi
- f'_c = specified compressive strength of concrete, psi
- f_{pu} = specified tensile strength of prestressing steel, psi
- f_{si} = initial stress in prestressing steel, psi
- f_y = yield strength of non-prestressing steel, psi

Random Variables for Service III Calibration *(continued)*

Random variables:

- h = girder depth, in.
- h_f = deck thickness, in.
- h_{f1} = top flange thickness, in.
- h_{f2} = bottom flange thickness, in.
- l = clear span length of the beam members, ft
- γ_c = unit weight of concrete, pcf
- $\Sigma 0$ = sum of reinforcing element circumferences, in.
- Δf_s = prestress losses, psi

Database of Existing Bridges for Service III Calibration

- A database of existing prestressed concrete girder bridges was extracted from the database of bridges used in the NCHRP 12-78 project.
- Bridges had different geometric characteristics.
- Bridges were assumed to have been designed for limiting tensile stress limit of $f_t = 0.19\sqrt{f_c'}$.
- The database included:
  - 30 I- and bulb-T girder bridges
  - 31 adjacent box girder bridges
  - 36 spread box girder bridges.

Average Reliability Index of Existing Bridges (Service III)

Performance Levels		ADTT			
		ADTT =1,000	ADTT =2,500	ADTT =5,000	ADTT =10,000
Decompression		0.95	0.85	0.74	0.61
Maximum Tensile Stress Limit	$f_t = 0.0948\sqrt{f'_c}$	1.15	1.01	0.94	0.82
	$f_t = 0.19\sqrt{f'_c}$	1.24	1.14	1.05	0.95
	$f_t = 0.25\sqrt{f'_c}$	1.40	1.27	1.19	1.07
Maximum Crack Width	0.008 in	2.29	2.21	1.99	1.85
	0.012 in	2.65	2.60	2.37	2.22
	0.016 in	3.06	2.89	2.69	2.56

Standard Normal Distribution

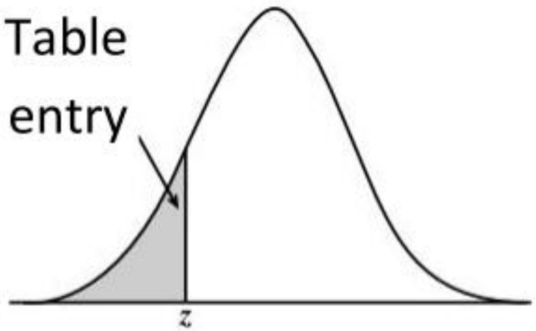


Table entry for Z is the area under the standard normal curve to the left of Z

z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
−1.5	.0668	.0655	.0643	.0630	.0618	.0606	.0594	.0582	.0571	.0559
−1.4	.0808	.0793	.0778	.0764	.0749	.0735	.0721	.0708	.0694	.0681
−1.3	.0968	.0951	.0934	.0918	.0901	.0885	.0869	.0853	.0838	.0823
−1.2	.1151	.1131	.1112	.1093	.1075	.1056	.1038	.1020	.1003	.0985
−1.1	.1357	.1335	.1314	.1292	.1271	.1251	.1230	.1210	.1190	.1170
−1.0	.1587	.1562	.1539	.1515	.1492	.1469	.1446	.1423	.1401	.1379
−0.9	.1841	.1814	.1788	.1762	.1736	.1711	.1685	.1660	.1635	.1611
−0.8	.2119	.2090	.2061	.2033	.2005	.1977	.1949	.1922	.1894	.1867
−0.7	.2420	.2389	.2358	.2327	.2296	.2266	.2236	.2206	.2177	.2148
−0.6	.2743	.2709	.2676	.2643	.2611	.2578	.2546	.2514	.2483	.2451
−0.5	.3085	.3050	.3015	.2981	.2946	.2912	.2877	.2843	.2810	.2776
−0.4	.3446	.3409	.3372	.3336	.3300	.3264	.3228	.3192	.3156	.3121
−0.3	.3821	.3783	.3745	.3707	.3669	.3632	.3594	.3557	.3520	.3483
−0.2	.4207	.4168	.4129	.4090	.4052	.4013	.3974	.3936	.3897	.3859
−0.1	.4602	.4562	.4522	.4483	.4443	.4404	.4364	.4325	.4286	.4247
−0.0	.5000	.4960	.4920	.4880	.4840	.4801	.4761	.4721	.4681	.4641

Database of Simulated Bridges for Service III Calibration

- A database of simulated bridges was developed:
  - Span lengths: 30, 60, 80, 100 and 140 ft.
  - Spacing 6, 8, 10 and 12 ft.
- Analysis cases:
  - Case 1: *AASHTO LRFD*, $f_t = 0.0948\sqrt{f_c'}$, pre-2005 losses
  - Case 2: *AASHTO LRFD*, $f_t = 0.0948\sqrt{f_c'}$, post-2005 losses
  - Case 3: *AASHTO LRFD*, $f_t = 0.19\sqrt{f_c'}$, pre-2005 losses
  - Case 4: *AASHTO LRFD*, $f_t = 0.19\sqrt{f_c'}$, post-2005 losses
- Smallest possible AASHTO section was used for Cases 2 and 4.
- Same section was also used for corresponding Cases 1 and 3.

Reliability Index for Simulated Bridges

- Bridges designed for $f_t = 0.0948\sqrt{f_c'}$ and 5,000 ADTT

Cases	Section Type	Span Length (ft)	Spacing (ft)	Case 1 Designed Using Pre-2005 Loss Method			Case 2 Designed Using Post-2005 Loss Method		
				Decomp.	Max. Tensile	Max. Crack	Decomp.	Max. Tensile	Max. Crack
1	AASHTO I	30	6	1.05	1.49	2.92	1.03	1.51	2.55
2	AASHTO I	30	8	0.90	0.94	2.41	0.93	1.00	2.32
3	AASHTO I	30	10	1.16	1.68	2.87	1.28	1.67	2.82
4	AASHTO I	30	12	1.28	1.67	2.91	0.63	0.97	2.29
Average for 30 ft Span				**1.10**	**1.45**	**2.78**	**0.97**	**1.29**	**2.50**
5	AASHTO II	60	6	0.66	1.01	3.35	0.23	0.61	2.47
6	AASHTO II	60	8	—	—	—	0.73	1.04	2.42
7	AASHTO III	60	10	1.22	1.62	3.01	0.43	0.76	1.97
8	AASHTO III	60	12	1.57	1.96	3.68	0.73	0.99	2.51
Average for 60 ft Span				**1.15**	**1.53**	**3.35**	**0.53**	**0.85**	**2.34**
9	AASHTO III	80	6	1.35	1.66	4.1	0.61	0.92	3.07
10	AASHTO III	80	8	1.8	2.14	5.23	0.82	1.13	3.64
11	AASHTO III	80	10	—	—	—	0.90	1.19	2.93
12	AASHTO IV	80	12	2.2	2.49	5.11	0.83	1.17	3.32
Average for 80 ft Span				**1.78**	**2.10**	**4.81**	**0.79**	**1.10**	**3.24**
13	AASHTO III	100	6	—	—	—	1.45	1.85	3.51
14	AASHTO IV	100	8	1.86	2.00	3.86	1.33	1.43	3.44
15	AASHTO IV	100	10	—	—	—	1.33	1.65	3.37
16	AASHTO V	100	12	1.68	1.99	4.08	0.93	1.24	3.33
Average for 100 ft Span				**1.77**	**2.00**	**3.97**	**1.26**	**1.54**	**3.41**
17	AASHTO IV	120	6	—	—	—	1.32	1.76	3.81
18	AASHTO V	120	8	1.54	2.05	3.65	0.92	1.4	3.14
19	AASHTO V	120	10	—	—	—	0.95	1.46	3.02
20	AASHTO VI	120	12	1.82	2.26	3.88	0.9	1.35	3.38
Average for 120 ft Span				**1.68**	**2.16**	**3.77**	**1.02**	**1.49**	**3.34**
21	AASHTO VI	140	6	1.48	1.99	3.91	0.86	1.36	2.32
22	AASHTO VI	140	8	—	—	—	0.99	1.47	2.79
23	AASHTO VI	140	10	—	—	—	1.05	1.53	3.22
24	—	140	12	—	—	—	—	—	—
Average for 140 ft Span				**1.48**	**1.99**	**3.91**	**0.97**	**1.45**	**2.78**
Average for All Spans				**1.44**	**1.80**	**3.66**	**0.92**	**1.28**	**2.94**

Reliability Index for Simulated Bridges *(continued)*

- Bridges designed for $f_t = 0.0948\sqrt{f'_c}$ and 5,000 ADTT

  Summary:

	Case 1			Case 2		
Cases	Designed Using Pre-2005 Loss Method			Designed Using Post-2005 Loss Method		
	Decomp.	Max. Tensile	Max. Crack	Decomp.	Max. Tensile	Max. Crack
Average for 30 ft. Span	1.10	1.45	2.78	0.97	1.29	2.50
Average for 60 ft. Span	1.15	1.53	3.35	0.53	0.85	2.34
Average for 80 ft. Span	1.78	2.10	4.81	0.79	1.10	3.24
Average for 100 ft. Span	1.77	2.00	3.97	1.26	1.54	3.41
Average for 120 ft. Span	1.68	2.16	3.77	1.02	1.49	3.34
Average for 140 ft. Span	1.48	1.99	3.91	0.97	1.45	2.78
Average for All Spans	1.44	1.80	3.66	0.92	1.28	2.94

Reliability Index for Simulated Bridges *(continued)*

- Bridges designed for $f_t = 0.19\sqrt{f_c'}$ and 5000 ADTT

Cases	Section Type	Span Length (ft)	Spacing (ft)	Case 3 Designed Using Pre-2005 Loss Method			Case 4 Designed Using Post-2005 Loss Method		
				Decomp.	Max. Tensile	Max. Crack	Decomp.	Max. Tensile	Max. Crack
1	AASHTO I	30	6	1.00	1.55	2.39	0.97	1.55	2.46
2	AASHTO I	30	8	0.94	0.92	2.35	0.91	1.00	2.16
3	AASHTO I	30	10	1.29	1.66	2.91	1.18	1.66	2.79
4	AASHTO I	30	12	1.30	1.72	3.02	1.26	1.70	2.91
Average for 30 ft Span				**1.13**	**1.46**	**2.67**	**1.08**	**1.48**	**2.58**
5	AASHTO II	60	6	0.74	1.13	3.11	0.18	0.58	2.41
6	AASHTO II	60	8	1.04	1.39	2.82	0.28	0.66	1.91
7	AASHTO III	60	10	0.42	0.79	2.05	0.42	0.78	2.07
8	AASHTO III	60	12	0.66	1.00	2.5	0.68	0.96	2.53
Average for 60 ft Span				**0.72**	**1.08**	**2.62**	**0.39**	**0.75**	**2.23**
9	AASHTO III	80	6	0.56	0.97	3.13	0.13	0.51	2.53
10	AASHTO III	80	8	1.06	1.46	3.43	0.42	0.78	3.2
11	AASHTO III	80	10	1.58	1.84	3.65	0.37	0.65	2.72
12	AASHTO IV	80	12	0.83	1.15	3.72	0.51	0.87	3.11
Average for 80 ft Span				**1.01**	**1.36**	**3.48**	**0.36**	**0.70**	**2.89**
13	AASHTO III	100	6	—	—	—	0.82	1.23	3.44
14	AASHTO IV	100	8	1.31	1.42	3.60	0.69	0.76	2.76
15	AASHTO IV	100	10	1.80	1.98	3.67	0.75	1.04	3.12
16	AASHTO V	100	12	1.08	1.37	3.43	0.40	0.72	2.55
Average for 100 ft Span				**1.40**	**1.59**	**3.57**	**0.67**	**0.94**	**2.97**
17	AASHTO IV	120	6	1.53	1.98	3.71	0.70	1.28	3.10
18	AASHTO V	120	8	0.90	1.30	3.31	0.46	0.85	2.55
19	AASHTO V	120	10	1.25	1.65	3.35	0.26	0.78	2.68
20	AASHTO VI	120	12	1.19	1.66	3.37	0.47	0.91	2.69
Average for 120 ft Span				**1.22**	**1.65**	**3.44**	**0.47**	**0.96**	**2.76**
21	AASHTO VI	140	6	0.84	1.41	3.23	0.28	0.82	2.41
22	AASHTO VI	140	8	1.22	1.68	3.30	0.53	0.98	3.04
23	AASHTO VI	140	10	—	—	—	0.62	1.08	2.46
24	—	140	12	—	—	—	—	—	—
Average for 140 ft Span				1.03	1.55	3.27	0.48	0.96	2.64
Average for All Spans				**1.07**	**1.43**	**3.15**	**0.58**	**0.96**	**2.68**

Reliability Index for Simulated Bridges *(continued)*

- Bridges designed for $f_t = 0.19\sqrt{f'_c}$ and 5,000 ADTT

Summary:

	Case 1			Case 2		
Cases	Designed Using Pre-2005 Loss Method			Designed Using Post-2005 Loss Method		
	Decomp.	Max. Tensile	Max. Crack	Decomp.	Max. Tensile	Max. Crack
Average for 30 ft Span	1.13	1.46	2.67	1.08	1.48	2.58
Average for 60 ft Span	0.72	1.08	2.62	0.39	0.75	2.23
Average for 80 ft Span	1.01	1.36	3.48	0.36	0.70	2.89
Average for 100 ft Span	1.40	1.59	3.57	0.67	0.94	2.97
Average for 120 ft Span	1.22	1.65	3.44	0.47	0.96	2.76
Average for 140 ft Span	1.03	1.55	3.27	0.48	0.96	2.64
Average for All Spans	1.07	1.43	3.15	0.58	0.96	2.68

Selection of the Target Reliability Index

Performance Level	Reliability Index				
	Average β for Existing Bridges in the NCHRP 12-78	Average β for Simulated bridges designed for $f_t = 0.0948\sqrt{f'_c}$ and pre-2005 loss method	Average β for Simulated bridges designed for $f_t = 0.19\sqrt{f'_c}$ and pre-2005 loss method	Proposed Target β for bridges in severe environment	Proposed Target β for bridges in normal environment
Decompression	0.74	1.44	1.07	1.20	1.00
Maximum Allowable Tensile Stress of $f_t = 0.19\sqrt{f'_c}$	1.05	1.80	1.43	1.50	1.25
Maximum Allowable Crack Width of 0.016 in.	2.69	3.68	3.15	3.30	3.10

Application of Calibration Procedure to Service III

- **Step 1:** Formulate the Limit State Function and Identify Basic Variables: Three limit state functions were identified as shown above. Expressions for resistance predictions were developed.
- **Step 2:** Identify and Select Representative Structural Types and Design Cases
- **Step 3:** Determine Load and Resistance Parameters for the Selected Design Cases: Statistical parameters for variations in dimensions and material properties were determined as discussed above.

Application of Calibration Procedure To Service III *(continued)*

- **Step 4:** Develop Statistical Models for Load and Resistance: Probability distribution and statistical parameters for live load presented and for other variables affecting the resistance were developed.

Application of Calibration Procedure To Service III *(continued)*

- **Step 5:** Develop the Reliability Analysis Procedure: A large number of random cases that are used in defining the mean and standard deviation of the resistance were developed using Monte Carlo simulation. The statistical information of all the required variables was used to determine the statistical parameters of the resistance.

For each girder, 1,000 values for each variable were determined using Monte Carlo simulation. 1,000 values for the dead load and resistance were determined each using one set of values of each random variable resulting. The mean and standard deviation of the dead load and the resistance were then calculated based on the 1,000 simulations.

Application of Calibration Procedure To Service III *(continued)*

- **Step 6:** Calculate the Reliability Indices for Current Design Code and Current Practice

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

β = reliability Index

μ_R = mean value of the resistance

μ_Q = mean value of the applied loads

σ_R = standard deviation of the resistance

σ_Q = standard deviation of the applied loads

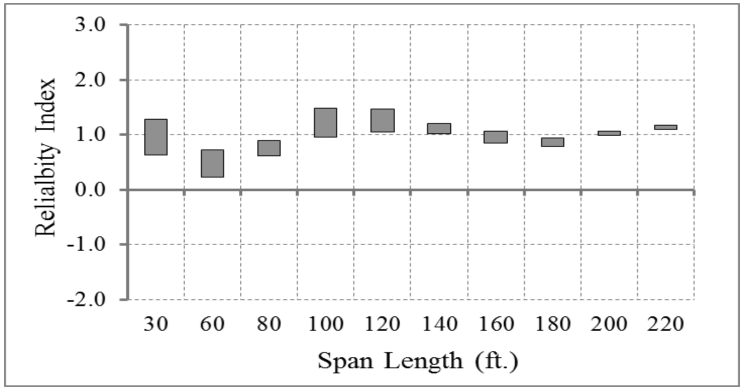
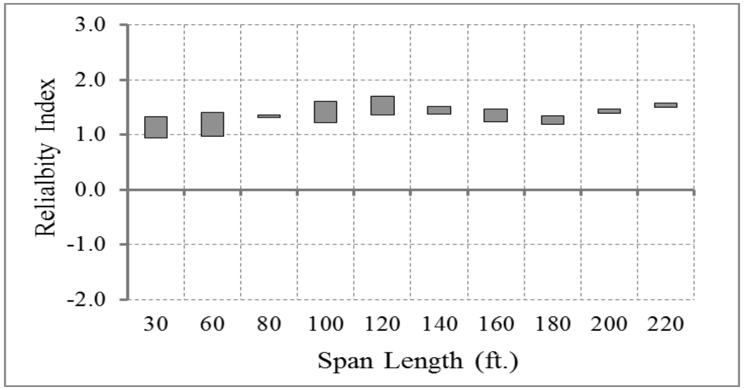
Application of Calibration Procedure To Service III *(continued)*

- **Step 7:** Review the Results and Select the Target Reliability Index β_T : This was performed as discussed above.
- **Step 8:** Select Potential Load and Resistance Factors for Service III: The Service III limit state resistance is affected by the tensile stress limit used in the design. Therefore, in addition to trying different load factors, different stress limits for the design were also investigated. Maximum concrete design tensile stress of $f_t = 0.0948\sqrt{f'_c}$, $f_t = 0.19\sqrt{f'_c}$ and $f_t = 0.25\sqrt{f'_c}$ were considered. In addition, the simulated bridge database used in determining the target resistance factor was further expanded to allow longer spans.

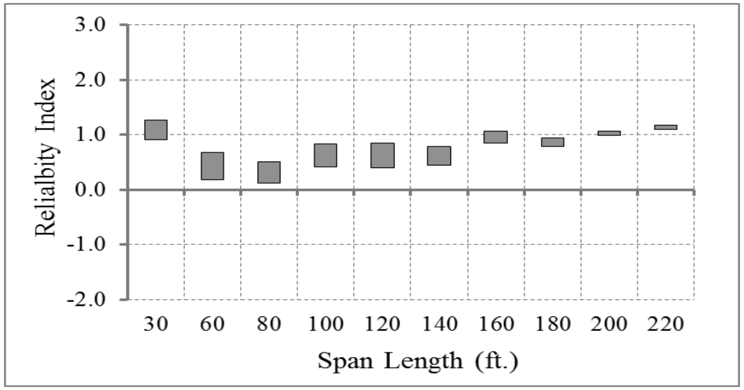
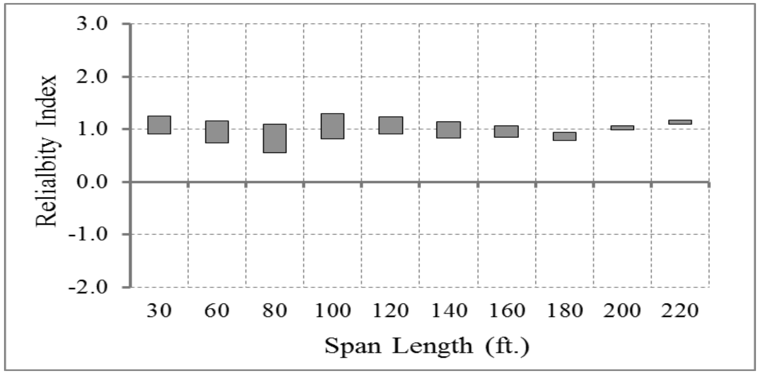
Application of Calibration Procedure To Service III *(continued)*

- **Step 8** *(cont'd):* Results for bridges designed for $f_t = 0.25\sqrt{f'_c}$ were less uniform and had a reliability level less than the target reliability index. Results for this case are not shown below.
- **Step 9:** Calculate Reliability Indices Using the Selected Load and Resistance Factors and Compare to Target Reliability Index

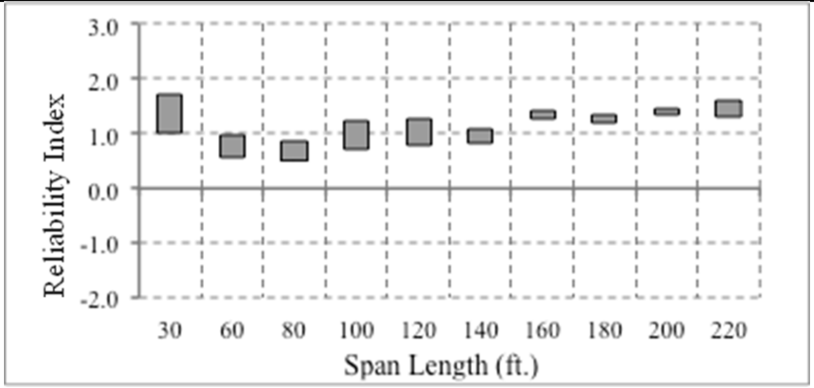
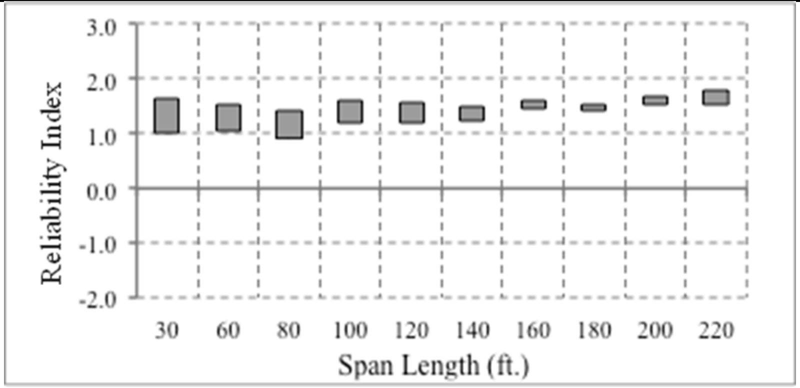
Application of Calibration Procedure To Service III *(continued)*

	
Decompression limit state Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=0.8, ($f_t = 0.0948\sqrt{f_c'}$) One-year return period	Decompression limit state Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=1.0, ($f_t = 0.0948\sqrt{f_c'}$) One-year return period

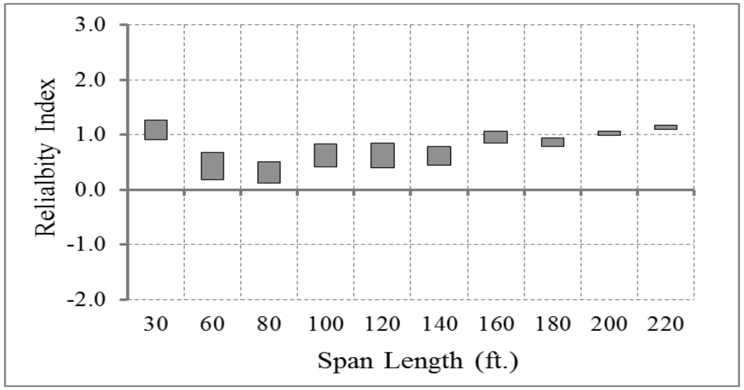
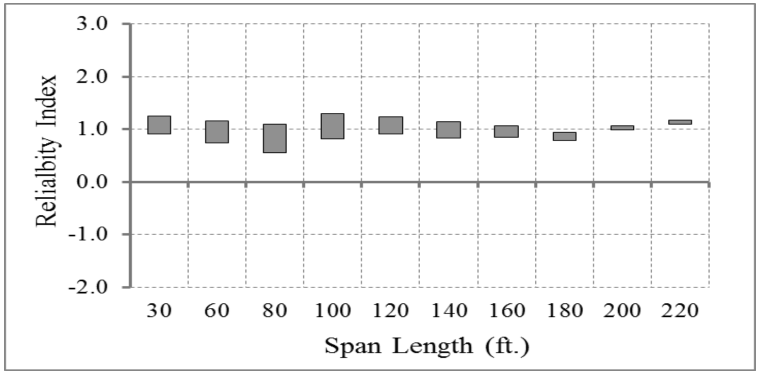
Application of Calibration Procedure To Service III *(continued)*

	
Decompression limit state Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=0.8, ($f_t = 0.19\sqrt{f'_c}$) One-year return period	Decompression limit state Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=1.0, ($f_t = 0.19\sqrt{f'_c}$) One-year return period

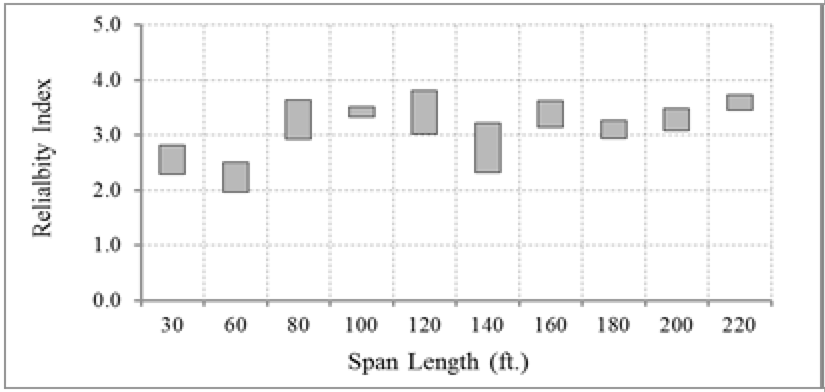
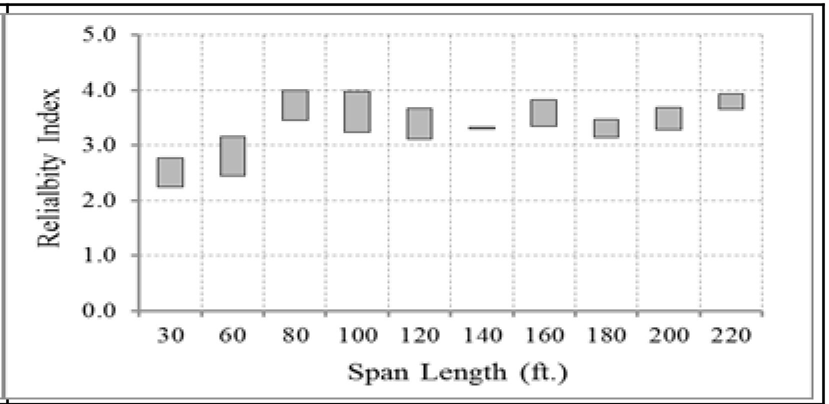
Application of Calibration Procedure To Service III *(continued)*

	
Max Tensile Stress Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=0.8, ($f_t = 0.0948\sqrt{f'_c}$) One-year return period	Max Tensile Stress Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=1.0, ($f_t = 0.0948\sqrt{f'_c}$) One-year return period

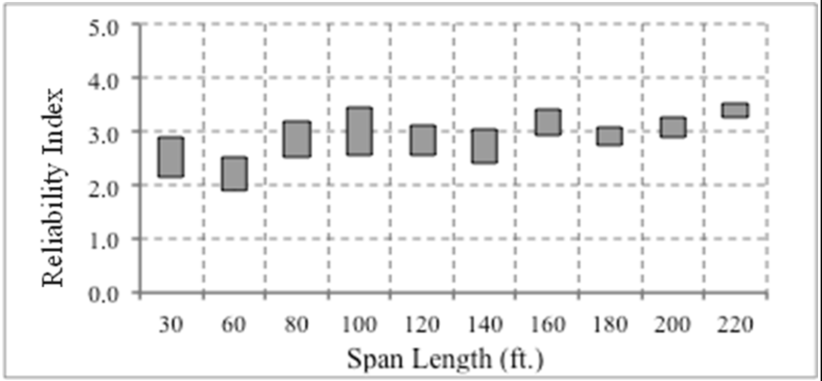
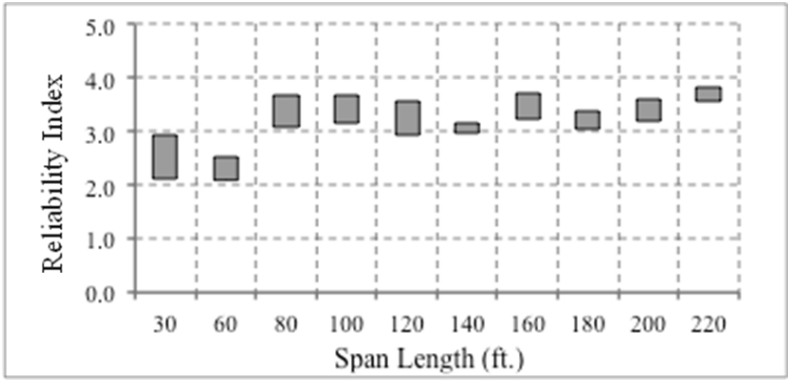
Application of Calibration Procedure To Service III *(continued)*

	
Max Tensile Stress Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=0.8, ($f_t = 0.19\sqrt{f'_c}$) One-year return period	Max Tensile Stress Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=1.0, ($f_t = 0.19\sqrt{f'_c}$) One-year return period

Application of Calibration Procedure To Service III *(continued)*

	
Max Crack Width Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5000, γ_{LL}=0.8, ($f_t = 0.0948\sqrt{f'_c}$) One year return period	Max Crack Width Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5000, γ_{LL}=1.0, ($f_t = 0.0948\sqrt{f'_c}$) One year return period

Application of Calibration Procedure To Service III *(continued)*

	
Max Crack Width Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=0.8, ($f_t = 0.19\sqrt{f'_c}$) One-year return period	Max Crack Width Limit State Reliability index of simulated I-girder bridges Refined time-dependant Losses and elastic gain considered ADTT=5,000, γ_{LL}=1.0, ($f_t = 0.19\sqrt{f'_c}$) One-year return period

Application of Calibration Procedure To Service III *(continued)*

Summary of Reliability Indices for Simulated Bridges

Bridges Designed for $f_t = 0.0948\sqrt{f_c'}$

ADTT	Live Load Factor=0.8			Live Load Factor=1.0		
	De-compression	Max Tensile Stress Limit	Max Crack Width (0.016 in.)	De-compression	Max Tensile Stress Limit	Max Crack Width (0.016 in.)
1,000	1.05	1.41	3.16	1.42	1.79	3.36
2,500	1.01	1.35	3.11	1.38	1.75	3.33
5,000	0.97 (Target 1.20)	1.31 (Target 1.50)	3.06 (Target 3.30)	1.33 (Target 1.20)	1.70 (Target 1.50)	3.32 (Target 3.30)
10,000	0.94	1.30	3.00	1.32	1.66	3.28

Application of Calibration Procedure To Service III *(continued)*

Summary of Reliability Indices for Simulated Bridges

Bridges Designed for $f_t = 0.19\sqrt{f'_c}$

ADTT	Live Load Factor=0.8			Live Load Factor=1.0		
	De-compression	Max Tensile Stress Limit	Max Crack Width (0.016 in.)	De-compression	Max Tensile Stress Limit	Max Crack Width (0.016 in.)
1,000	0.84	1.27	2.92	1.11	1.53	3.25
2,500	0.70	1.15	2.87	1.04	1.46	3.17
5,000	0.68 (Target 1.00)	1.10 Target (1.25)	2.82 (Target 3.1)	1.00 (Target 1.00)	1.41 (Target 1.25)	3.14 (Target 3.1)
10,000	0.64	1.07	2.78	0.98	1.34	3.11

Effect of the Higher Live Load Factor On the Design

Cases	Section Type	Span Length (ft)	Girder Spacing (ft)	$f_t = 0.0948\sqrt{f'_c}$, γLL=0.8, Pre-2005 losses	$f_t = 0.0948\sqrt{f'_c}$, γLL=0.8, Post-2005 losses	$f_t = 0.0948\sqrt{f'_c}$, γLL=1.0, Post-2005 losses	$f_t = 0.19\sqrt{f'_c}$, γLL=0.8, Pre-2005 losses	$f_t = 0.19\sqrt{f'_c}$, γLL=0.8, Post-2005 losses	$f_t = 0.19\sqrt{f'_c}$, γLL=1.0, Post-2005 losses
1	AASHTO I	30	6	8	8	8	8	8	8
2	AASHTO I	30	8	10	10	10	10	10	10
3	AASHTO I	30	10	12	12	12	12	12	12
4	AASHTO I	30	12	14	14	14	14	14	14
5	AASHTO II	60	6	20	16	20	18	16	16
6	AASHTO II	60	8	-	22	26	24	20	22
7	AASHTO III	60	10	22	20	22	20	20	20
8	AASHTO III	60	12	28	24	28	24	24	24
9	AASHTO III	80	6	28	24	28	24	22	24
10	AASHTO III	80	8	38	30	34	32	28	30
11	AASHTO III	80	10	-	36	40	42	32	38
12	AASHTO IV	80	12	40	34	38	34	32	34
13	AASHTO III	100	6	-	40	46	-	38	42
14	AASHTO IV	100	8	50	42	46	44	38	42
15	AASHTO IV	100	10	-	48	54	56	44	50
16	AASHTO V	100	12	56	46	50	48	42	46
17	AASHTO IV	120	6	-	52	58	58	48	52
18	AASHTO V	120	8	62	52	58	54	48	52
19	AASHTO V	120	10	-	60	68	68	54	60
20	AASHTO VI	120	12	74	58	64	64	54	58
21	AASHTO VI	140	6	62	54	58	54	48	52
22	AASHTO VI	140	8	-	64	70	68	58	64
23	AASHTO VI	140	10	-	74	-	-	68	74
24		140	12	-	-	-	-	-	-

AASHTO Revisions

- In 2015, AASHTO approved revisions to Section 3 that appeared in the 2016 interims of the specifications.

Item #1

In Table 3.4.1-1, replace the load factor for live load in the Service III Load Combinations with γ_{LL}.

Item #2

In Article 3.4.1, add new Table 3.4.1-4 as follows:

Table 3.4.1-4—Load Factors for Live Load for Service III Load Combination, γ_{LL}

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

Key Points

- The tensile stress limit in prestressed components determines the probability of cracks forming in prestressed components under severe loading and determines the frequency of these cracks opening under live load.
- The live load factor originally in AASHTO LRFD was not statistically calibrated and was developed in conjunction of specific method for prestressed losses.
- The method of determining the prestress losses changed in 2005.

Key Points *(continued)*

- The limit state function (failure criteria) for Service III calibration can be defined several different ways. De-compression, specific stress limit, and specific crack width were investigated.
- For the same limit state function, the reliability index for Service III is a function of the stress limits used in the design.
- To maintain the average reliability of the current system and the uniformity of reliability index, the live load factor needed to be increased to 1.0.




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Introduction and Course Objectives

Lecture 5: Calibration of Service Limit States in AASHTO LRFD (2)
Other Structural Limit State

Wagdy Wassef, PhD, PE
AECOM

Eastern Federal Lands
June 27-28, 2017

U.S. Department of Transportation
Federal Highway Administration

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO

Learning Outcomes

- **Understand** the background of the calibration of:
  - Cracking of reinforced concrete (Service I)
  - Live load deflection (Service I)
  - Yielding of steel (Service II)
  - Fatigue (Fatigue limit state)
- **Review** the revisions to AASHTO LRFD due to the calibration of the limit states listed above

Calibration of Cracking
of
Reinforced Concrete Components
(Service I)

Cracking of Reinforced Concrete Components (Service I)

- Typically, reinforced concrete components are designed for the strength limit state requirements and checked for Service I load combination.
- The purpose of the Service I check is to ensure that the crack width remains within tolerable limits to control reinforcement corrosion.
- The specifications provisions are written in a form emphasizing reinforcement details, i.e., limiting bar spacing rather than crack width.
- Limiting bar spacing may require using smaller bars or more reinforcement.

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Two exposure conditions exist in the specifications:
  - **Class 1**: Used where reduced concerns of appearance and/or corrosion exist. Class 1 corresponds to an assumed crack width of 0.017 in.
  - **Class 2**: Used where increased concerns of appearance and/or corrosion exist. Class 2 corresponds to an assumed crack width of 0.01275 in.
- Previous research indicated that there appears to be **little or no correlation between crack width and corrosion**.
- The different classes of exposure conditions have been so defined in the design specifications in order to **provide flexibility in the application** of these provisions to meet the needs of the bridge owner.

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Load factors for Service I load combination:
  - DL Load Factor = 1.0
  - LL Load factor 1.0
- Calibration was performed for decks designed using the conventional design method only for the following reasons:
  - Typically designers use the smallest possible thickness and determine the reinforcement using #5 bars and only switch to #6 bars when the spacing of #5 bars becomes too small.
  - This allows the creation of database of decks where each deck will have the same design regardless of the designer and, thus, the same reliability.
  - Other types of deck do not allow this (empirical or P/S decks).

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Live load model for calibration:
  - The heavy axle of the design truck
  - ADTT 1,000, 2,500, 5,000 and 10,000 considered. ADTT of 5,000 was used for the calibration.
  - Axle load statistical parameters were determined for different time periods (1 day to 100 years).

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Variables included in the calibration:

A_s = area of steel rebar, in^2

b = the equivalent strip width of concrete deck, in.

C_{Ec} = constant parameter for concrete elasticity modulus.

d = effective depth of concrete section, in.

d_c = bottom cover measured from center of lowest bar, in

E_s = modulus of elasticity of steel reinforcement, psi

f_c' = specified compressive strength of concrete, psi

f_y = yield strength of steel reinforcement, psi

h = the thickness of the deck, in.

γ_c = unit weight of concrete, pcf

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Database of reinforced concrete decks

Deck Group #	Girder Spacing (ft.)	Deck Thickness (in.)
1	6	7.0
		7.5
		8.0
2	8	7.5
		8.0
		8.5
3	10	8.0
		8.5
		9.0
		9.5
4	12	8.0
		8.5
		9.0
		9.5
		10.0

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Target reliability index
  - Monte Carlo simulation was used to determine the statistical parameters for the resistance

ADTT	Positive Moment Region		Negative Moment Region	
	Reliability Index (Class 1)	Reliability Index (Class 2)	Reliability Index (Class 1)	Reliability Index (Class 2)
1000	2.44	1.54	2.37	1.77
2500	1.95	1.07	1.79	1.27
5000	1.66	0.85	1.61	1.05
10000	1.39	0.33	1.02	0.50
Avg.	1.86	0.95	1.70	1.15
Max.	2.44	1.54	2.37	1.77
Min.	1.39	0.33	1.02	0.50
Std Dev.	0.45	0.50	0.56	0.53
COV	24%	53%	33%	46%

Cracking of Reinforced Concrete Components (Service I) *(continued)*

- Target reliability index (continued)
  - Class 2 exposure required more reinforcement, yet, as a result of the more stringent requirements, the reliability index was lower for Class 2 exposure.
  - Positive moment reinforcement is typically controlled by Strength I requirements, i.e., more reinforcement than required by Service I is provided. This results in positive moment region reliability higher than shown above when the reinforcement is determined based on Strength I.
  - For ADTT = 5,000, the selected reliability indices are 1.6 and 1.0 for Class 1 and Class 2 exposure, respectively.

Application of Calibration Procedure to Cracking of RC Decks (Service I)

- **Step 1:** Formulate the Limit State Function and Identify Basic Variables: The limit state function considered is the limit on the estimated crack width. In the absence of information suggesting that the current provisions based on a crack width of 0.017 in. and 0.01275 in. for Class 1 and Class 2, respectively, are not adequate, the current crack widths were maintained as the limiting criteria.
- **Step 2**: Identify and Select Representative Structural Types and Design Cases
- **Step 3**: Determine Load and Resistance Parameters for the Selected Design Cases: Statistical parameters for variations in dimensions and material properties were determined as discussed above.

Application of Calibration Procedure to Cracking of RC Decks (Service I) *(continued)*

- **Step 4**: Develop Statistical Models for Load and Resistance: Probability distribution and statistical parameters for live load (axle loads) and for other variables affecting the resistance were developed.
- **Step 5**: Develop the Reliability Analysis Procedure: A large number of random cases that are used in defining the mean and standard deviation of the resistance were developed using Monte Carlo simulation (1,000 values for the load and 1,000 for the resistance for each simulation). The statistical information of all the required variables was used to determine the statistical parameters of the resistance.

Application of Calibration Procedure to Cracking of RC Decks (Service I) *(continued)*

- **Step 6**: Calculate the Reliability Indices for Current Design Code and Current Practice

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

β = reliability Index

μ_R = mean value of the resistance

μ_Q = mean value of the applied loads

σ_R = standard deviation of the resistance

σ_Q = standard deviation of the applied loads

Application of Calibration Procedure to Cracking of RC Decks (Service I) *(continued)*

- **Step 7**: Review the Results and Select the Target Reliability Index β_T : This was performed as discussed above.
- **Step 8**: Select Potential Load and Resistance Factors for Service III: The reliability indices for different cases are shown below. The results were uniform. This indicated that no need for changes to the load factor.
- **Step 9**: Calculate Reliability Indices Using the Selected Load and Resistance Factors and Compare to Target Reliability Index (not needed).

Application of Calibration Procedure to Cracking of RC Decks (Service I) *(continued)*

<table>
<tr><td>

Positive Moment</td><td>

Negative Moment</td></tr>
<tr><td></td><td>Class 1 Exposure
ADTT 5,000
One-year return
Reliability Index 1.6</td></tr>
</table>

Application of Calibration Procedure to Cracking of RC Decks (Service I) *(continued)*

	
Positive Moment	Negative Moment
	Class 2 Exposure ADTT 5,000 One-year return Reliability Index 1.0

Calibration of Live-Load Deflections (Service I)

Calibration of Live-Load Deflections

- History of live-load deflection requirements
  - 1871: Phoenix Bridge Company, L/1200 for a train moving 30 miles per hour
  - Early 1900s: The American Railway Engineering and Maintenance of Way Association (AREMA) adopted span-to-depth ratios, based on engineering judgement

    Pony trusses and plate girders: L/10 (currently L/10 for trusses and L/12 for rolled shapes and plate girders)

Calibration of Live-Load Deflections *(continued)*

- History of LL deflection requirements
  - Early 1900s *(cont'd):* AREMA committee could not reach an acceptable guidance on how to achieve economy and limit vibrations particularly when higher strength materials are used.
  - The committee report states:

    "We established the rule because we could not agree on any. Some of us in designing a girder that is very shallow in proportion to its length decrease the unit stress or increase the section according to some rule which we guess at. We put that in there so that a man would have a warrant for using whatever he pleased."

Calibration of Live-Load Deflections *(continued)*

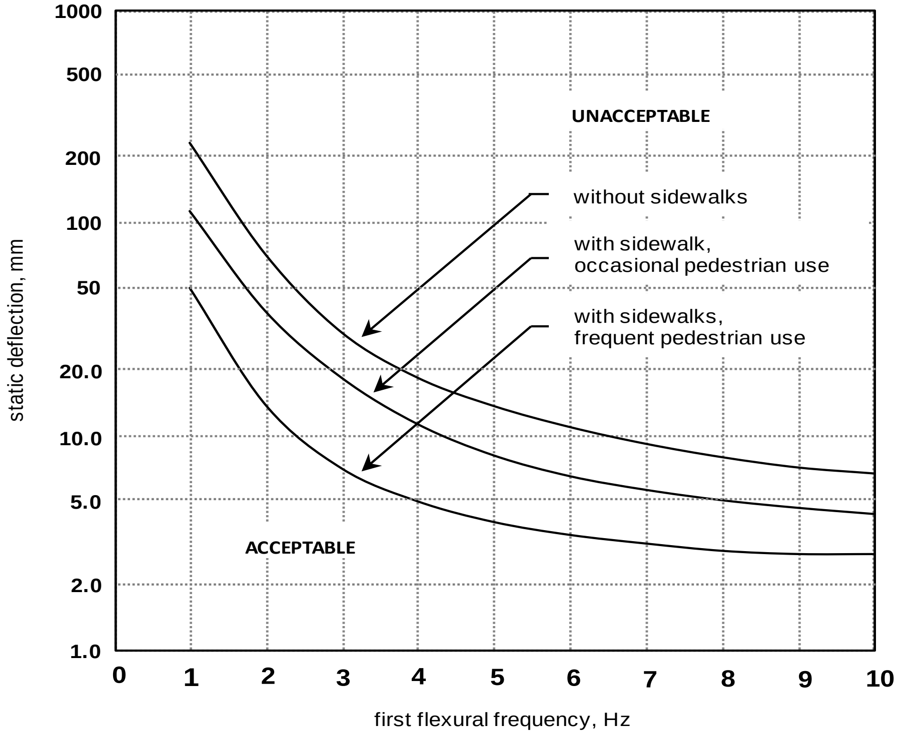
- History of LL deflection requirements *(cont'd):*
  - 1913: span-to-depth ratios for highway bridges, adopted by AASHTO in 1924
  - 1930: Bureau of Public Roads, L/800 and L/1000 without and with pedestrians, respectively, and L/300 for cantilevers. Meant to limit vibrations.

Calibration of Live-Load Deflections *(continued)*

- History of LL deflection requirements *(cont'd):*
  - 1958: The ASCE Committee on Deflection Limitations of Bridges. State DOTs survey concluded:
    - Passage of medium weight vehicles, not heavy vehicles, caused maximum oscillations
    - More often, objectionable vibrations came from continuous span bridges than simple span bridges
    - There is no defined level of vibration which constitutes being undesirable

Calibration of Live-Load Deflections *(continued)*

- History of LL deflection requirements *(cont'd):*
  - Canadian Highway Bridge design Code (CHBDC) includes a deflection check based on limiting the accelerations associated with vibrations.



Current US Practices for Live-Load Deflections Limits

- Span-to-depth ratio changes in AASHTO

Year	Trusses	Plate Girders	Rolled Shapes
1913, 1924	1/10	1/12	1/20
1931	1/10	1/15	1/20
1935, 1941, 1949, 1953	1/10	1/25	1/25
2012	1/10	1/25	1/25

- L/XX limits (2007)

Bridges without pedestrian access	Bridges with pedestrian access
L/1600 (1 state) L/1100 (1 state) L/1000 (5 states) L/800 (40 states)	L/1600 (1 state) L/1200 (2 states) L/1100 (1 state) L/1000 (39 states) L/800 (3 states)

Current US Practices for Live-Load Deflections Limits *(continued)*

- Live loads used in deflection checks when using LFD
  - HS20 truck only (1 state)
  - HS20 truck plus impact (16 states)
  - HS20 lane load plus impact (1 state)
  - HS20 truck plus lane load without impact (1 state)
  - Larger of HS20 truck plus impact or HS20 lane load plus impact (7 states)
  - HS20 truck plus lane plus impact (17 states)
  - Military or permit vehicles (4 states)
  - HS25 truck (8 states)

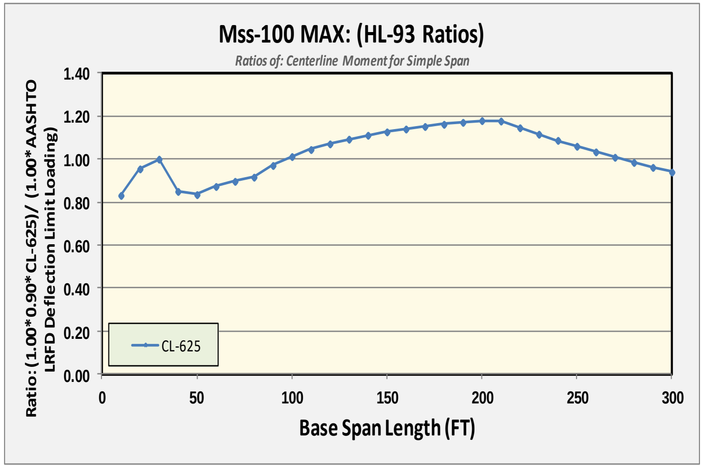
Can the Canadian Approach Be Used For U.S. Bridges?

- Humans are more sensitive to acceleration than displacement per se, especially when stationary on a bridge.
- A direct comparison needs to consider design live load, dynamic load allowance, load factors, and analysis assumptions.

Can the Canadian Approach Be Used For U.S. Bridges?

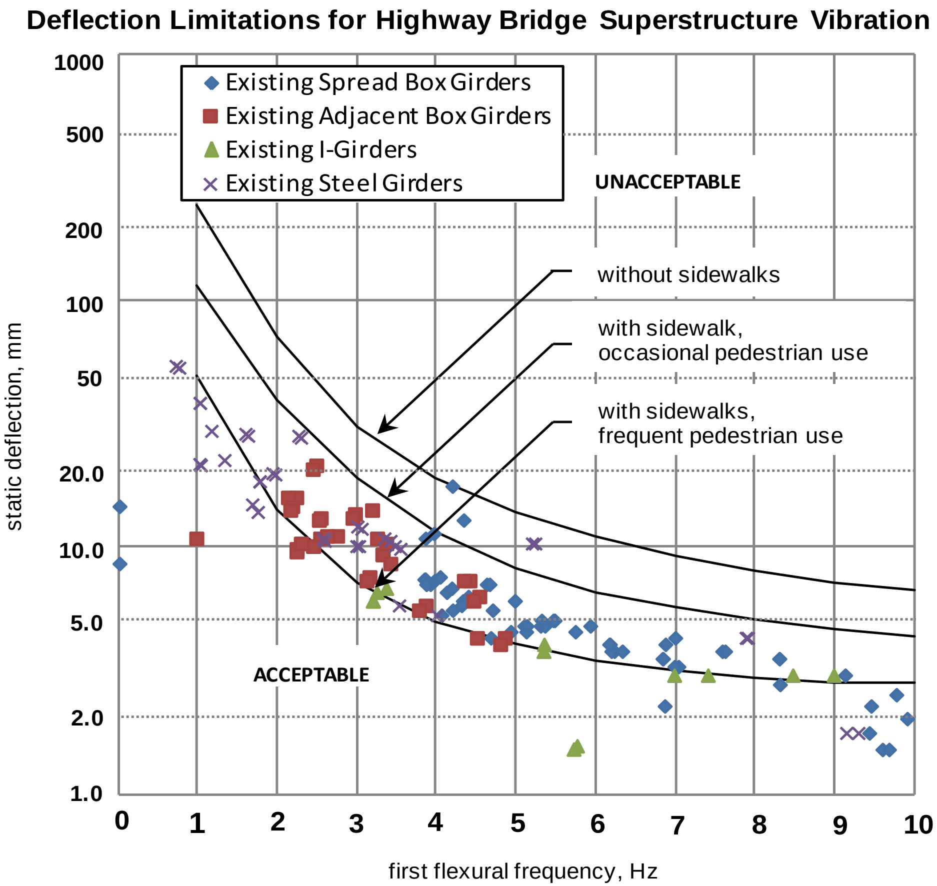
- AASHTO and CHBDC use different loads for deflection calculations. To get a feel of the difference, the deflections from one lane of the deflection design load is shown.

- Ratio of deflections is 0.8 to 1.2.
- It was concluded that if the bridges designed to AASHTO criteria also seem to satisfy the CHBDC, the L/ criteria in AASHTO will be considered deemed-to-satisfy.



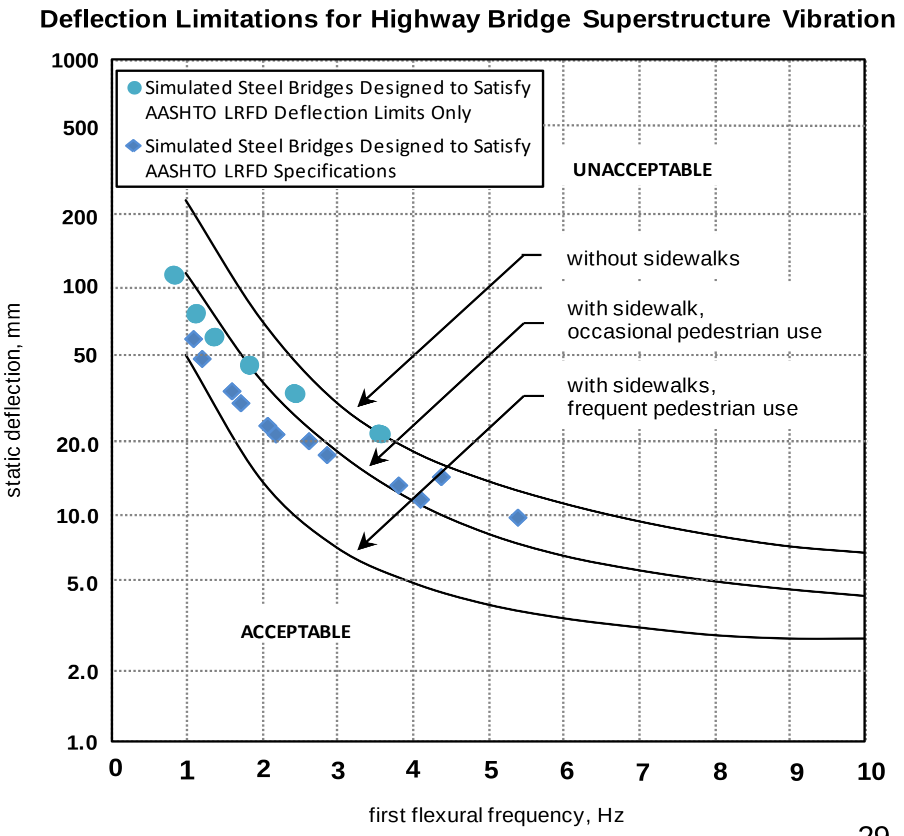
Database of Existing Bridges

- 41 bridges of different types
- For the most part, deflections satisfy the CHBDC requirements for bridges with no sidewalks and with occasional pedestrian use.



Database of Simulated Bridges

- Steel I-Girders
- Spans: 60, 90, 120, 160, 200, and 300 ft.
- Girder spacing: 9 and 12 ft.
- Designed for deflection only
- Redesigned for all requirements



Assessment of the Results of Deflection Calibration

- **Theoretical Conclusions**: LRFD specifications may be revised to satisfy frequency, perception, and deflection by adopting the CHBDC provisions.
- **Practical assessment of the results**: Variations in the application of current requirements by different DOTs produce more differences in the results than would revising the design load.
- **Conclusion**: No compelling reason to change current requirements. Current provisions may be considered “deemed-to-satisfy”. However, it was suggested that the fatigue truck may be used for deflection analysis as it better represents actual trucks.

Calibration of Yielding
of
Steel Components under Service Load
(Service II)

Background of Service II

- The limit state is intended to prevent changes in riding quality and appearance resulting from permanent deflections in service
- Achieved by limiting stresses to 95% of yield in a composite girder or 80% of yield in a non-composite girder under an overload and to design slip critical connections for the same overload requirements
- In AASHTO standard specifications' LFD design, the overload was dead load plus 5/3 of the HS20 loading
- In AASHTO LRFD, Service II is used to investigate these requirements with 1.30 load factor on live load

Background of Service II (Overloads)

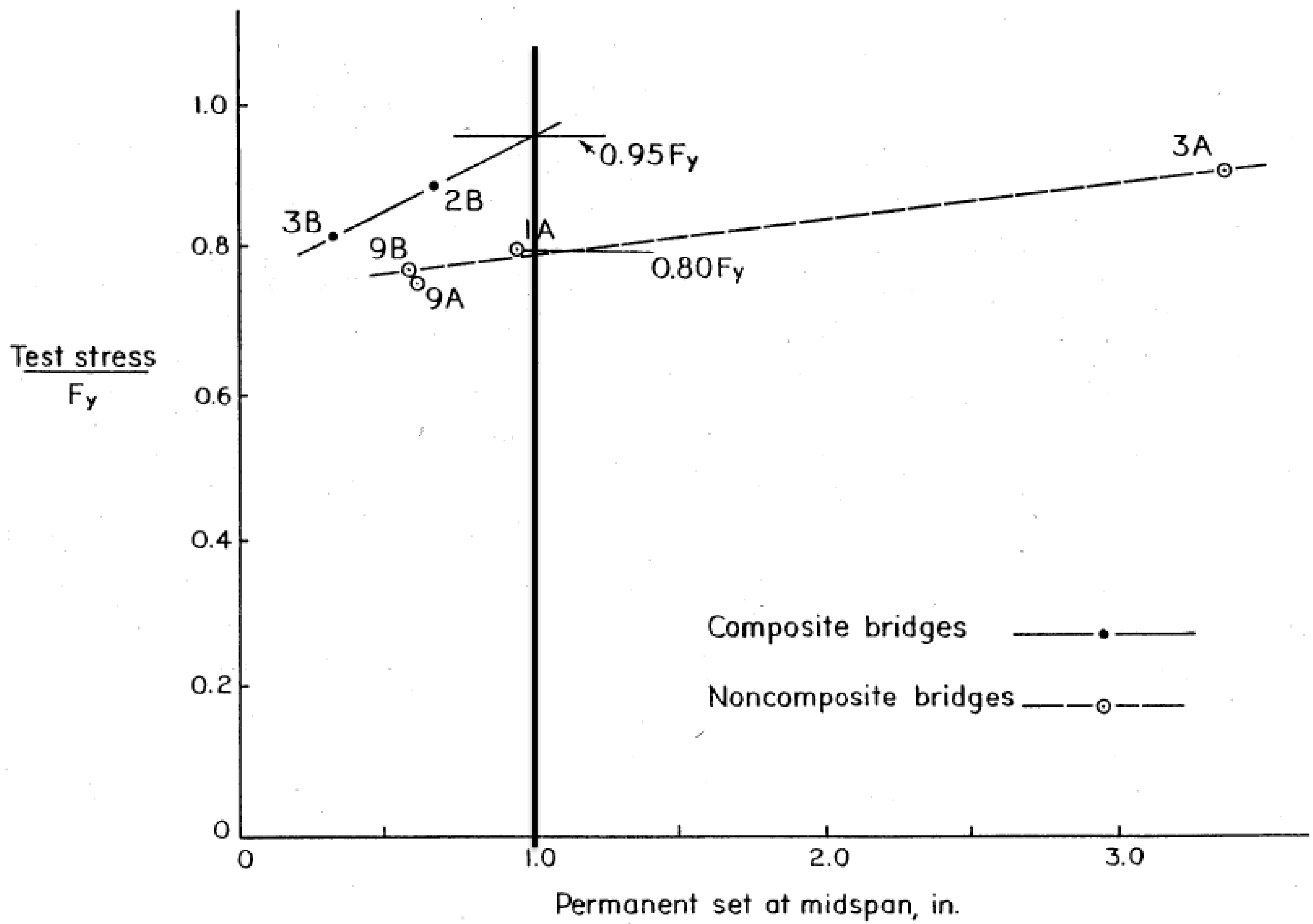
- The issue was originally investigated in the AASHO (now AASHTO) Road Test in the late 1950s and early 1960s. Structures were subjected to repeated relatively high stresses.

Summary of Initial Stresses in Steel Bridges						No. of Vehicle Passages	
Design Stress (ksi)			Actual Stress (ksi)				
Bridge	Center Beam	Exterior Beam	Interior Beam	Center Beam	Exterior Beam	To First Cracking	Total
(a) Non-composite Bridges							
1A	27.0	—	25.3	27.7	30.1	536,000	557,400
1B	34.8	—	32.5	35.4	40.5	—	235
2A	35.0	—	35.0	39.4	41.1		26
3A	27.3	—	28.6	30.9	35.4	—	392,400
4A	34.7	—	35.9	38.9	41.1	—	106
4B	34.7	—	39.1	42.1	42.3	—	106
9A	—	27.0	22.9	24.7	25.5	477,900	477,900
9B	—	27.0	24.0	24.6	26.0	477,900	477,900
(b) Composite Bridges							
2B	35.0	—	30.2	33.8	35.8	531,500	558,400
3B	26.9	—	26.0	28.8	31.0	535,500	557,800

Background of Service II *(continued)*

- The stress limits (0.95 fy for composite and 0.8 fy for non-composite girders) correspond to 1 inch permanent set at midspan of approximately 50 ft. spans. Only two data points existed for composite girders and four data points for non-composite girders.

Background of Service II *(continued)*



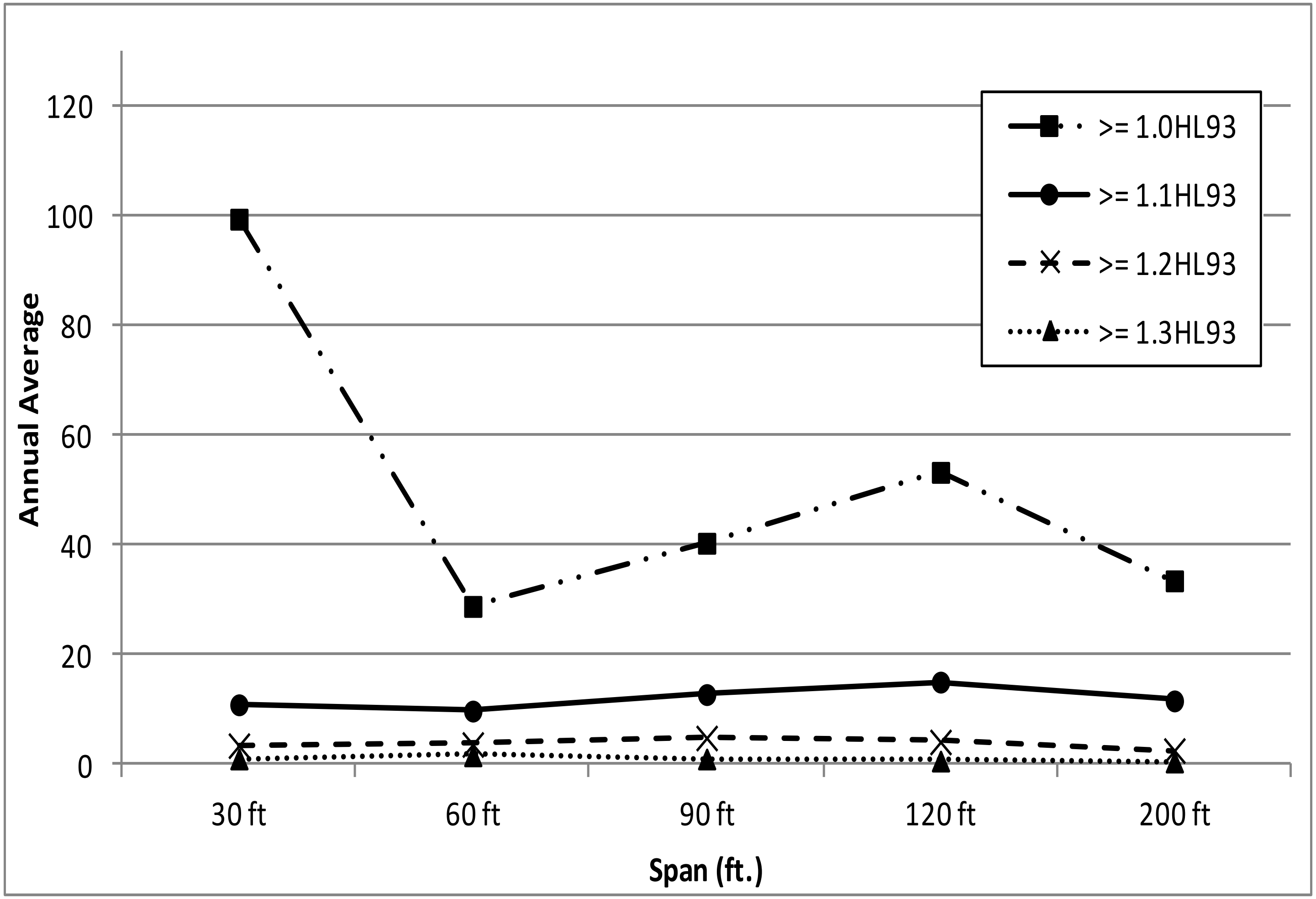
Live Load for Service II

- Annual average exceedances per site versus span

Site	MOMENT – Exceedances Per Year																			
	Ratio Truck/HL-93 >= 1.0					Ratio Truck/HL-93 >= 1.1					Ratio Truck/HL-93 >= 1.2					Ratio Truck/HL-93 >= 1.3				
	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft
AZ SPS-1	4	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
AS SPS-2	0	2	6	5	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
AR SPS-2	14	10	17	10	0	2	7	3	0	0	0	3	0	0	0	0	0	0	0	0
CO SPS-2	0	5	6	6	2	0	2	5	4	0	0	0	2	0	0	0	0	0	0	0
DE SPS-1	140	48	33	27	1	36	33	22	11	0	10	22	10	1	0	1	11	1	0	0
IL SPS-6	1	3	4	4	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
IN SPS-6	27	32	24	19	14	5	19	19	17	3	3	7	9	7	0	0	0	2	0	0
KS SPS-2	42	47	80	96	10	16	33	35	31	2	7	16	17	7	0	6	7	6	0	0
LA SPS-1	76	16	25	30	13	44	6	12	14	7	26	6	7	7	0	6	6	5	4	0
ME SPS-5	6	7	8	7	1	4	4	5	2	0	0	4	2	0	0	0	2	0	0	0
MD SPS-5	25	8	8	2	1	5	6	2	2	0	0	1	1	0	0	0	1	0	0	0
MN SPS-5	9	8	18	19	2	7	5	6	5	0	4	2	2	1	0	2	1	1	0	0
NM SPS-1	1	1	1	3	0	0	1	1	1	0	0	0	0	0	0	0	0	0	0	0
NM SPS-5	12	7	7	9	4	4	1	1	3	0	3	0	0	0	0	0	0	0	0	0
PA SPS-6	155	45	22	21	1	32	22	17	14	1	13	17	13	1	0	3	13	2	0	0
TN SPS-6	2085	29	8	7	0	53	4	4	0	0	5	1	0	0	0	1	0	0	0	0
VA SPS-1	7	10	1	2	1	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
WI SPS-1	6	3	5	4	2	1	0	3	3	1	0	0	1	1	0	0	0	0	0	0
CA Antelope EB	0	13	25	31	25	0	1	0	0	7	0	0	0	0	0	0	0	0	0	0
CA Antelope WB	0	30	71	100	84	0	7	6	19	40	0	0	0	1	13	0	0	0	0	1
CA Bowman	0	3	3	8	16	0	0	0	3	3	0	0	0	0	3	0	0	0	0	0
CA LA-710 NB	10	99	150	153	85	1	34	55	56	16	0	7	26	21	0	0	0	4	1	0
CA LA-710 SB	3	62	105	111	54	1	17	45	48	14	0	3	18	19	0	0	0	1	1	0
CA Lodi	0	110	137	281	417	0	5	19	55	168	0	0	1	2	38	0	0	0	0	2
FL I-10	279	141	159	264	152	81	41	47	77	38	23	16	14	18	5	10	5	4	5	2
FL I-95	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MS I-10	41	48	53	53	44	26	24	34	36	24	8	2	11	21	2	2	2	2	2	1
MS I-55UI	0	4	5	11	8	0	0	0	1	3	0	0	0	0	0	0	0	0	0	0
MS I-55R	142	100	255	349	89	20	31	50	61	33	7	8	17	22	20	2	3	5	8	9
MS US-49	0	3	11	13	7	0	0	2	1	0	0	0	0	0	0	0	0	0	0	0
MS US-61	0	1	5	8	6	0	0	1	2	1	0	0	1	1	0	0	0	0	0	0
FL US-29	1291	995	651	496	204	673	510	332	253	109	371	274	179	123	53	183	165	85	61	22
Annual Average	99.6	28.9	40.4	53.4	33.6	11.0	9.8	12.8	15.1	11.7	3.5	3.7	4.9	4.2	2.6	1.1	1.7	1.1	0.7	0.5

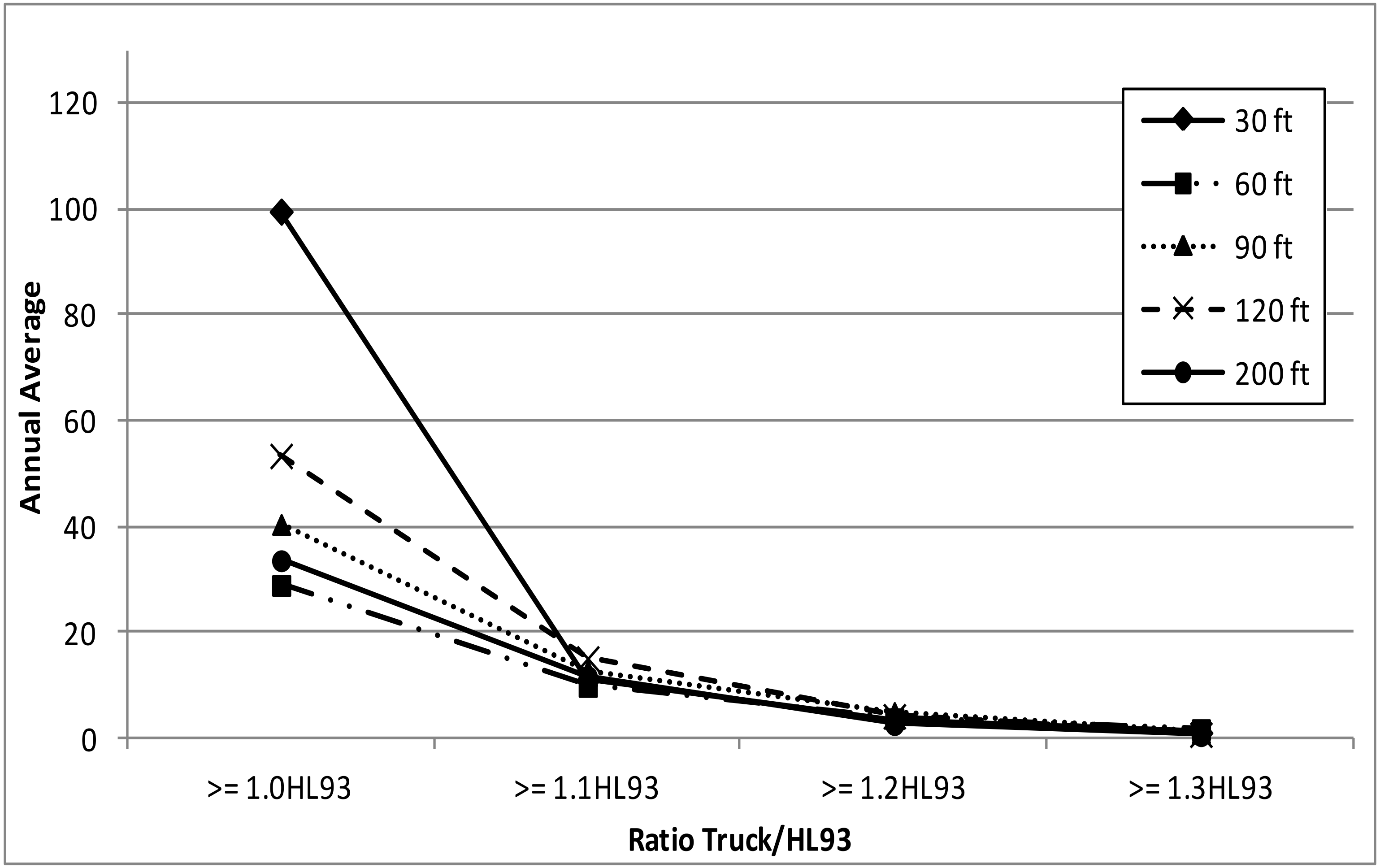
Live Load for Service II *(continued)*

- Annual average exceedances per site versus span



Live Load for Service II *(continued)*

- Annual average exceedances per site versus ratio to HL93



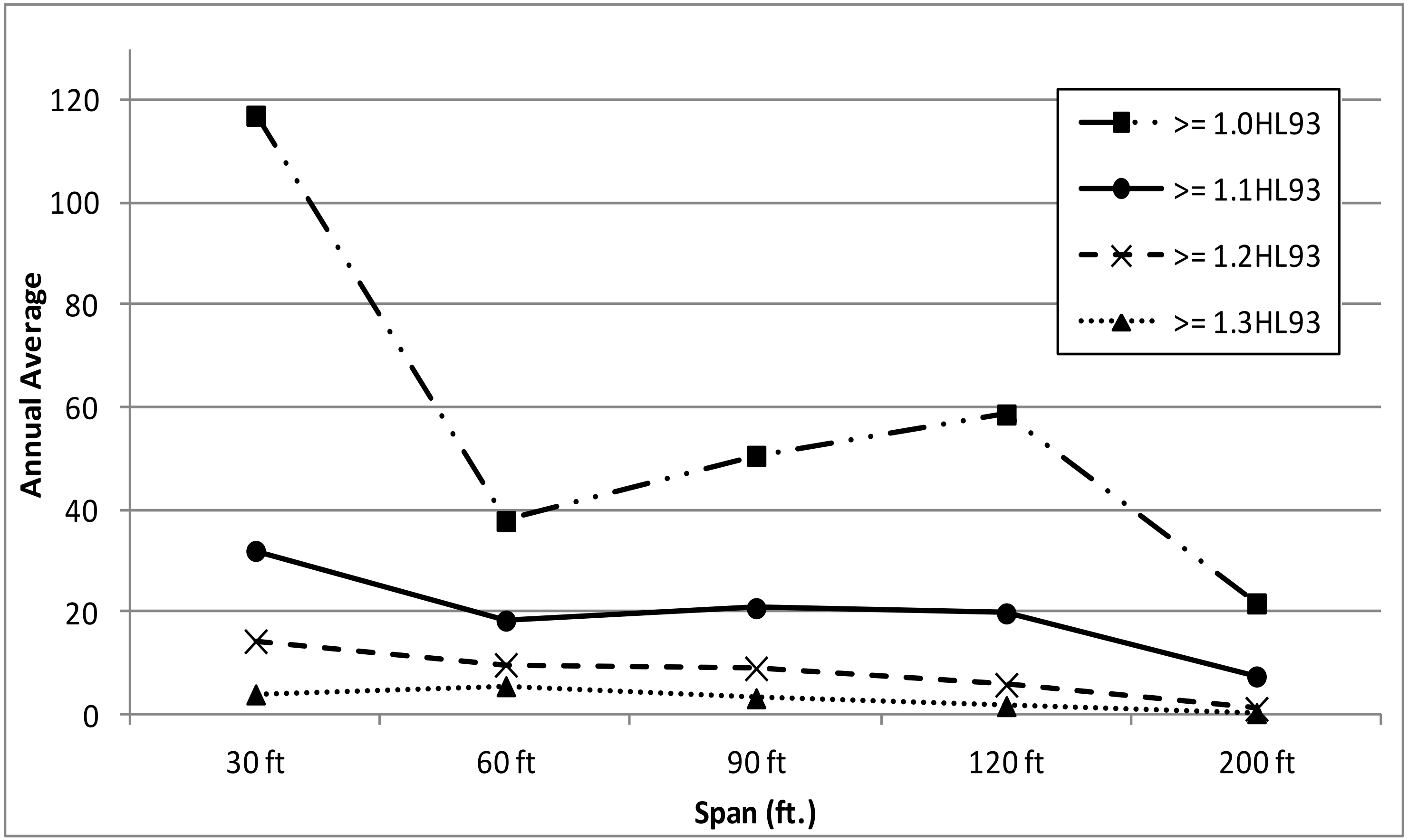
Live Load for Service II *(continued)*

- Annual average exceedances per site versus span scaled to ADTT 2,500

Site	MOMENT – Events Per Year Scaled to ADTT = 2500																			
	Ratio Truck/HL-93 >= 1.0					Ratio Truck/HL-93 >= 1.1					Ratio Truck/HL-93 >= 1.2					Ratio Truck/HL-93 >= 1.3				
	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft	30 ft	60 ft	90 ft	120 ft	200 ft
AZ SPS-1	103	0	0	26	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
AS SPS-2	0	1	4	3	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
AR SPS-2	8	5	9	5	0	1	4	2	0	0	0	2	0	0	0	0	0	0	0	0
CO SPS-2	0	13	16	16	5	0	5	13	11	0	0	0	5	0	0	0	0	0	0	0
DE SPS-1	633	217	149	122	5	163	149	100	50	0	45	100	45	5	0	5	50	5	0	0
IL SPS-6	1	3	4	4	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
IN SPS-6	79	94	69	54	39	15	54	54	49	10	10	20	25	20	0	0	0	5	0	0
KS SPS-2	80	90	153	183	19	31	63	67	59	4	13	31	32	13	0	11	13	11	0	0
LA SPS-1	808	170	266	319	138	468	64	128	149	74	277	64	74	74	0	64	64	53	43	0
ME SPS-5	30	35	40	35	5	20	20	25	10	0	0	20	10	0	0	0	10	0	0	0
MD SPS-5	139	44	44	11	6	28	33	11	11	0	0	6	6	0	0	0	6	0	0	0
MN SPS-5	148	131	296	312	33	115	82	99	82	0	66	33	33	16	0	33	16	16	0	0
NM SPS-1	8	8	8	16	0	0	8	8	8	0	0	0	0	0	0	0	0	0	0	0
NM SPS-5	45	/	/	*	8	8	2	2	3	0	3	0	0	0	0	0	0	0	0	0
PA SPS-6	95	27	13	13	1	20	13	10	9	1	8	10	8	1	0	2	8	1	0	0
TN SPS-6	1173	16	4	4	0	30	2	2	0	0	3	1	0	0	0	1	0	0	0	0
VA SPS-1	25	35	4	7	4	0	0	4	4	0	0	0	0	0	0	0	0	0	0	0
WI SPS-1	24	12	20	16	8	4	0	12	12	4	0	0	4	4	0	0	0	0	0	0
CA Antelope EB	0	10	20	24	20	0	1	0	0	5	0	0	0	0	0	0	0	0	0	0
CA Antelope WB	0	20	48	68	57	0	5	4	13	27	0	0	0	1	9	0	0	0	0	1
CA Bowman	0	1	1	4	8	0	0	0	1	1	0	0	0	0	1	0	0	0	0	0
CA LA-710 NB	2	20	31	31	17	0	7	11	11	3	0	1	5	4	0	0	0	1	0	0
CA LA-710 SB	1	12	21	22	11	0	3	9	9	3	0	1	4	4	0	0	0	0	0	0
CA Lodi	0	25	32	65	96	0	1	4	13	39	0	0	0	1	9	0	0	0	0	1
FL I-10	151	76	86	142	82	44	22	26	42	21	12	9	8	9	3	6	3	2	3	1
FL I-95	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
MS I-10	0	2	3	6	4	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
MS I-55UI	0	2	3	6	4	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
MS I-55R	93	66	167	229	58	13	21	33	40	22	5	5	11	14	13	1	2	3	5	6
MS US-49	0	2	8	10	5	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
MS US-61	0	6	23	40	29	0	0	6	11	6	0	0	6	6	0	0	0	0	0	0
FL US-29	2922	2252	1473	1122	462	1524	1155	751	572	247	840	621	406	278	119	413	373	191	138	49
Annual Average	117.0	37.8	50.6	58.7	21.7	32.0	18.4	20.8	19.8	7.5	14.3	9.7	9.1	5.8	1.2	4.0	5.6	3.2	1.7	0.3

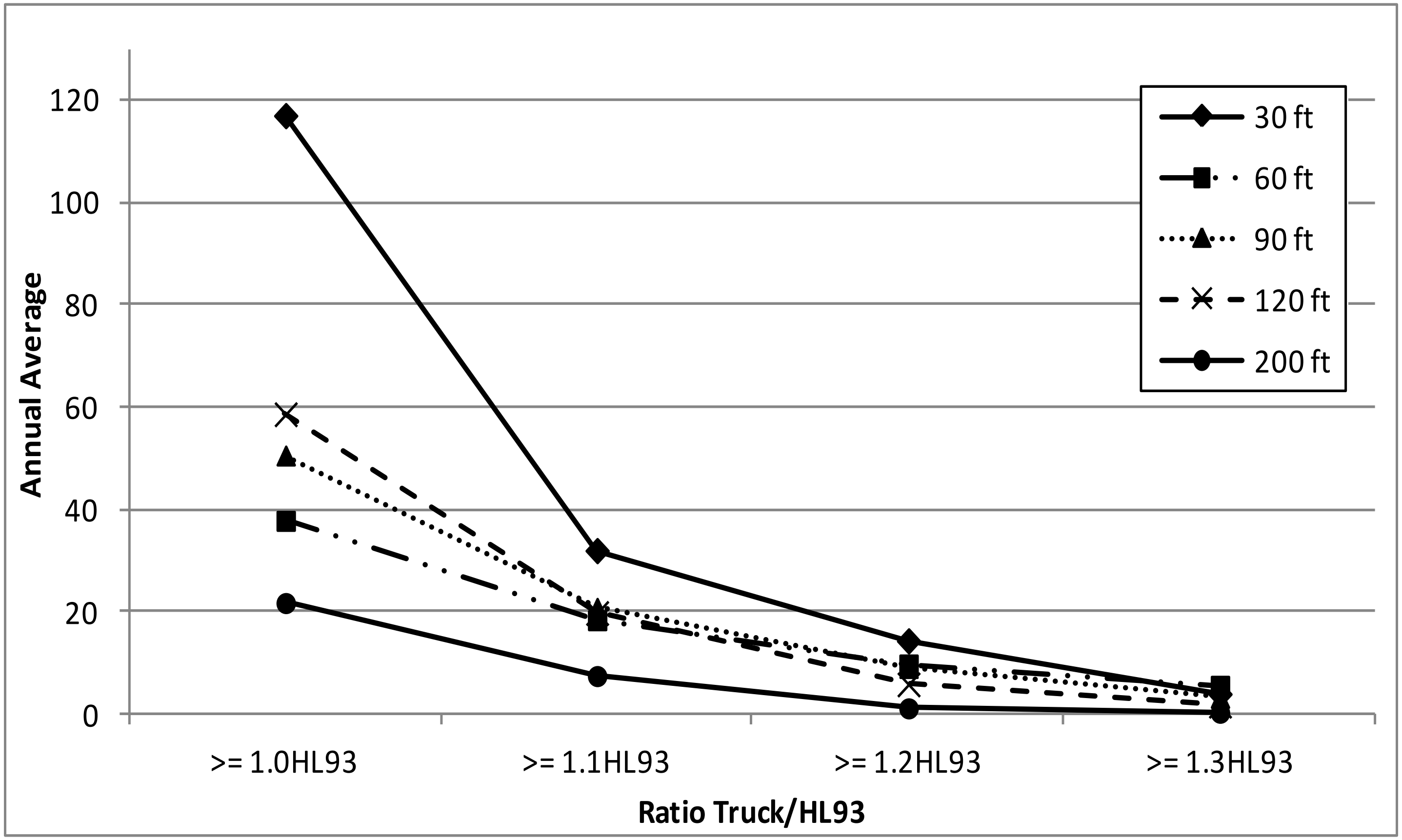
Live Load for Service II *(continued)*

- Annual average exceedances per site versus span scaled to ADTT 2,500



Live Load for Service II *(continued)*

- Annual average exceedances per site versus ratio to HL93 scaled to ADTT 2,500



Calibration of Service II

- A database of 41 existing bridges was analyzed using 5/3 of single lane of LFD live load (the load likely used in the existing bridge design). The analysis was repeated, assuming multiple lanes of HL 93. The inherent reliability of existing structures was determined.

Live Load	β	COV
Single Lane (Reality)	1.8	0.32
Multiple Lane (Assumed)	1.6	0.92

Calibration for Service II *(continued)*

- The target reliability was taken equal to the reliability of the existing structures.
- Monte Carlo simulations were performed assuming HL 93 (the load confirmed by the WIM data study to represent the current traffic loads).
- The reliability index calculated was 1.8 with a COV of 0.9; very similar to the target reliability index.
- No revisions to AASHTO LRFD seemed necessary.

Calibration of the Fatigue Limit State

Background of Fatigue Provisions

- The current AASHTO fatigue design approach was developed in the 1970s.
- Fatigue cracking is caused by the accumulation of fatigue damage caused by the passage of vehicles producing varying load effects.
- The standard design truck with axle spacings of 14 ft. does not resemble the majority of actual trucks.
- The design truck with a rear axle spacing of 30 ft. is more representative of actual trucks and was selected as the design load for fatigue.
- Fatigue provisions in AASHTO Standard Specifications and AASHTO LRFD Specifications have the same background but the presentation is different.

Background of Fatigue Provisions *(continued)*

- In AASHTO standard specifications, the allowable fatigue stress range is based on the number of cycles selected from a table (100,000; 500,000; 2,000,000; and over 2,000,000).
- In AASHTO LRFD, fatigue design is based on a stress range threshold that varies for different fatigue categories. The threshold determines whether the detail has an infinite or finite fatigue life. If the detail has a finite fatigue life, the stress range limit is determined for a number of cycles based on the ADTT and a bridge life of 75 years.
- The approach in AAHSTO LRFD is more transparent.

Background of Fatigue Provisions *(continued)*

- In AASHTO Standard Specifications

TABLE 10.3.1A Allowable Fatigue Stress Range

Redundant Load Path Structures[a]				
	Allowable Range of Stress, F_{sr} (ksi)[b]			
Category (See Table 10.3.1B)	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	63 (49)[c]	37 (29)[c]	24 (18)[c]	24 (16)[c]
B	49	29	18	16
B′	39	23	14.5	12
C	35.5	21	13	10 12[d]
D	28	16	10	7
E	22	13	8	4.5
E′	16	9.2	5.8	2.6
F	15	12	9	8

Nonredundant Load Path Structures				
	Allowable Range of Stress, F_{sr} (ksi)[b]			
Category (See Table 10.3.1B)	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	50 (39)[c]	29 (23)[c]	24 (16)[c]	24 (16)[c]
B	39	23	16	16
B′	31	18	11	11
C	28	16	10 12[d]	9 11[d]
D	22	13	8	5
E[e]	17	10	6	2.3
E′	12	7	4	1.3
F	12	9	7	6

Background of Fatigue Provisions *(continued)*

- AASHTO LRFD

Table 6.6.1.2.5-3—Constant-Amplitude Fatigue Thresholds

Detail Category	Threshold (ksi)
A	24.0
B	16.0
B′	12.0
C	10.0
C′	12.0
D	7.0
E	4.5
E′	2.6
A 325 Bolts in Axial Tension	31.0
A 490 Bolts in Axial Tension	38.0

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \quad (6.6.1.2.5\text{-}2)$$

in which:

$$N = (365)(75)n(ADTT)_{SL} \quad (6.6.1.2.5\text{-}3)$$

where:

A = constant taken from Table 6.6.1.2.5-1 (ksi^3)

n = number of stress range cycles per truck passage taken from Table 6.6.1.2.5-2

$(ADTT)_{SL}$ = single-lane $ADTT$ as specified in Article 3.6.1.4

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold taken from Table 6.6.1.2.5-3 (ksi)

Background of Fatigue Provisions *(continued)*

- For any detail, there is a level of stress under the fatigue design load below which no fatigue cracking is expected to take place regardless of the number of cycles during the design life of the bridge. This results in “infinite fatigue life”. The load factor for infinite fatigue life, i.e., Fatigue I, is selected such that it is not exceeded more than one time in each 10,000 truck passages on the bridge.
- In case Fatigue I requirements are not satisfied, the detail has a finite fatigue life (Fatigue II limit state). The load factor is selected such that the fatigue damage caused by the passage of an “average” truck is equal to that caused by the factored fatigue truck.

Background of Fatigue Provisions *(continued)*

- Based on load studies performed during the development of the original fatigue provisions, the load factors used in AASHTO LRFD up to 2016 were:

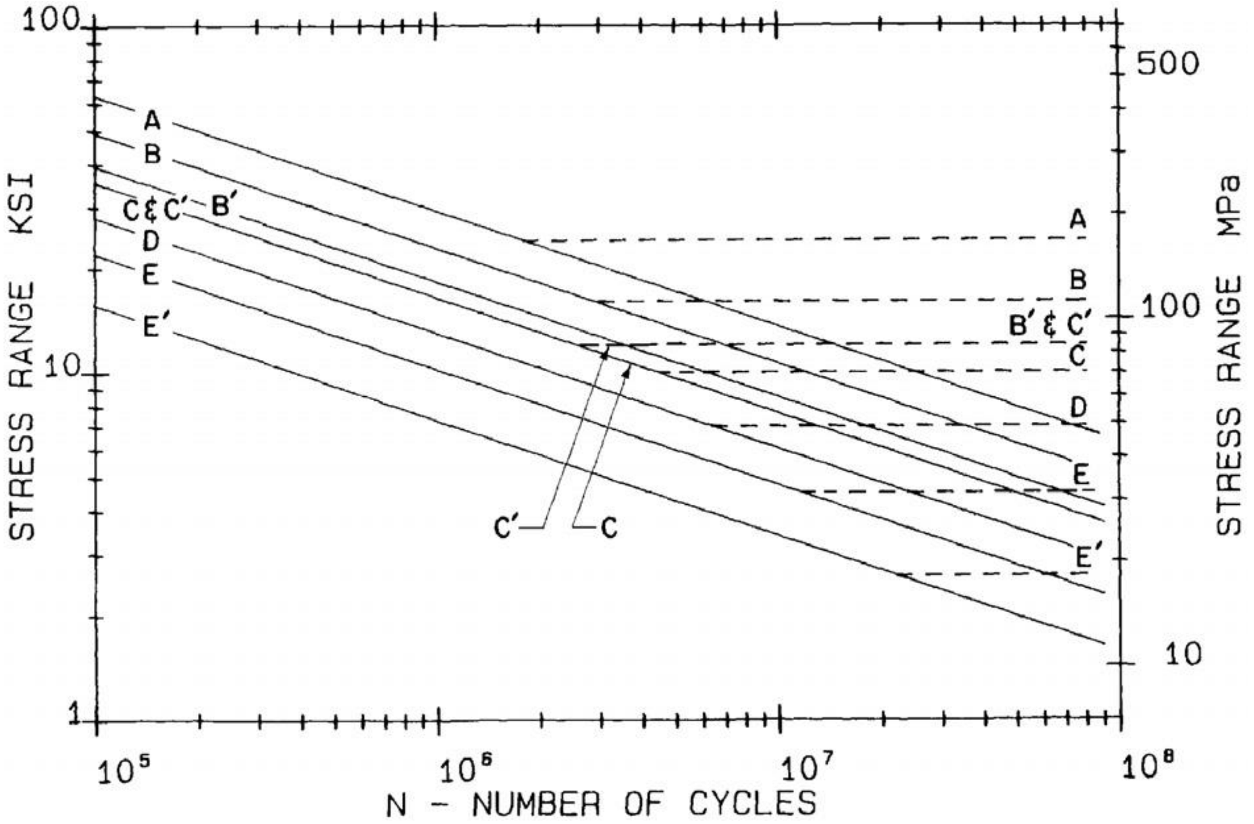
Fatigue Limit State Load Combination	LL Load Factor
Fatigue I	1.50
Fatigue II	0.75

- Based on load studies performed under SHRP2 *Service Limit State Design*, the load factors for fatigue were revised in 2017 to:

Fatigue Limit State Load Combination	LL Load Factor
Fatigue I	1.75
Fatigue II	0.80

Background of Fatigue Provisions *(continued)*

- S-N Curve for fatigue



Background of Fatigue Provisions *(continued)*

- The finite-life fatigue resistance (in other words, the allowable stress range to reach a certain number of cycles) is defined by the general equation:

$$\Delta\sigma = \left(\frac{A}{N}\right)^{\frac{1}{3}}$$

Where:

A = a constant defined for each detail category, and

N = the number of cycles to failure

Background of Fatigue Provisions *(continued)*

Constant A for mean fatigue resistance

Detail Category	A (times 10^8)
A	700
B	240
B′	146
C	57
C′	57
D	35
E	18
E′	10

Statistical Parameters for Finite-life Fatigue Resistance

Detail Category	Bias	Coefficient of Variation
A	2.8	0.59
B	2.0	0.71
B′	2.4	0.67
C	1.3	0.83
C′	1.3	0.83
D	1.6	0.77
E	1.6	0.77
E′	2.5	0.63

Background of Fatigue Provisions *(continued)*

Infinite-life nominal constant amplitude fatigue threshold

Detail Category	Nominal Constant-Amplitude Fatigue Threshold (ksi)
A	24
B	16
B′	12
C	10
C′	12
D	7
E	4.5
E′	2.6

Due to the time it takes to test for infinite fatigue life, these limits have not been as thoroughly verified as the finite-fatigue life.

Database of Fatigue Testing

- Statistics of fatigue resistance were determined using a comprehensive database of fatigue test data and included:
  - Constant and variable amplitude fatigue test results
  - Various welded steel bridge detail types
  - Data from various domestic and international sources
- Based on regression analysis performed on the stress range versus cycle relation:

$$log\ N = log\ A - B\, log\ S_r \quad \text{or} \quad N = A S_r^{-B}$$

where

N = number of cycles to failure

S_r = constant amplitude stress range, ksi

$log\ A$ = log-N-axis intercept of S-N curve from *AASHTO LRFD*

B = slope of the curve

Load Uncertainties

- The mean and COV were determined from the study of the WIM data:

Limit State	Mean	COV
Fatigue I	2.0*	0.12
Fatigue II	0.8	0.07

* was later reduced to 1.75

Resistance Uncertainties

- Test data was analyzed to determine an effective stress range for each detail using:

$$(S_r)_{eff} = \left(\Sigma\, \gamma_i S_{ri}^{\ 3}\right)^{1/3}$$

$(S_r)_{eff}$ = effective constant amplitude stress range

γ_i = percentage of cycles at a particular stress range

S_{ri} = constant amplitude stress range for a group of cycles (ksi)

- The fatigue damage parameter is then determined by introducing the number of cycles

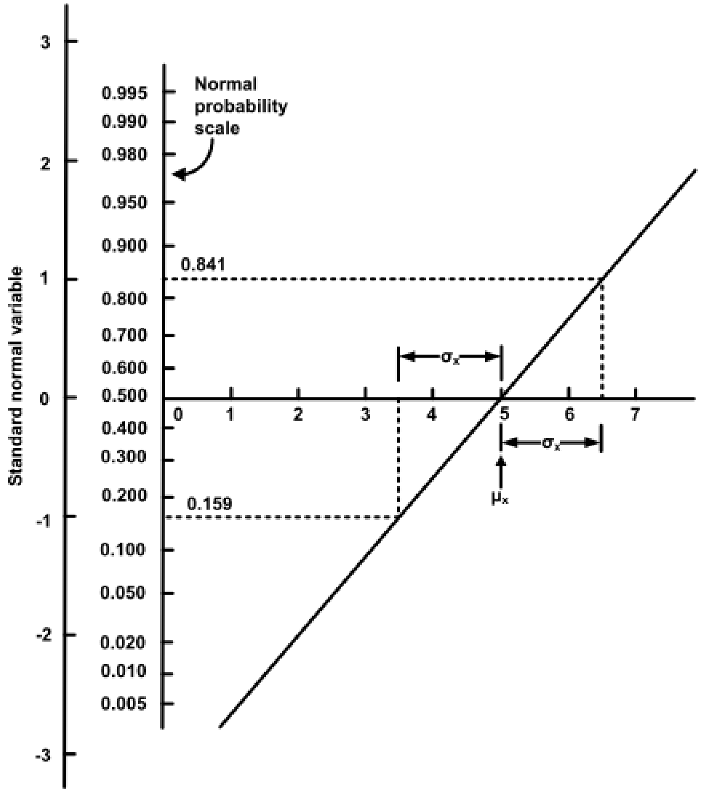
$$S_{fi} = \left(N * S_{ri}^{\ 3}\right)^{1/3}$$

Where: S_{fi} = fatigue damage parameter

- Normal distribution was determine to best characterize fatigue data

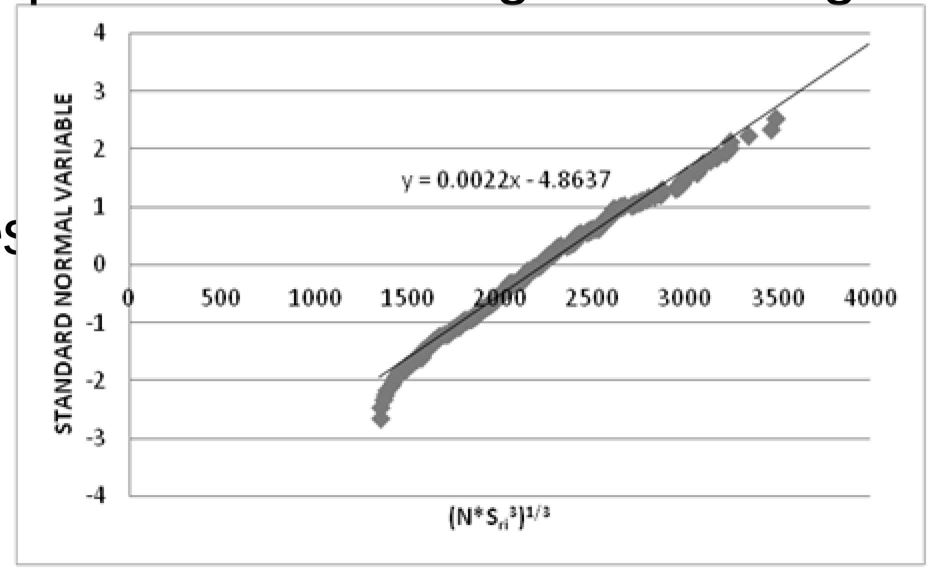
Resistance Uncertainties *(continued)*

- Use of normal probability paper
- The cumulative distribution function (CDF) is a straight line when the data follows normal distribution
- Steeper line reflects smaller standard deviation



Resistance Uncertainties *(continued)*

- Test data was plotted on probability paper for different fatigue categories
- Data was filtered to only include points that fit the detail fatigue behavior. Typically the lower tail of the data was included as it contains the points where fatigue cracking is to occur
- Regression analysis was used to determine bes
- Statistical parameters were determined for each fatigue category.



Category C and C’ are shown

Resistance Uncertainties *(continued)*

- Mean value of the stress parameter is the intersection with the horizontal axis.
- The inverse of the slope of the line is the standard deviation.
- The coefficient of variation (COV) is the standard deviation divided by the mean value.
- The COV and the mean of the fatigue resistance were used along with the nominal fatigue resistance to determine the bias of the data.
- The nominal value of the chosen fatigue parameter was calculated using AASHTO LRFD Eq. 6.6.1.2.5-2 and rearranged to achieve the relationship in terms of the desired fatigue damage parameter as shown in next slide.

Resistance Uncertainties *(continued)*

- $S_{f\_AASHTO} = \left(N * Sr^3\right)^{1/3} = A^{1/3}$

  S_{f_AASHTO} = nominal value of the fatigue parameter using *AASHTO LRFD* for each detail category

  A = constant taken from *AASHTO LRFD* Table 6.6.1.2.5-1 for the various detail categories

- The bias value is then determined as

  $$Bias = S_{f\_Mean} / S_{f\_AASHTO}$$

  S_{f_Mean} = mean value of the fatigue parameter using the fatigue data for each detail category

Resistance Uncertainties *(continued)*

Statistical parameters of the resistance:

Category	Standard Deviation	COV	Bias	S_{f_Mean}	S_{f_AASHTO}	Cutoff Standard Normal Variable
A	1000.0	0.24	1.43	4167	2924	1
B	666.7	0.22	1.34	3077	2289	1
B’	250.0	0.11	1.28	2336	1827	1
C and C’	454.6	0.21	1.35	2210	1638	1
D	185.2	0.10	1.36	1773	1300	1
E	140.9	0.12	1.17	1207	1032	1
E’	232.6	0.20	1.56	1140	730	1

The reliability indices inherent for various fatigue category were then determined using Monte Carlo simulations.

Calibration of the Fatigue Limit State

Proposed resistance factors and associated reliability index:

Fatigue I

Detail Category	Proposed Resistance Factor, ϕ	Reliability Index, β
A	1.0	1.2
B	1.0	1.1
B’	1.10	0.9
C	1.0	1.2
C’	1.0	1.2
D	1.15	1.1
E	1.0	0.9
E’	1.20	1.0

Fatigue II

Detail Category	Proposed Resistance Factor, ϕ	Reliability Index, β
A	1.0	1.0
B	1.0	0.9
B’	1.0	1.0
C	1.0	0.9
C’	1.0	0.9
D	0.95	1.0
E	1.10	1.0
E’	0.90	1.0

Calibration of the Fatigue Limit State *(continued)*

In order not to use variable resistance factors, the desired reliability index could also be achieved by using a resistance factor of 1.0 (same as has always been implied), a revised "A" constant and revised constant amplitude fatigue threshold:

Detail Category	Current Constant A Times 10^8	Proposed Constant A Times 10^8
A	250	250
B	120	120
B′	61	61
C	44	44
C′	44	44
D	22	21
E	11	12
E′	3.9	3.5

Detail Category	Current Constant-Amplitude Fatigue Threshold (ksi)	Proposed Constant-Amplitude Fatigue Threshold (ksi)
A	24	24
B	16	16
B′	12	13
C	10	10
C′	12	12
D	7	8.0
E	4.5	4.5
E′	2.6	3.1

Calibration of the Fatigue Limit State *(continued)*

- AASHTO decided not to revise the constant A or the constant amplitude fatigue threshold as these values have been entrenched in current practice and are needed in order to continue to match other design specifications.

Revisions to AASHTO LRFD for Fatigue of Steel Components

- Load factors (also applied to Fatigue I for reinforcement)

Fatigue I— *LL*, *IM* & *CE* only	—	~~1.50~~ 1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL*, *IM* & *CE* only	—	~~0.75~~ 0.80	—	—	—	—	—	—	—	—	—	—	—	—

Revisions to AASHTO LRFD for Fatigue of Steel Components *(continued)*

- Number of cycles per truck passage:

Longitudinal Members	~~Span Length~~	
	~~>40.0 ft~~	~~≤40.0 ft~~
Simple Span Girders	1.0	~~2.0~~
Continuous Girders		
1) near interior support	1.5	~~2.0~~
2) elsewhere	1.0	~~2.0~~
Cantilever Girders	5.0	
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling	5.0	
Trusses	1.0	
Transverse Members	Spacing	
	> 20.0 ft	≤20.0 ft
	1.0	2.0

Revisions to AASHTO LRFD for Fatigue of Steel Components *(continued)*

- Revised ADTT equivalent to infinite fatigue life

Detail Category	75-yrs $(ADTT)_{SL}$ Equivalent to Infinite Life (trucks per day)
A	~~530~~ 690
B	~~860~~ 1120
B′	~~1035~~ 1350
C	~~1290~~ 1680
C′	~~745~~ 975
D	~~1875~~ 2450
E	~~3530~~ 4615
E′	~~6485~~ 8485

Item #5

In Article C6.6.1.2.3, revise Eq. C6.6.1.2.3-1 as follows:

$$\cancel{75\_Year(ADTT)_{SL} = \frac{A}{\left[\frac{(\Delta F)_{TH}}{2}\right]^3 (365)(75)(n)}}$$

$$75 - Year(ADTT)_{SL} = \frac{A}{\left[\frac{0.80(\Delta F)_{TH}}{1.75}\right]^3 (365)(75)(n)}$$

Calibration for Fatigue of Concrete and Reinforcement

- Fatigue I limit state
- Fatigue of concrete is accounted for in the limit on compressive stress under all loads
- Pre-2017 equations for fatigue resistance of reinforcement:
  - For straight reinforcement bars and welded wire w/o cross welds in high stress region:

    $$\left(\Delta F\right)_{TH} = 24 - 20\, f_{min} / f_y$$

  - For welded wire with cross welds in high stress region:

    $$\left(\Delta F\right)_{TH} = 16 - 0.33\, f_{min}$$

Calibration for Fatigue of Concrete and Reinforcement *(continued)*

- An approach similar to that outlined above for steel components was used resulting in:
  - For straight reinforcement bars and welded wire without cross welds in high stress region:

$$(\Delta F)_{TH} = 26 - 22 f_{min} / f_y$$

  - For welded wire with cross welds in high stress region:

$$(\Delta F)_{TH} = 18 - 0.36 f_{min}$$

- The result is a moderate increase in the fatigue resistance of reinforcement (less than 2 ksi).

Key Points

- No revisions in the specifications were required for cracking of reinforced concrete under service load (service I), deflections (Service I), or yielding of steel components under service loads (Service II).
- The load factor for fatigue limit state was increased from 1.5 to 1.75 for Fatigue I and from 0.75 to 0.8 for Fatigue II.
- The number of load cycles per truck passage was revised.
- The fatigue stress threshold for reinforcement in tension was moderately increased.




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Foundation Deformations

Lecture 6
White Paper Chapter 1 to Chapter 4

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NCS GeoResources, LLC

June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Foundation Deformations

Chapter 1 – Introduction

Naresh C. Samtani, PhD, PE
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June 27, 2017

SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

U.S. Department of Transportation
Federal Highway Administration

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AASHTO

Learning Outcomes

- Understand the history of work related to foundation deformations
- Be able to locate material in White Paper titled "Incorporation of Foundation Deformations in *AASHTO LRFD* Bridge Design Process"
- Become familiar with the various conventions used in the White Paper

Work Under TRB-SHRP2

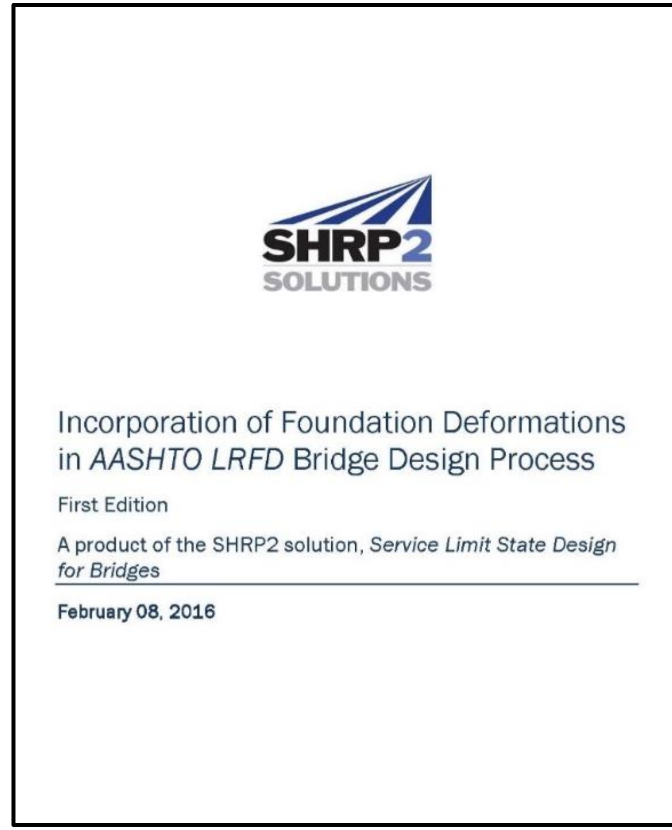
- General calibration process was developed for SLS and was revised to fit specific requirements for different limit states
- **The following limit states were calibrated**:
  - Fatigue I and Fatigue II limit states for steel components
  - Fatigue I for compression in concrete and tension in the reinforcement
  - Tension in prestressed concrete components
  - Crack control in decks
  - Service II limit state for yielding of steel and for bolt slip
  - Foundation deformation(s)

History of Work Related to Foundation Deformations

- TRB Project R19B work started in 2008 and final report published in January 2015
- Presentations related to calibration of foundation deformations at AASHTO SCOBS Annual T-15 Committee Meetings:
  - 2012, New Orleans, LA
  - 2014, Columbus, OH
  - 2015, Saratoga Springs, NY
  - 2016, Minneapolis, MN
- Presentation at AASHTO SCOBS Mid-Year Joint Meeting of T-15 and T-5 committees on October 28, 2015, in Chicago, IL; included a flow chart.
- Presentation at 2017 42nd Southwest Geotechnical Conference in Phoenix, AZ
- Development of examples, draft agenda items for T-15 and T-5 committees, and a white paper

Incorporation of Foundation Deformations – White Paper

- Based on Project R19B report and includes additional work beyond Project R19B
- Incorporates comments received at various meetings and presentations
- Will be updated as necessary.
- Latest copy can be found at the AASHTO SHRP2 R19B product page



SHRP2 SOLUTIONS

Incorporation of Foundation Deformations in *AASHTO LRFD* Bridge Design Process

First Edition

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

February 08, 2016

R19B Product Page

http://shrp2.transportation.org/Pages/R19B_ServiceLimitStateDesignforBridges.aspx

Special Acknowledgment

Dr. John Kulicki

Correlation of Presentation and White Paper

- The presentation slides closely follow the information in the white paper:
  - Reference to material in White Paper will be prefixed by “WP”, e.g., WP Figure 2-1 refers to Figure 2-1 in Chapter 2 of the White Paper

- Supplementary materials from references cited in the White Paper will be presented as needed

- Review agenda for topics related to foundation deformations in context of White Paper

Conventions for Style and Organization – Appendix A

- *AASHTO LRFD* for AASHTO LRFD Bridge Design Specifications
  - Necessary to fulfill AASHTO's citation requirements
  - Refers to 7th Edition (2014) and Interims
- AASHTO *Standard Specifications for Highway Bridges*
- Notations (Symbols): γ_{SE}, SE, Δ_d, L_S
- Font differences: Times New Roman (in AASHTO), Calibri (in some FHWA manuals), Arial (in these slides; sometimes *italicized*)
- Format: 2-column versus full-page (no column)
- Sections, Articles, Commentary, Tables, Figures
- Terminology: Settlement, movement, deformation

Key Points

- The effort to calibrate the service limit state for foundation deformations started with Project R19B in 2009

- A comprehensive White Paper is available for reference




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Foundation Deformations

Chapter 2 – Bridge Foundation Types and Deformations

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June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

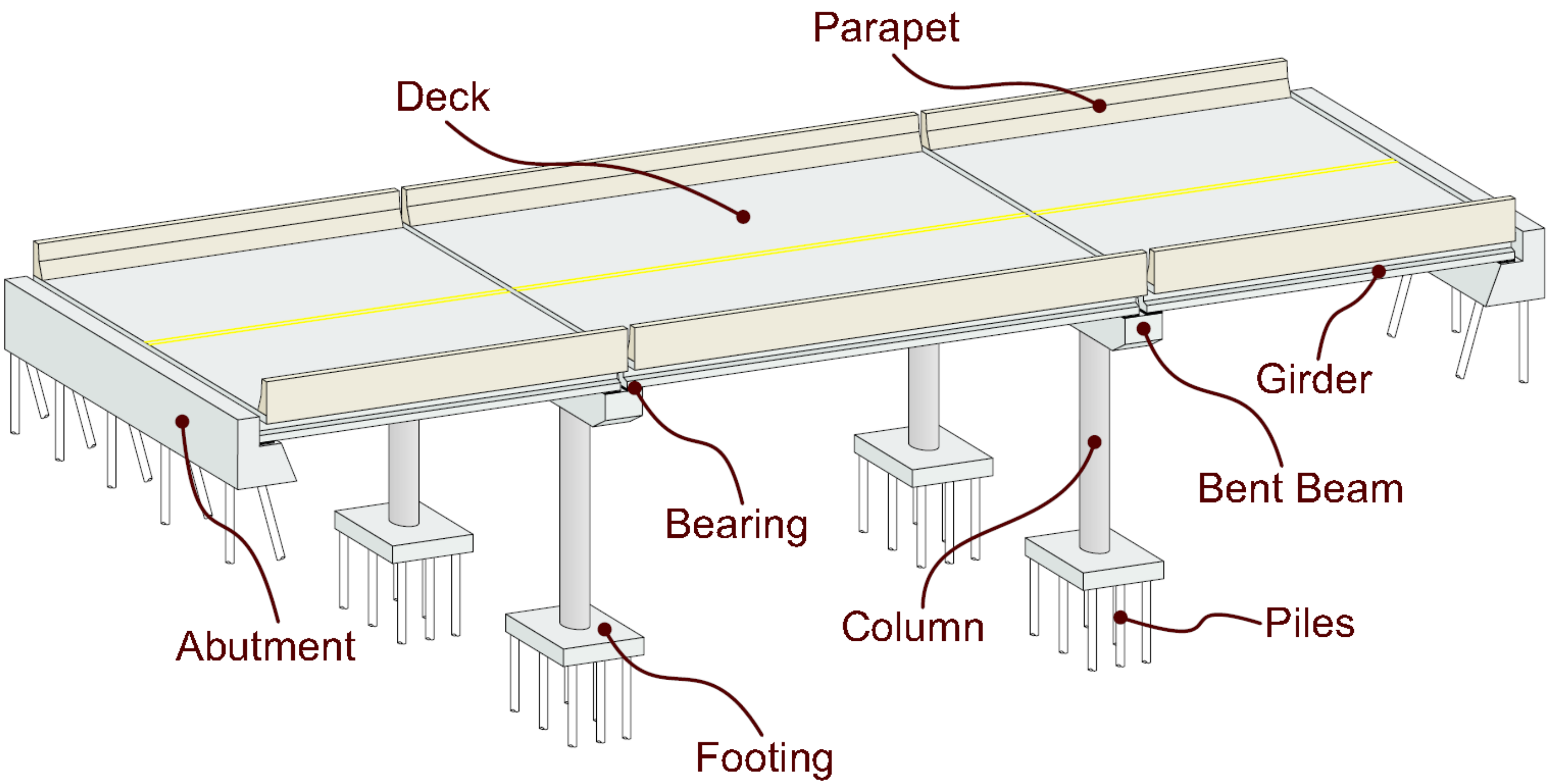
American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

- Define major components of a bridge structure
- Identify types of bridge foundations and deformations

Major Components of a Bridge Structure

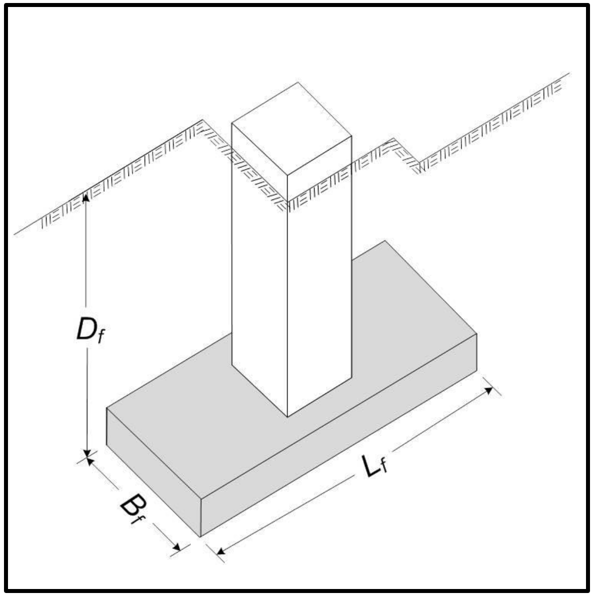
WP Figure 2-1



Reference: Nielson (2005)

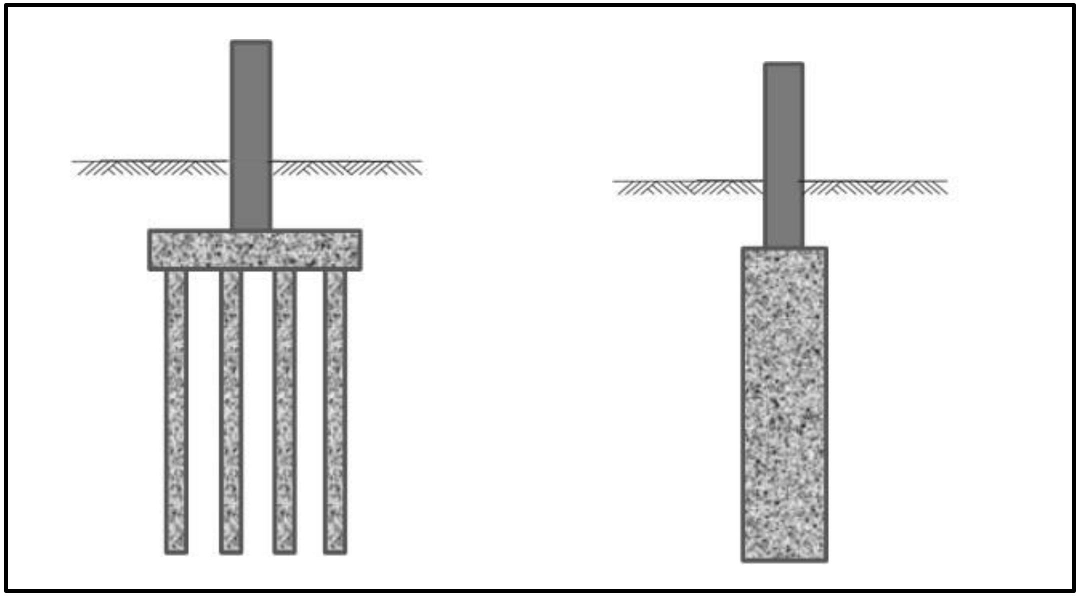
Foundation Types

WP Figure 2-2



Shallow foundations

WP Figure 2-3 (a) and (b)

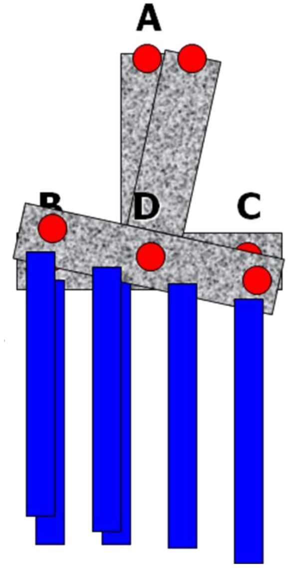


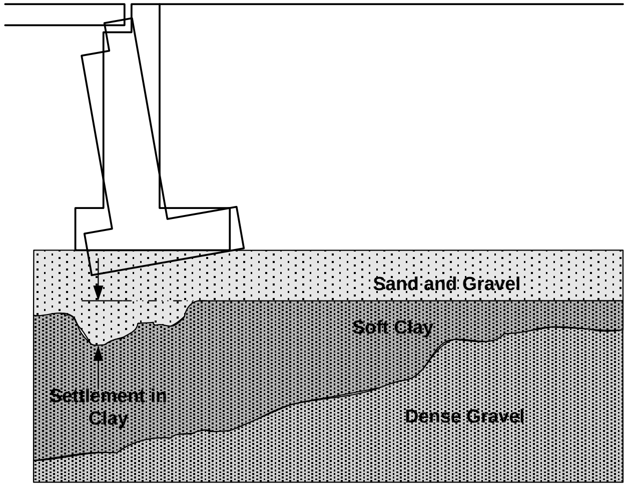
Deep foundations (group, single)

Foundation Deformations

- Foundation deformations can have multiple degrees of freedom
- Broad categorization of foundation deformations:
  - Vertical (settlement)
  - Lateral (horizontal)
  - Rotation (combined effect of vertical and lateral deformations)
  - Torsional
- Bridge foundations and other geotechnical features, such as approach embankments, should be designed so that their deformations will not cause damage to the bridge structure

Impact of Foundation Deformations

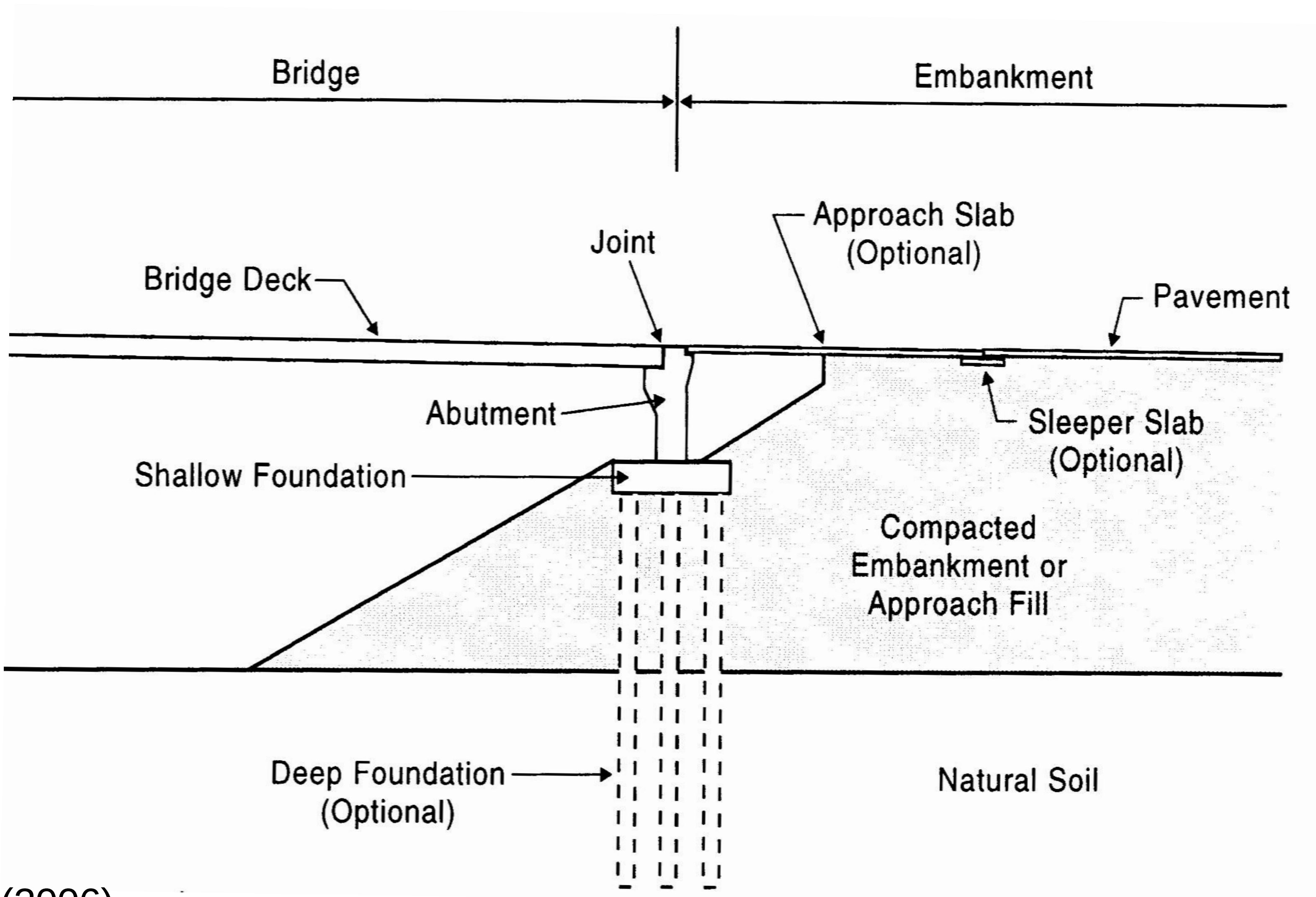




- Regardless of the type of foundation, the key point of interest is the effect of the foundation deformation on the various elements of the bridge substructure, and superstructure components above the foundations

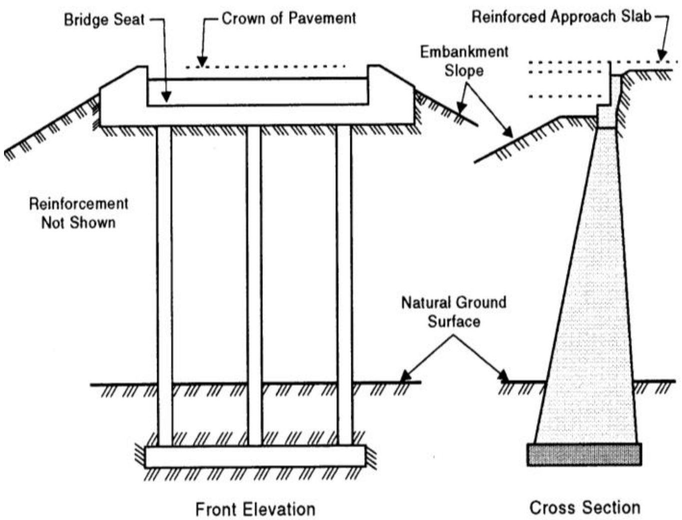
- Impact of foundations deformations could be more severe on superstructure and bearings particularly when lateral deformations are combined with settlements

Bridge Approach System

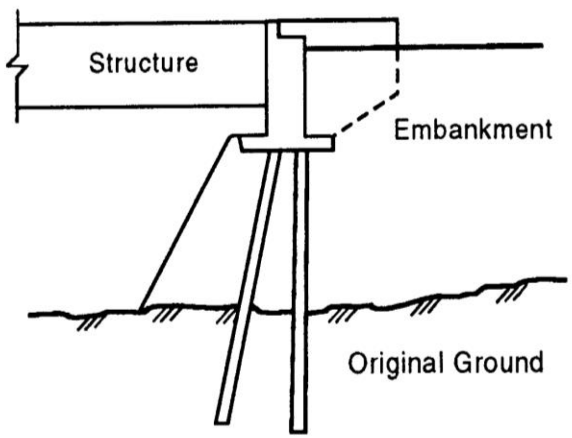


FHWA (2006)

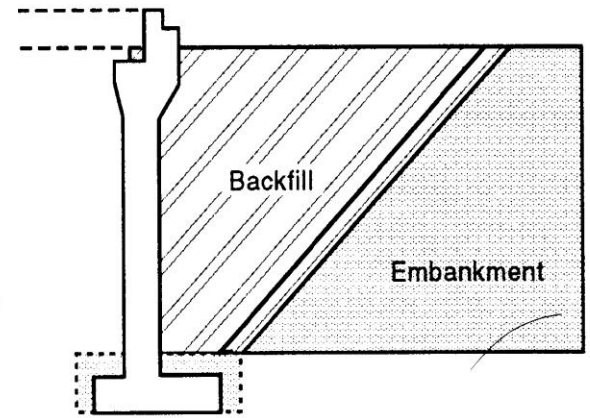
Many Types of Abutments



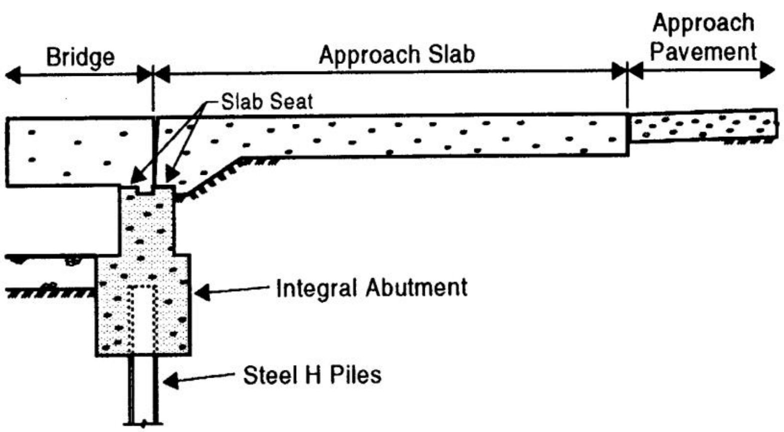
Typical Spill-Through Abutment



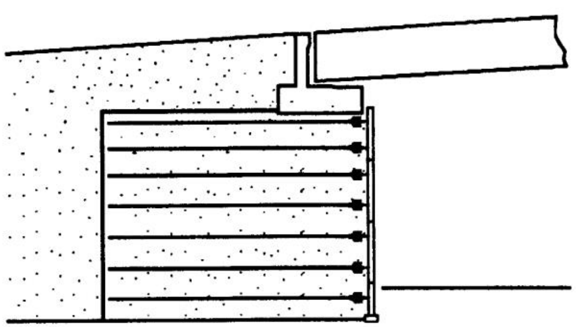
Typical Perched Abutment



Typical Full-Height Closed or High Abutment



Typical Integral Abutment



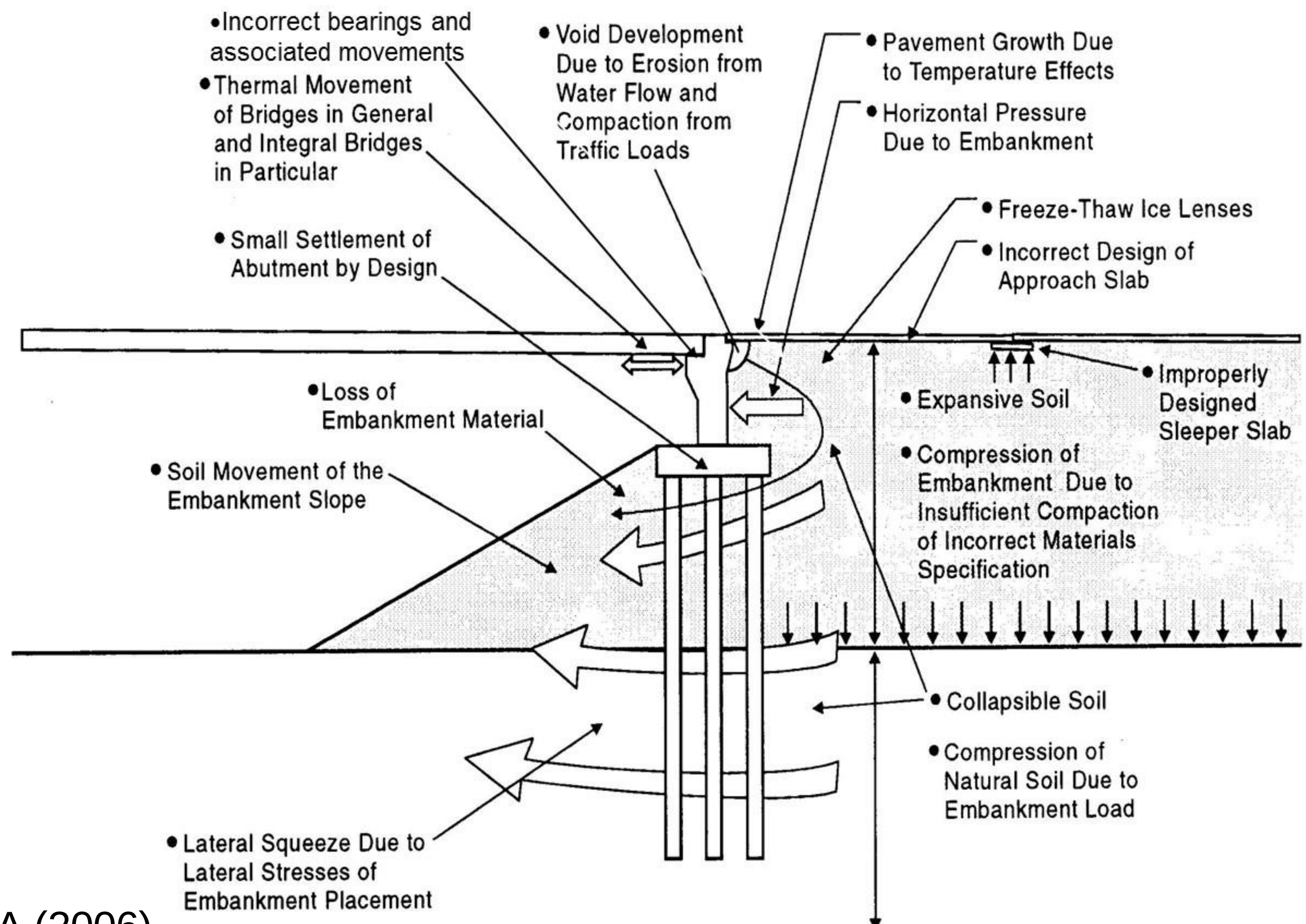
Mechanically Stabilized Abutment

FHWA (2006)

Approach Roadway Embankments
Major Design Considerations

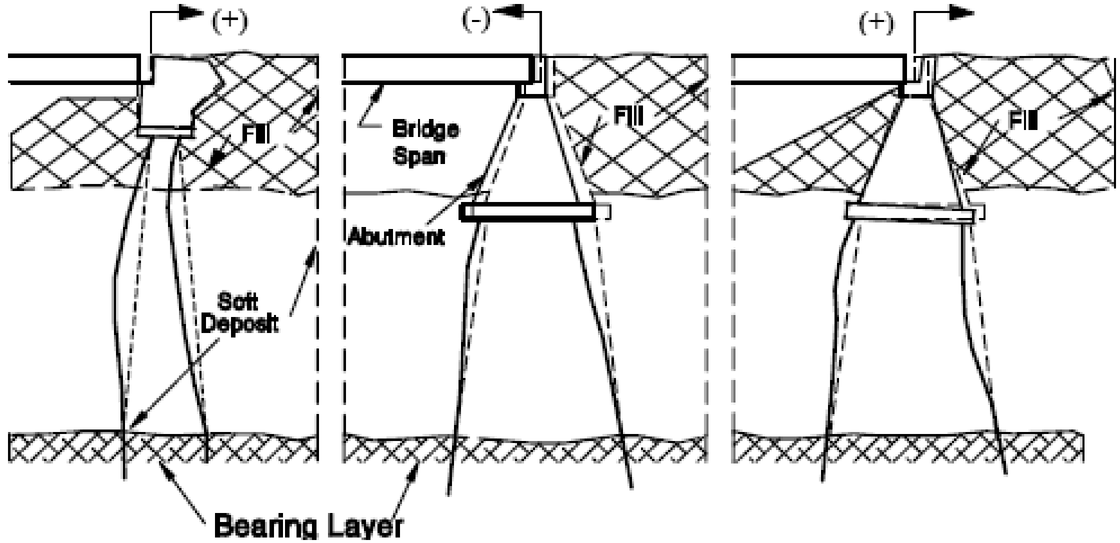
- Global Stability
- Deformations
  - Vertical
  - Lateral
- Effects on the Structure
  - Bump at the end of the bridge
  - Tilting

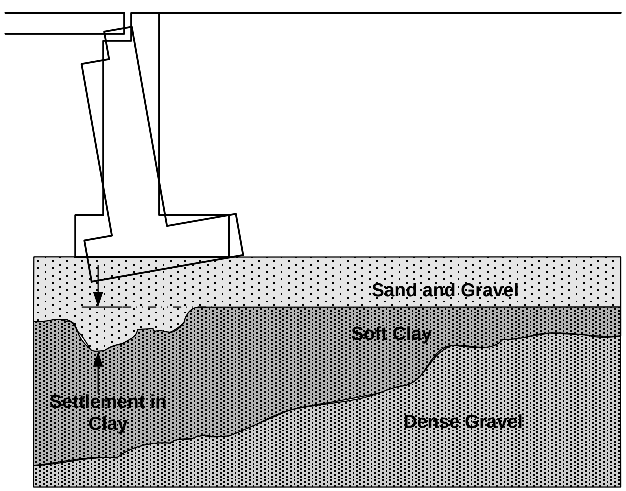
Potential Deformations at Bridge Approaches



FHWA (2006)

Identify Critical Deformation Modes

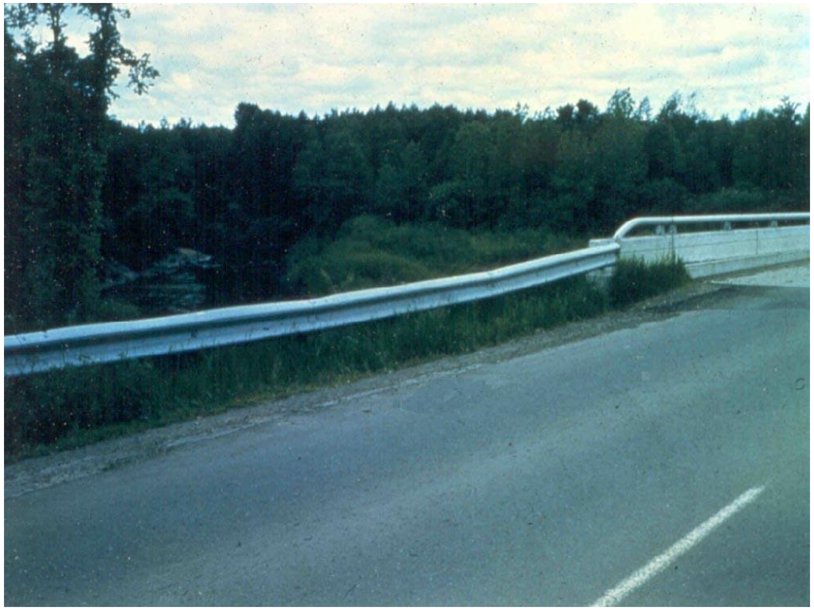




FHWA (2006)

Bumps

At End of Bridge



At End of Approach Slab

FHWA (2006)

Approach Roadway Deformations

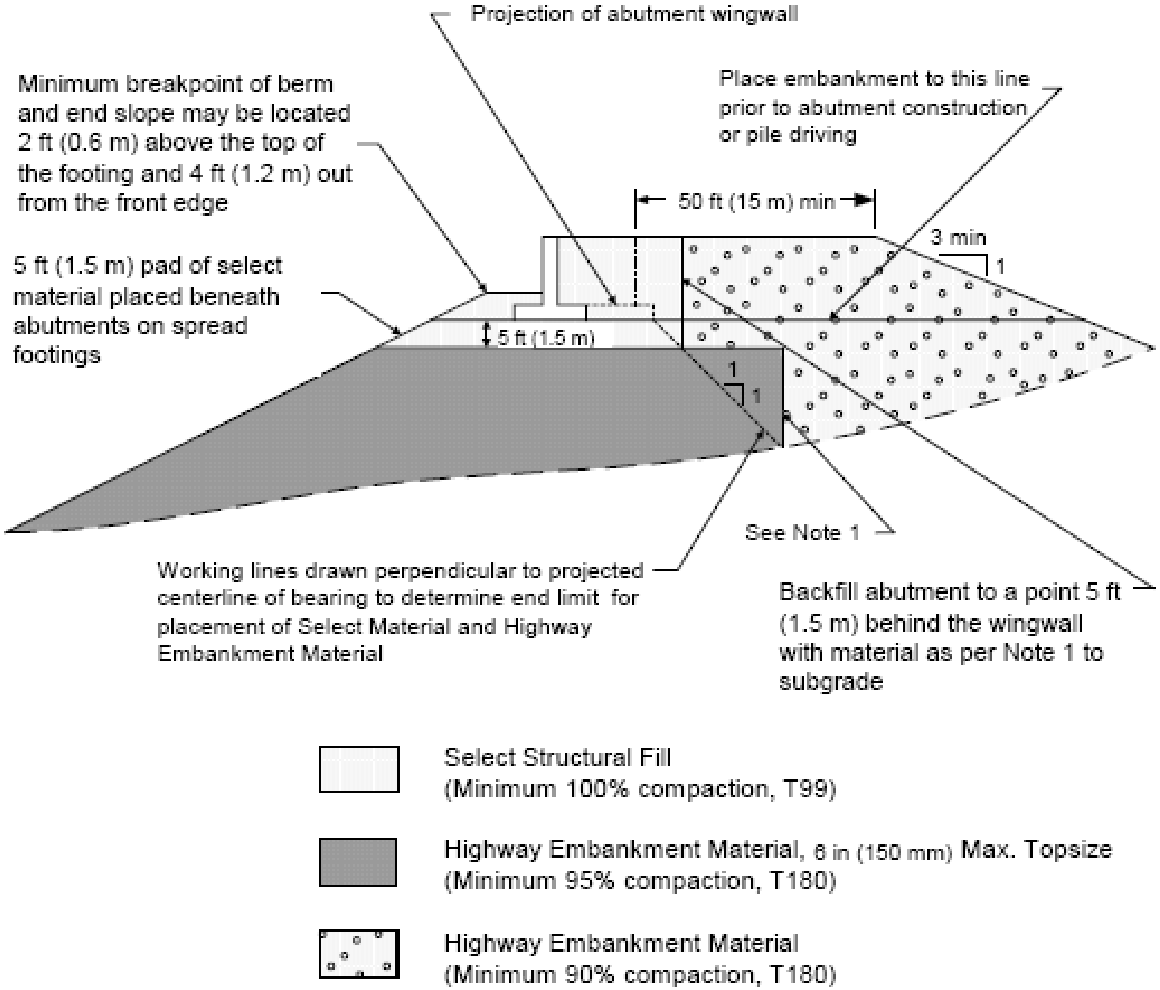
- Internal
  - Within the embankment fill
    - Due to compression of the fill materials
    - Poor drainage

- External
  - In the native soils below the embankment fill
    - Vertical and lateral deformation of native soils
    - Vertical: Immediate and consolidation settlements
    - Lateral: Squeeze (cause tilting of structures)

Control Geomaterials and Placement Procedures

FHWA (2006)

Control the Geomaterials at Abutments and Approaches



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

FHWA (2006)

Material and Construction Specifications

- Bridge designers need to have material and construction specifications that are consistent with service limit state calibrations

- The calibration of limit states is based on the assumption that appropriate material and construction specifications have been developed and implemented

- Minimum level of subsurface investigations as per Article 10.4 of *AASHTO LRFD* must be performed

Key Points

- Foundation deformations can occur in several ways

- The effects of foundation deformations need to be evaluated in terms of the ramifications for the bridge substructure and superstructure

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Foundation Deformations

Chapter 3 – Consideration of Foundation Deformations in AASHTO Bridge Design Specifications

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June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

- Identify and discuss the articles related to foundation deformations in the *AASHTO LRFD* Bridge Design Specifications

- Understand the treatment of foundation deformations in the AASHTO *Standard Specifications for Highway Bridges*

AASHTO LRFD Table 3.4.1-1

WP Figure 3-1

Superimposed Deformations

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	—	—	—	—	—	—	—	—	—	—	—	—

AASHTO LRFD Table 3.4.1-2

WP Figure 3-2

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	1.4	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			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.5	0.9
○ Thermoplastic Culverts		1.3	0.9
○ All others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

AASHTO LRFD Table 3.4.1-3

WP Figure 3-3

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 • using $I_{effective}$	 0.5 1.0	 0.5 1.0
Steel Substructures	1.0	1.0

Key to *AASHTO LRFD* Loads and Load Designations

WP Figure 3-4

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

Is *SE* Load Type Transient?

- As per **Article 3.3.2** of *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 because of *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.

Similarity between *SE* and *DD* Load Types

- *DD*: "downdrag force"

- 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

Category of Superimposed Deformations

- As per **Article 3.12** of *AASHTO LRFD*, the *SE* load type is considered to be similar to load types *TU*, *TG*, *SH*, *CR*, and *PS*, in that it generates force effects because of superimposed deformations.

- It is the induced force effects of foundation deformations that need to be included in the design of bridge structure. Therefore, the effect of foundation deformations has been included in the *SE* load type in *AASHTO LRFD*, Section 3, Table 3.4.1-1.

Is *SE* Load Type Only Applicable for Settlements?

- Although *AASHTO LRFD* uses the word "settlement," the broader meaning of *SE* load type applies to foundation movements or deformations, whether it is settlement (vertical deformation) or lateral deformation or rotation.
- **Article 3.12.1** of *AASHTO LRFD* used the word "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."
- Any reference to *SE* load type should, in general, be considered a reference to foundation deformation, whether it is vertical deformation (settlement) or lateral deformation or rotation.

In Which Limit States Does *SE* Load Type Occur?

WP Figure 3-1

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	—	—	—	—	—	—	—	—	—	—	—	—

Superimposed Deformations – Article 3.4.1

"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……"

Superimposed Deformations – Article 3.4.1

“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.”

Superimposed Deformations – Article 3.4.1

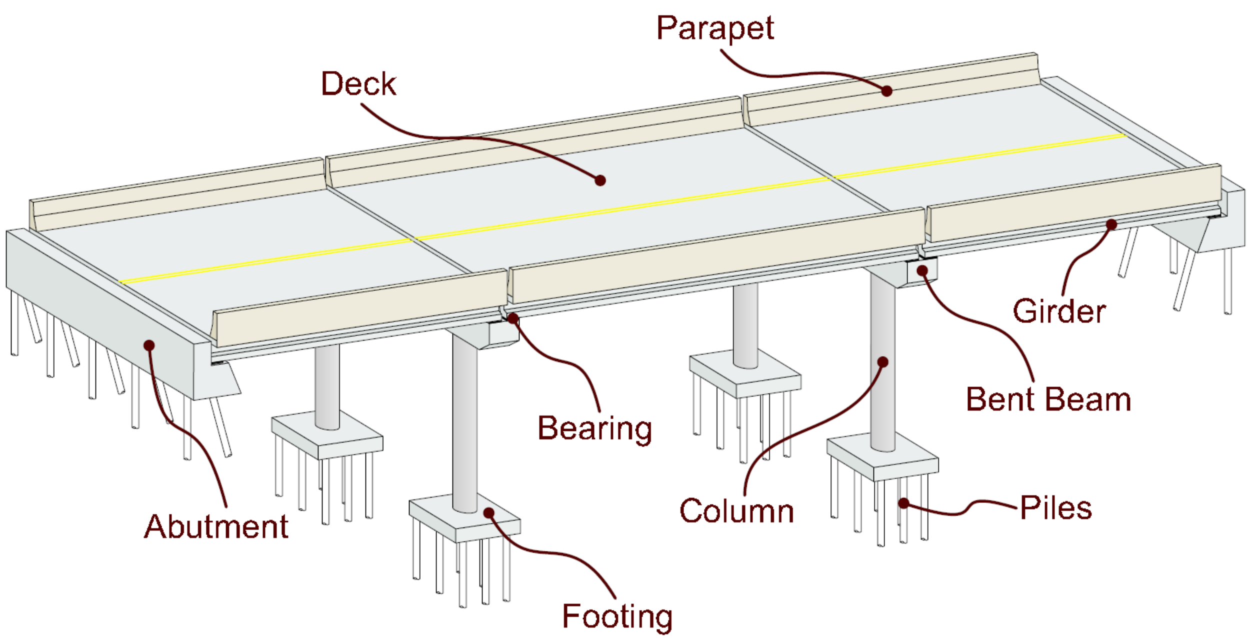
- **Article 3.4.1** of *AASHTO LRFD* states the following for selection of a value of $\boldsymbol{\gamma_{SE}}$:

  "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."

Superimposed Deformations – Article 3.12.6

Article 3.12.6 – Settlement

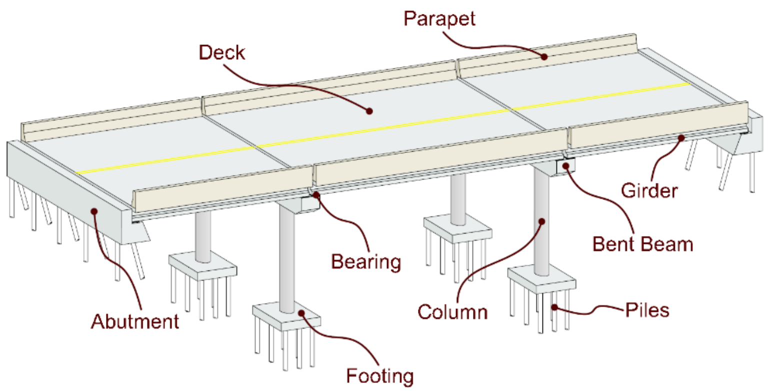
"Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered."



Superimposed Deformations – Article 3.12.6

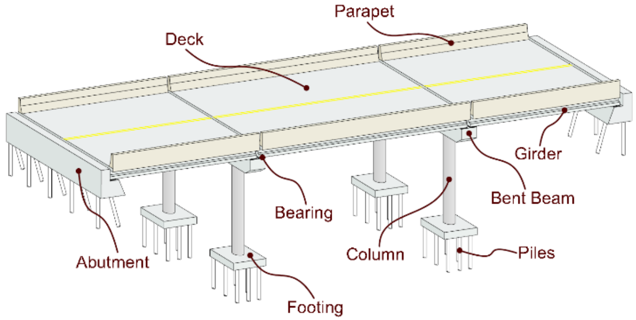
Commentary

"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."



Standard Specifications – 17th Edition (2002)

- **Article 3.3 – DEAD LOAD**
  3.3.2.1 "If differential settlement is anticipated in a structure, consideration should be given to stresses resulting from this settlement."



- Since the above stipulation is under the parent article **(3.3, Dead Load)**, it implies that settlement effects should be considered wherever dead load appears in the allowable stress design (ASD) or load factor design (LFD) load combinations.

Key Points

- Evaluation of differential deformation is mandated by AASHTO bridge design specification regardless of design platform (ASD, LFD, or LRFD).
  - It is not a new requirement.
- In LRFD platform,
  - Category of superimposed deformations
  - The γ_{SE} load factor appears in both strength and service limit state load combinations.
- The **uncertainty** of predicted deformations needs to be calibrated for the γ_{SE} load factor within the overall framework of limit state design.

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Foundation Deformations

Chapter 4 – Effect of Foundation Deformations on Bridge Structures and Uncertainty

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June 27, 2017

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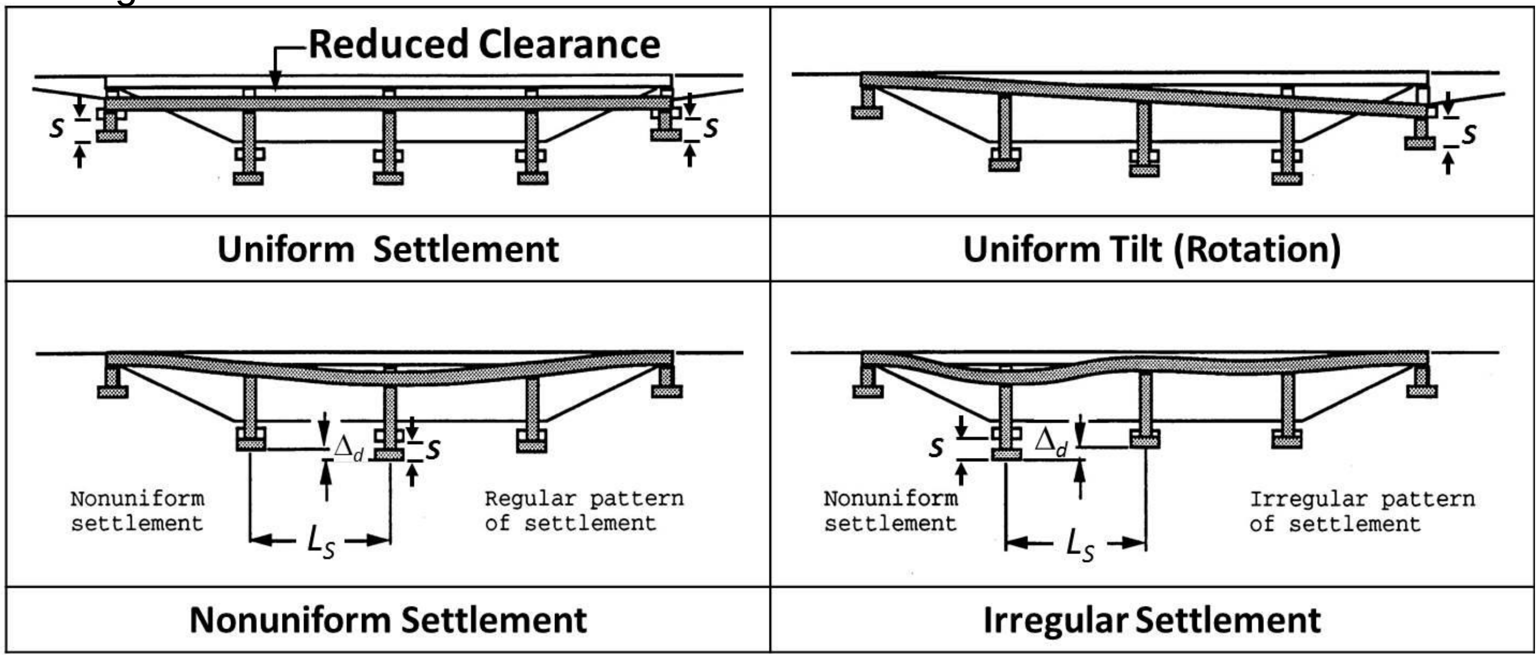
Learning Outcomes

- Define terminology to express settlement profile of a bridge structure

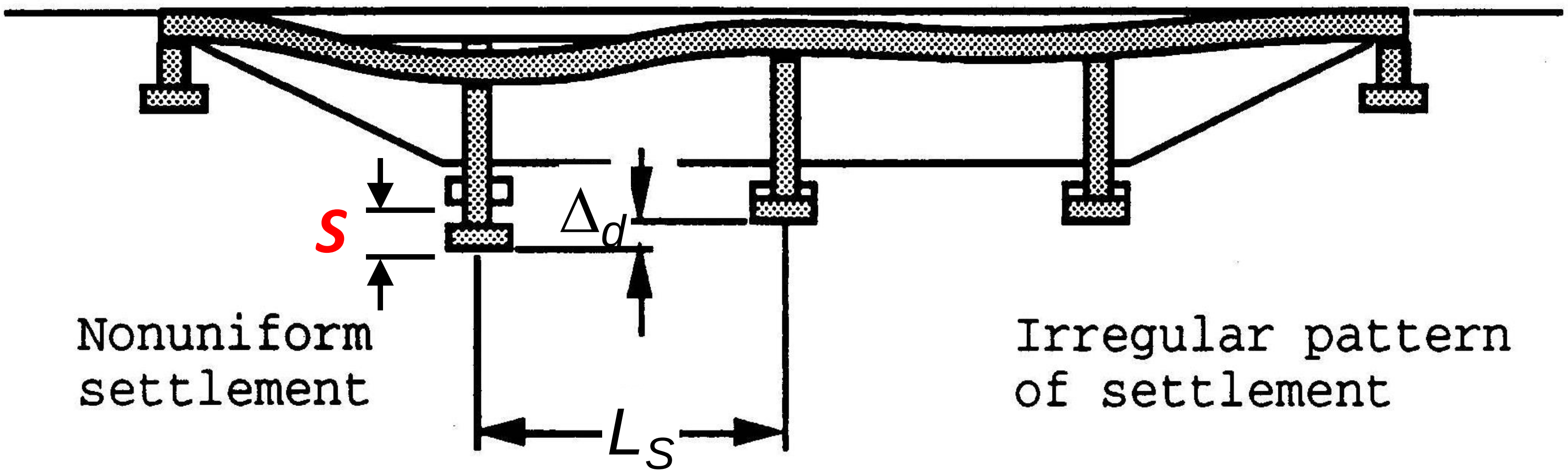
Idealized Vertical Deformation Patterns for Bridges

WP Figure 4-1



- *S*: Settlement at a foundation location
- L_S: Span length between adjacent bridge substructure elements
- Δ_d: Difference in settlement between two adjacent foundations
- Effect of foundation deformations
  - induce force effects within superstructure
  - affect approach features, rideability, deck drainage, etc.

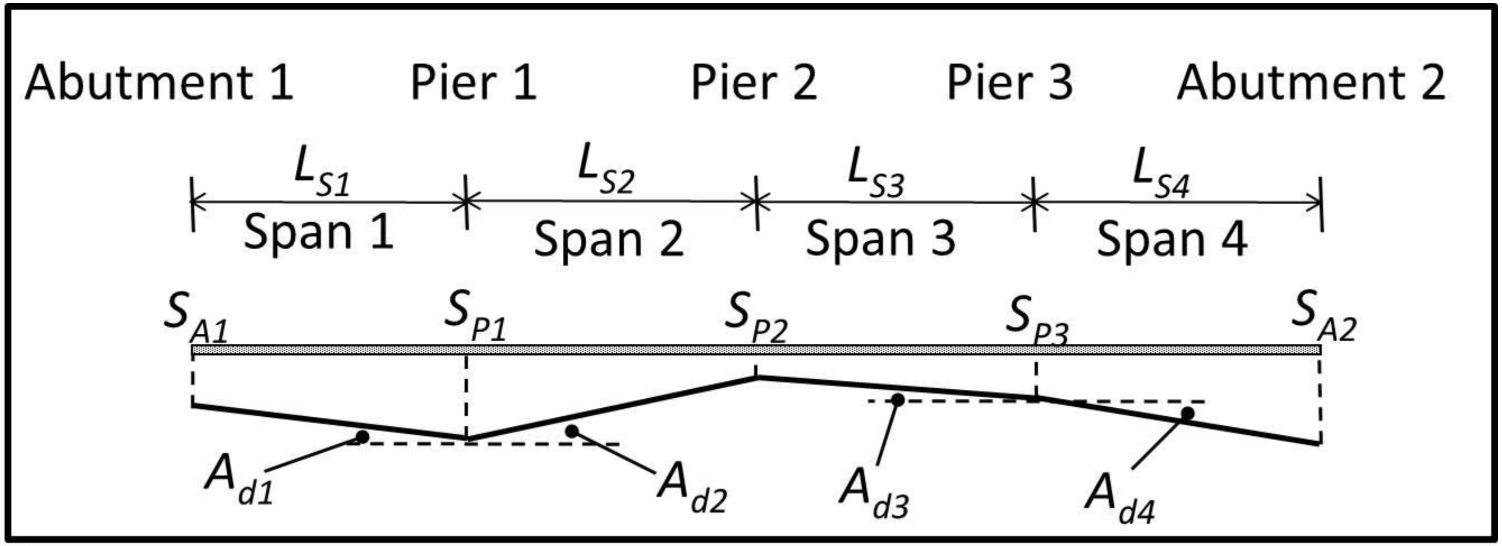
Differential Settlement, Δ_d



- Differential settlement, Δ_d, induces force effects within superstructure
- Differential settlement, Δ_d, when normalized by span length, L_S, is an expression of angular distortion

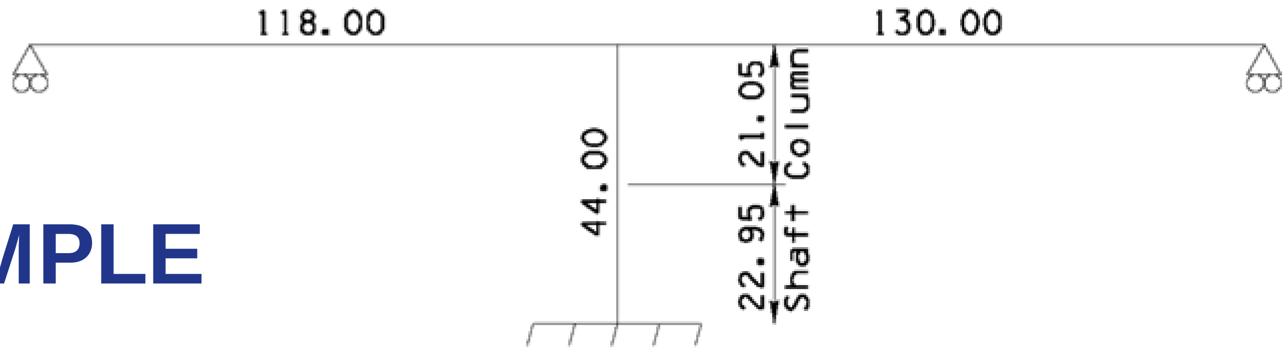
Concept of Differential Settlement and Angular Distortion

WP 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	$\Delta_{d4} = \lvert S_{P3} - S_{A2} \rvert$	$A_{d4} = (\lvert S_{P3} - S_{A2} \rvert)/L_{S4}$

Induced Moments in Continuous-Span Bridges



EXAMPLE

$$M_\Delta = \frac{6EI\Delta_d}{L_S^2} = 6\left(\frac{EI}{L_S}\right)\left(\frac{\Delta_d}{L_S}\right)$$ WP Equation 4-1, 4-2

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

EI/L_S is a representation of **Structure Stiffness**

Δ_d/L_S is **Angular Distortion** (dimensionless)

Damage Due to Differential Settlements

- Damage to bridge structure due to differential settlements can vary significantly depending on:
  - Type of superstructure
  - Connections between the superstructure and substructure units
  - Span lengths and widths
  - Continuity of superstructure with respect to substructure

- Because the induced force effect (e.g., moment) due to differential settlement is a direct function of EI/L_S, stiffness should be appropriate to the considered limit state.

Damage Due to Differential Settlements

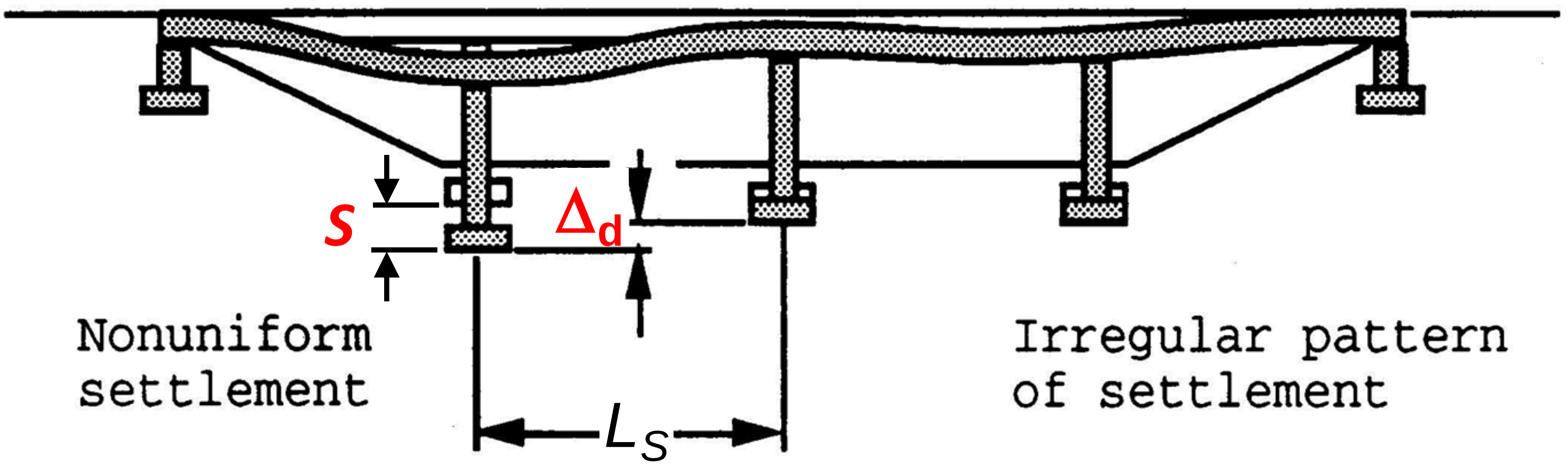
- For concrete bridges, the determination of stiffness of bridge components should consider the following effects:
  - Cracking
  - Creep
  - Inelastic responses

Damage Due to Differential Settlements

- To a lesser extent, differential settlements can also cause damage to a simple-span bridge.
  - Quality of riding surface
  - Adverse deck drainage
  - Aesthetics

- Because of 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.

Settlement, *S*, and Angular Distortion, $A_d = \Delta_d / L_S$



- What is a ***tolerable*** value of Δ_d / L_S ?
- How ***reliable*** is the value of *S* ?

Key Points

- Differential settlement induces force effects in the superstructure

- Damage to a bridge structure is a function of angular distortion and structure stiffness

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Foundation Deformations

Lecture 7

White Paper Chapter 5 to Chapter 8

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Foundation Deformations

Chapter 5 – Tolerable Foundation Deformation Criteria

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June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

- Identify limiting (tolerable) angular distortion values from *AASHTO LRFD*

- Discuss arbitrary use of limiting (tolerable) angular distortion values by different agencies

Tolerable Movement Criteria for Highway Bridges

WP Table 5-1

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

- Based on *AASHTO LRFD* Article C10.5.2
- Tolerable → Limiting
- Movement is expressed in terms of angular distortion
- What is the history of these criteria?

Limiting (Tolerable) Angular Distortion

- Moulton et al. (1985) – For FHWA
- AASHTO – Standard (ASD) and LRFD Specifications

Type of Bridge	Limiting Angular Distortion, Δ_d/L_S	
	Moulton et al. (1985)	**AASHTO**
Continuous Span	**0.004** (4.8" in 100')	**0.004** (4.8" in 100')
Simple Span	**0.005** (6.0" in 100')	**0.008** (9.6" in 100')
For rigid frames, perform case-specific analysis		

Moulton's Evaluation

- 314 bridges in US and Canada
- Steel, concrete, and concrete/steel structures
- Variety of foundations, spans, and span lengths
- Field evaluation
- Analytical evaluation

- Tolerance to movements was often judged qualitatively by responding agencies in accord with TRB definition

Definition of Intolerable Movement in Moulton's Study

- Per TRB Committee A2K03 (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.***"

Arbitrary Use of AASHTO Limiting Values

Arbitrary (no consistency in application)

- 0.004 → 0.0004 or 0.008 → 0.0008
- I-25/I-40 TI (BIG-I), NM: 0.004 → 0.002, 0.008 → 0.004
- WSDOT (From Chapter 8 of Geotech Design Manual)

WP Table 5-2

Total Settlement, δ, at Pier or Abutment	Differential Settlement over 100 ft within Pier or Abutments and Differential Settlement Between Piers [Implied Limiting Angular Distortion, radians]	Action
$\delta \leq 1"$	$\Delta_{d100'} \leq 0.75"$ **[0.000625]**	Design & construct
$1" < \delta \leq 4"$	$0.75" < \Delta_{d100'} \leq 3"$ **[0.000625-0.0025]**	Ensure structure can tolerate settlement
$\delta > 4"$	$\Delta_{d100'} > 3"$ **[> 0.0025]**	Need Dept approval

Another Example from a DOT

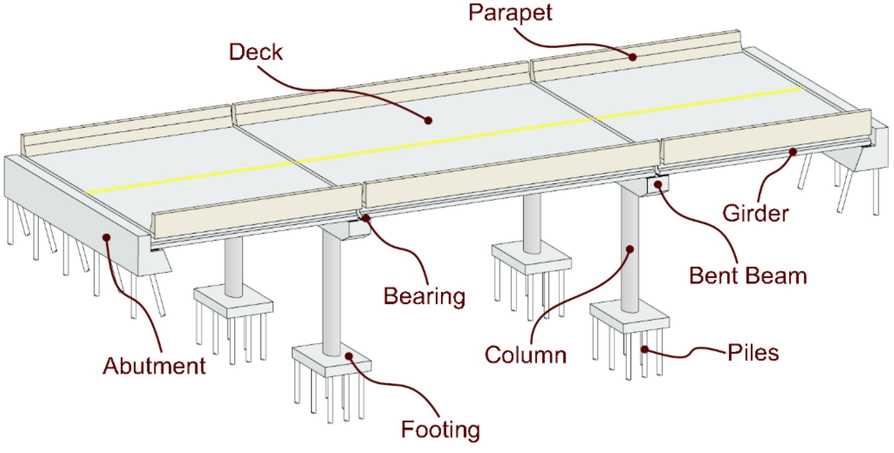
- *Chapter 10 of Bridge Design Guidelines of the Arizona Department of Transportation (ADOT, 2015) states the following:*

  "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."

Selection of Tolerable Deformation

A 3-step process

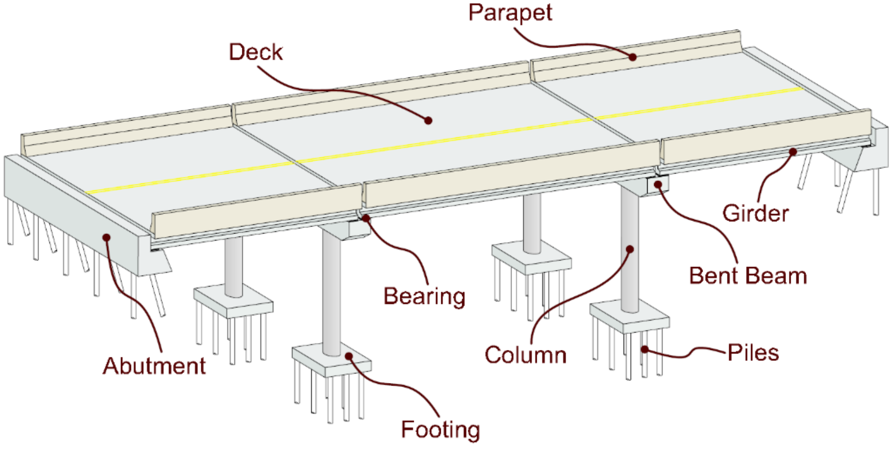
- Based on consideration of all elements associated with a bridge and approach structures
  – Superstructure elements, substructure elements, approach elements, joints, utilities, clearances, etc.



Selection of Tolerable Deformation

Step 1

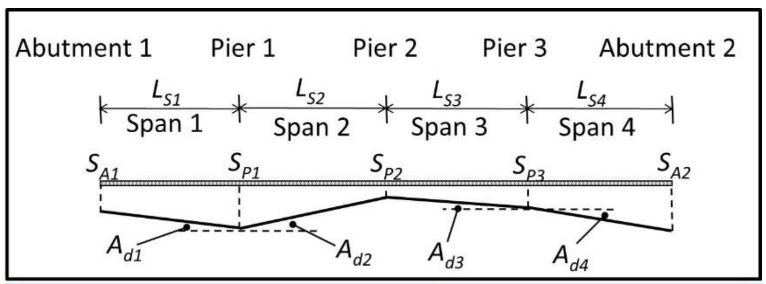
- Identify all possible facilities associated with the bridge structure and the movement tolerance of those facilities
- Examples: deck, parapet, joints, attached utilities, etc.



Selection of Tolerable Deformation

Step 2

- Determine the differential settlement profile along the bridge by using conservative assumptions for geomaterial properties and prediction methods
- Estimate the angular distortion based on construction-point concept



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	$\Delta_{d4} = \lvert S_{P3} - S_{A2} \rvert$	$A_{d4} = (\lvert S_{P3} - S_{A2} \rvert)/L_{S4}$

WP Figure 4-2

Selection of Tolerable Deformation

- **Step 3**
  - Compare the angular distortion from *Step 2* with the various tolerances identified in *Step 1* and AASHTO's limiting angular distortion values
  - Identify the critical component of the facility
  - Review this critical component to check if 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 construction may help mitigate the issues related to intolerable deformations

Tolerable Horizontal Deformation Criteria

- Horizontal deformations cause more severe and widespread problems for highway bridge structures than equal magnitudes of vertical movement
- Tolerances to horizontal movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects
- Moulton's findings for horizontal movements:
  - < 1": tolerable
  - > 2": intolerable
  - Recommended: 1.5"
  - Horizontal movements result in more damage when accompanied by settlement than when occurring alone

Evaluation by Moulton et al. (1985)

Basis

- 1977 – 12th Edition of Standard Specifications
- HS20-44 wheel loading or its equivalent lane loading

Key observation of 1985 study

- Attempts to establish tolerable movements from analyses of the effects of differential settlement on the stresses in bridges significantly underestimated the criteria established from field observations
- Analytical evaluation leads to overly conservative angular distortion criteria

Evaluation by Moulton et al. (1985)

Reasons for Conservatism

- Discrepancy between analytical studies and field observations is because the analytical studies often do not account for the construction time of the structure and the construction-point concept (next topic)

- Building materials like concrete (especially concrete while it is curing) are able to undergo a considerable amount of stress relaxation when subjected to deformations
  - Under conditions of very slowly imposed deformations, the effective value of the Young's modulus of concrete is considerably lower than the value for rapid loading

Key Points

- ***AASHTO LRFD*** specifies limiting angular distortion criteria
- Agencies often use arbitrary criteria for angular distortion, which may not be rational

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Foundation Deformations

Chapter 6 – Construction-Point Concept

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June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

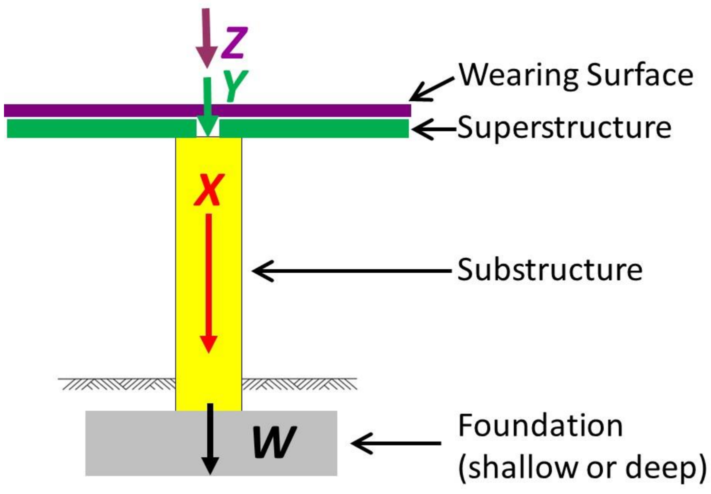
- Introduce and understand the construction-point concept

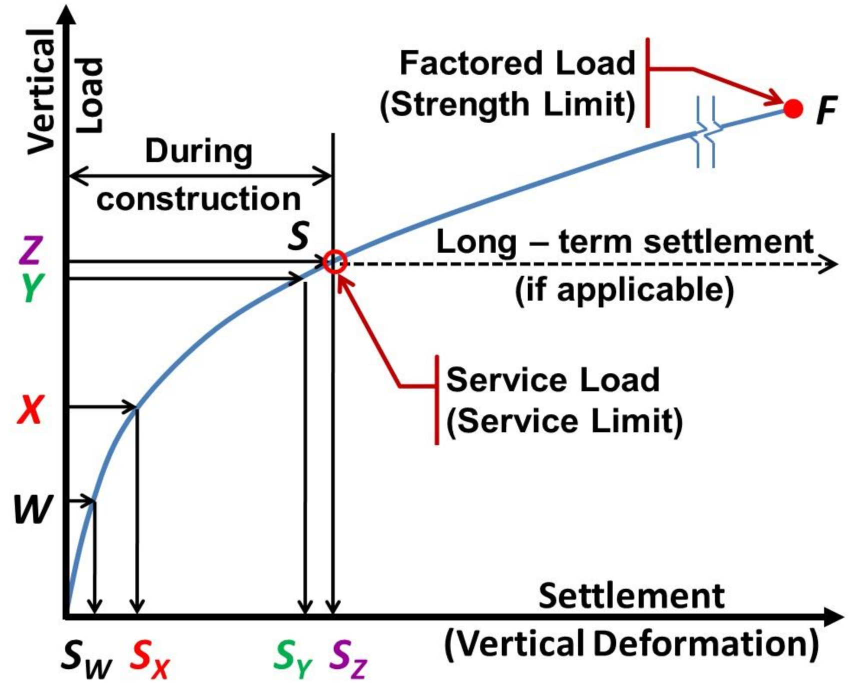
When is a Bridge Structure Affected?

Construction-Point Concept

Example: Bridge Pier

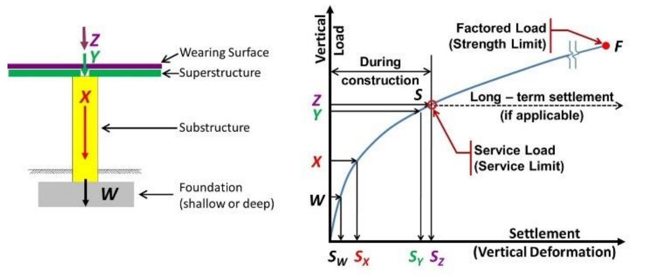
WP Figure 6-1 (a) and (b)

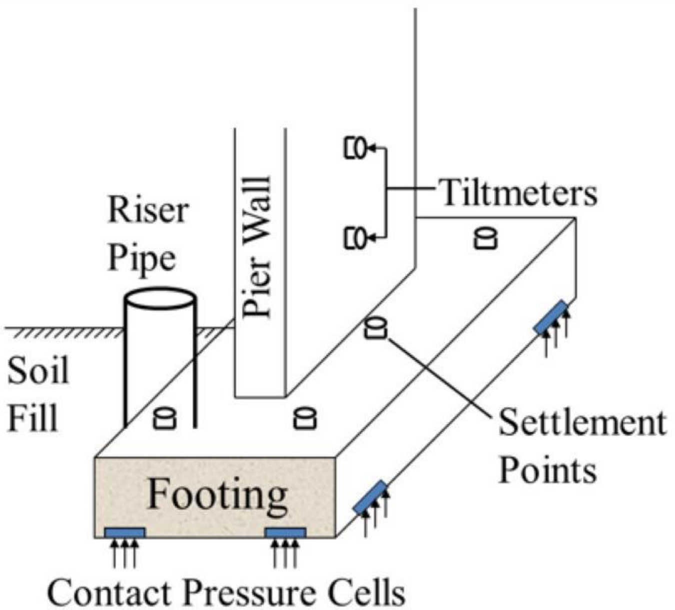




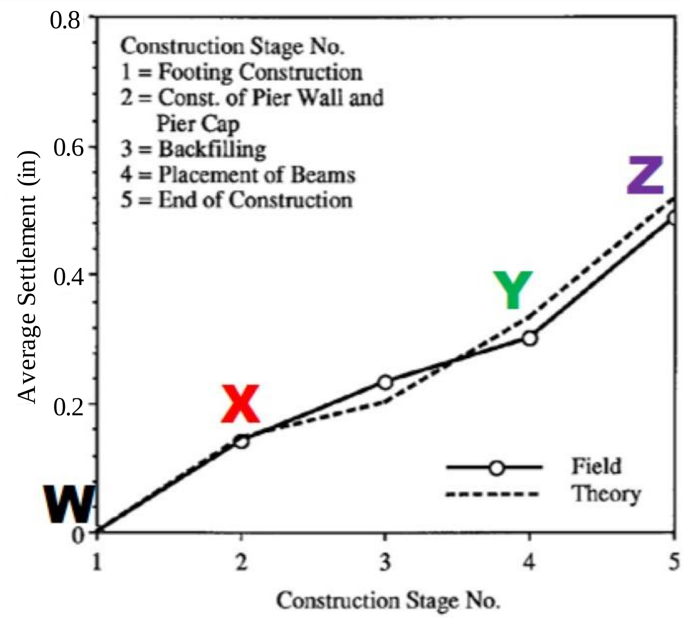
When is a Bridge Structure Affected?

Construction-Point Concept





Instrumentation



Example Data (Pier)

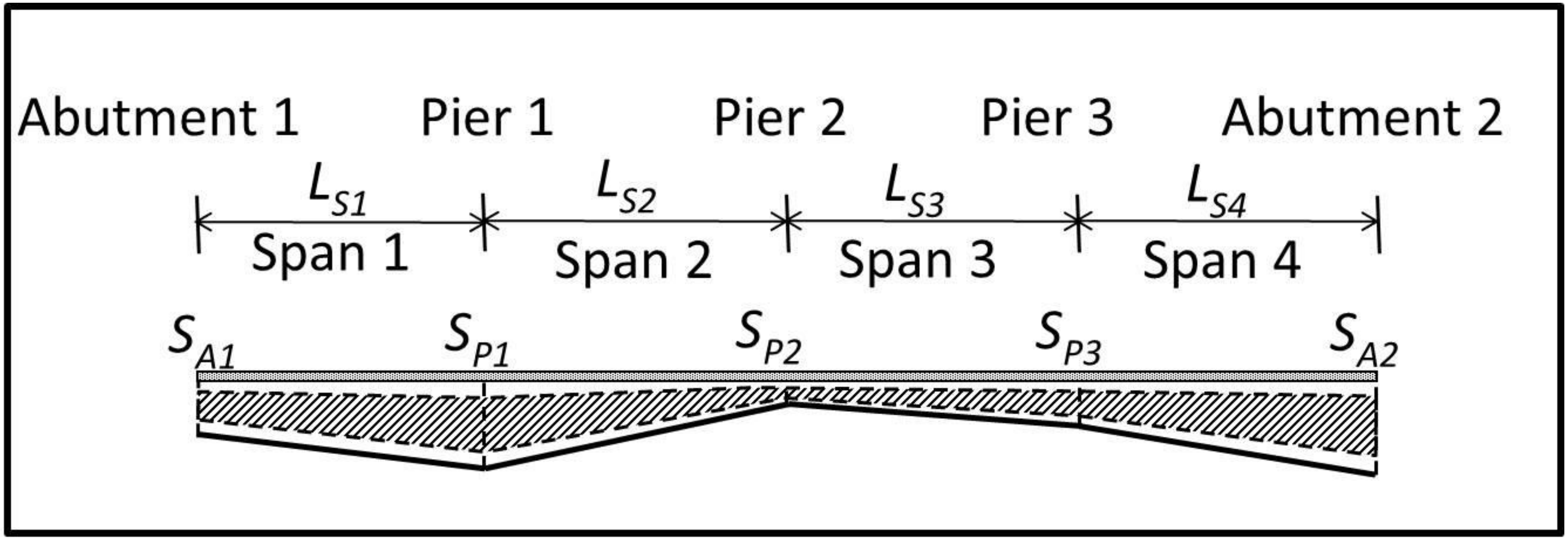
Reference: Sargand, et al., (1999); Ohio DOT

General Observations

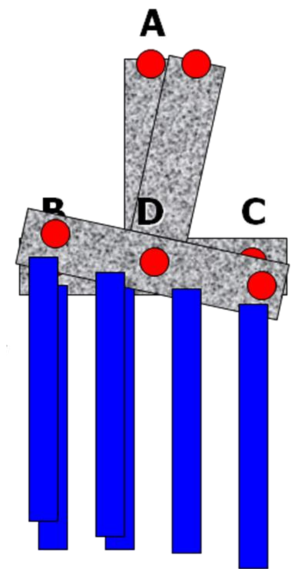
- The percentage of settlement between placement of beams and end-of-construction is generally between 25 to 75 percent of the total settlement
  - This observation applies to all other deformations, e.g., lateral and rotation
- Construction-point concept is applicable to immediate deformations
  - Evaluation of total settlement and maximum (design) angular distortion must also account for long-term settlement
  - Continued long-term deformation of the structure after end-of-construction may not be acceptable, e.g., reduced clearance under a bridge

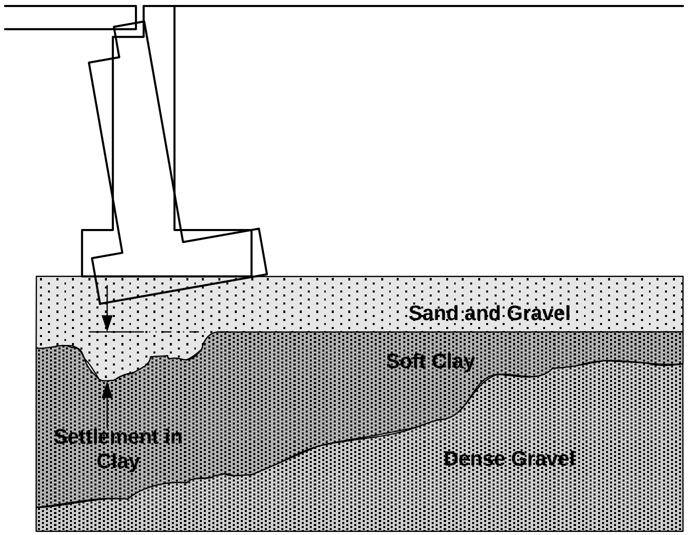
Relevant Angular Distortion in Bridges

WP Figure 6-2



Horizontal Deformations





- The limiting horizontal movements are strongly dependent on the type of superstructure, and the connection with the substructure
  - Acceptable values of horizontal deformations are project specific

Key Points

- Use of total foundation deformations based on assumption that all loads are applied instantaneously is not realistic

- The percentage of settlement between placement of beams and end-of-construction is generally between 25 to 75 percent of the total settlement

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Foundation Deformations

Chapter 7 – Reliability of Predicted Foundation Deformations

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Federal Highway Administration

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Learning Outcomes

- Discuss reliability of predicted foundation deformations

Reliability of Predicted Foundation Deformations

- All analytical methods (models) for predicting foundation deformations have some degree of uncertainty

- The reliability of predicted foundation deformations varies as a function of the chosen analytical method

- Since the induced force effects (for example, moments) are a direct function of foundation deformations, the values of the induced force effects are only as reliable as the estimates of the foundation deformations

Reliability of Predicted Foundation Deformations

- It is important to quantify the uncertainty in foundation deformations by calibrating the analytical method used to predict the foundation deformations using stochastic procedures

- In the LRFD framework, the uncertainty is calibrated through use of load and/or resistance factors

- *AASHTO LRFD* considers uncertainty of foundation deformations in terms of the induced effects through the use of γ_{SE} load factor

What Does All of This Mean?

Need to:

1. **Re-evaluate** past data in LRFD framework
2. **Re-survey** using revised definition of intolerable movements in LRFD context
3. Using reliability considerations, **evaluate** foundation/soil response with substructure/superstructure interaction
4. Calibrate the γ_{SE} load factor

Key Points

- It is important to understand and quantify the uncertainty in predicted foundation deformations




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Foundation Deformations

Chapter 8 – Calibration Procedures

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June 27, 2017

U.S. Department of Transportation
Federal Highway Administration

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO

Learning Outcomes

- Identify overarching characteristics that apply to service limit states

- Discuss the incorporation of load-deformation behavior into calibration of service limit state for foundation deformations

Relevant *AASHTO LRFD* Articles

WP Table 8-1

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 AASHTO LRFD for estimating horizontal (lateral) deformations of deep foundations. Use of Strain Wedge Method (SWM) is allowed per C10.7.2.4.
Note: Section 11 (Abutments, Piers and Walls), Article 11.6.2 of AASHTO LRFD refers back to the various Articles noted in the left column of this table. Therefore, the Articles noted in this table also apply to fill retaining walls and their foundati ons.	

Overarching Characteristics to Be Considered

1. Load-driven versus non-load-driven limit states

2. Reversible versus irreversible limit states

3. Consequences of exceeding deformation-related limit states and target reliability indices

4. Calculation models

Overarching Characteristics to be Considered:

1. Load-Driven versus Non-Load-Driven Limit States

Load-Driven versus Non-Load-Driven Limit States

- 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

Load-Driven Limit States

- Damage occurs because of accumulated applications of external loads, usually live load (trucks)

- Examples:
  - Decompression and cracking of prestressed concrete
  - Vibrations
  - Deflection

- Damage caused by exceeding these limit states may be reversible or irreversible and hence the cost of repair may vary significantly

Non-Load-Driven Limit States

- Damage occurs because of deterioration or degradation over time and aggressive environment or as inherent behavior from certain material properties

- Examples:
  - Penetration of chlorides leading to corrosion of reinforcement
  - Leaking joints leading to corrosion under the joints
  - Shrinkage cracking of concrete components
  - Corrosion and degradations of reinforcements in reinforced soil structures (e.g., MSE walls)

- In these limit states, the external load occurrence plays a secondary role

Load-Driven versus Non-Load-Driven Limit States

- In the case of foundation deformations, computations are usually performed as follows:
  - Consider live load (load-driven) for short-term deformations
  - Do not consider live load for long-term or time-dependent deformations

Overarching Characteristics to be Considered:

2. Reversible versus Non-Reversible Limit States

Reversible versus Irreversible Limit States

- Reversible limit states are those for which no consequences remain once a load is removed from the structure

- Irreversible limit states are those for which consequences remain once a load is removed from the structure

- Foundation deformation may be considered as a irreversible limit state with respect to foundation elements

Concept of Reversible-Irreversible Limit States

- Reversible-irreversible limit state is one where the effect of an irreversible limit state may be reversed by intervention

- Example: Foundation deformation, which is an irreversible limit state with respect to foundation elements but may be reversible in terms of its effect on the bridge superstructure through intervention, e.g., through use of shims or jacking

FHWA (2006)

Reversible versus Irreversible Limit States

- Because of their reduced service implications, irreversible limit states, which do not concern the safety of 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 compared to irreversible limit states

Overarching Characteristics to be Considered:

3. Consequences of Exceeding Deformation-Related Limit States and Target Reliability Indices

Consequences and Target Reliability Indices

- Factors to be considered while differentiating between different limit states in terms of consequences:
  - Irreversible versus reversible limit states
    - Irreversible limit states may have higher target reliability than reversible limit states
    - Reversible-irreversible limit states may have target reliability similar to reversible limit states
  - Relative cost of repairs
    - Limit state that have the potential to cause damage that is costly to repair may have a higher target reliability than limit states that have the potential of causing only minor damage

Consequences and Target Reliability Indices

- Strength (or ultimate) limit states pertain to structural safety and the loss of load-carrying capacity
  - Consequences of collapse can be severe.
  - Reliability indices for strength limit states range from 3.0 to 3.5 for bridge structures and 2.3 to 3.5 for geotechnical features

- 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
  - Since 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

Consequences and Target Reliability Indices

- Foundation deformations induce secondary force effects in a bridge structure (e.g., increased moment or potential cracking)

- Force effect due to settlement, relative to the forces effect due to dead and live loads, would generally be small
  - Load factor, γ_{SE}, is only one of the many load factors in all the Service and Strength limit state load combinations
  - The primary moments due to the sum of dead and live loads are much larger than the additional (secondary) moments due to settlement

Target Reliability Index for Structural Service Limit States

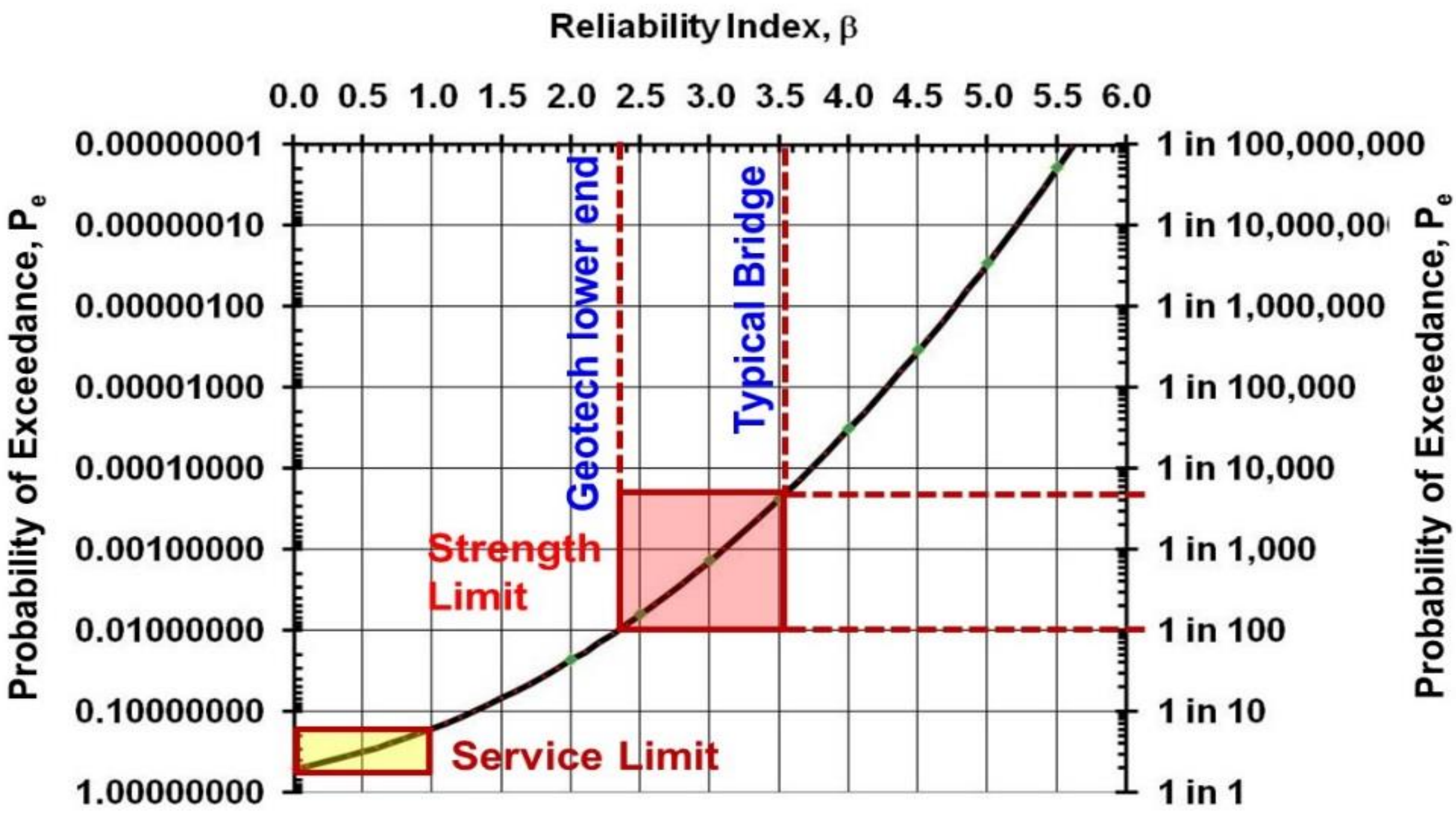
WP Table 9-11

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 reliability index can be used as per R19B, the subcommittees have expressed a desire not to change the values implied by the current standard.		

Consequences and Target Reliability Indices

- Based on various considerations noted in previous slides and consideration of reversible and irreversible service limit states for bridge superstructures, a target reliability index, β_T, in the range of 0.50 to 1.00 for calibration of load factor, γ_{SE}, for foundation deformation in the Service I limit state is recommended by Project R19B

Consequences and Target Reliability Indices



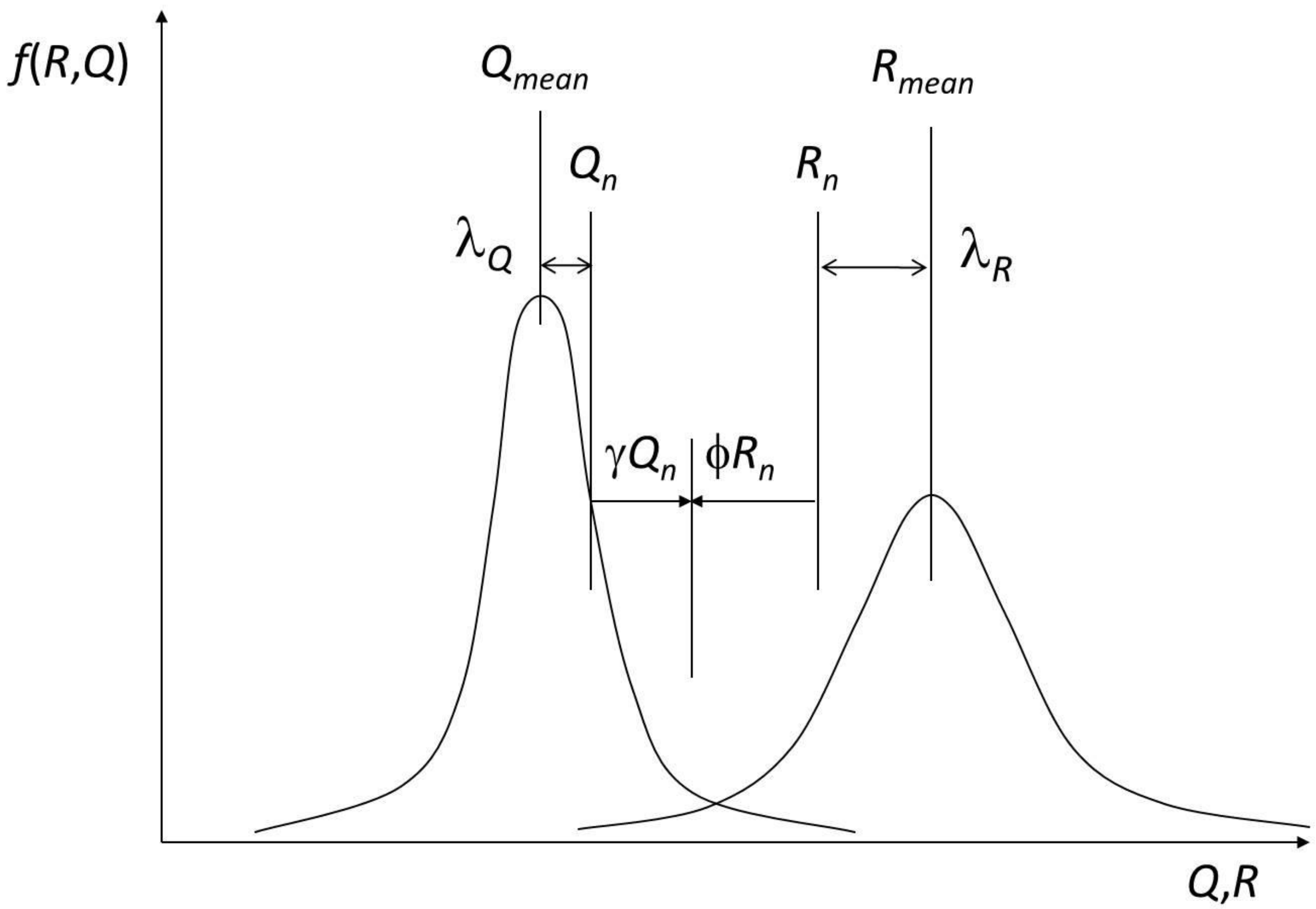
Note: Plot is based on the assumption that the load and the resistance are normally distributed and statistically independent (i.e., uncorrelated)

Overarching Characteristics to be Considered:

4. Calculation Models

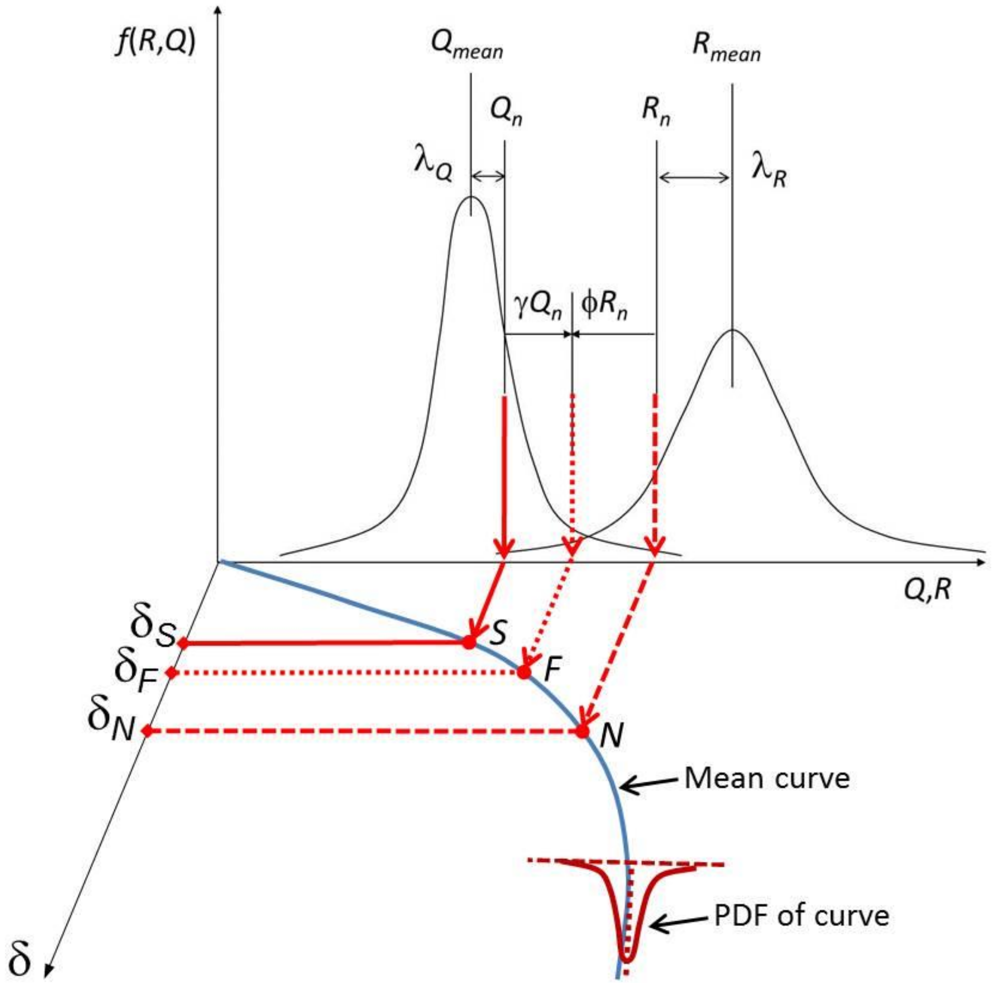
Basic LRFD Concept

WP Figure 8-1



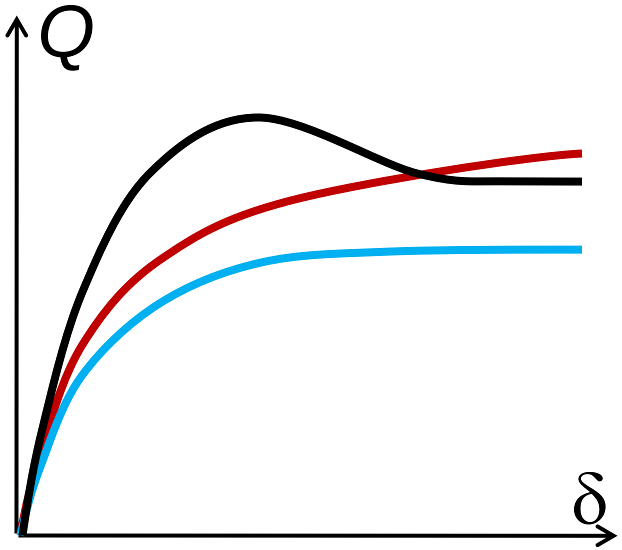
The Q-δ Dimension

WP Figure 8-2



Q-δ Model

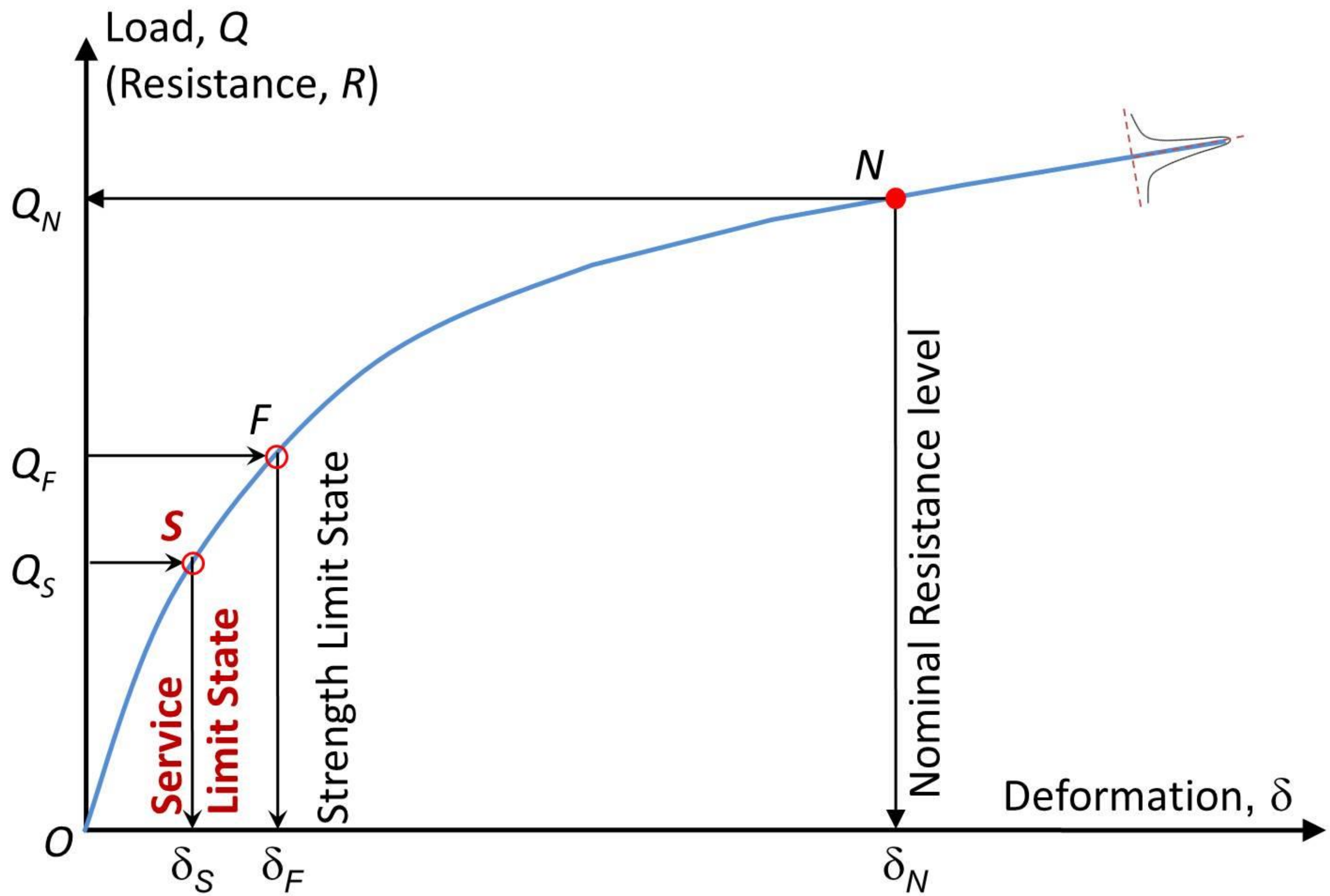
- *Q* is force effect such as applied load, induced stress, moment, shear, etc.
  - Could be expressed as resistance, *R*
- δ is deformation such as settlement, rotation, strain, curvature, etc.
- Q-δ *curves can have many shapes*
  - Only 3 shapes are shown in the figure as examples



- **Formulation is general and applies to both geotechnical and structural aspects.** Some examples are as follows:
  - Lateral load – lateral displacement (*P*-*y*) curves
  - Moment-curvature (*M*-ϕ) curves
  - Shear force-shear strain curves

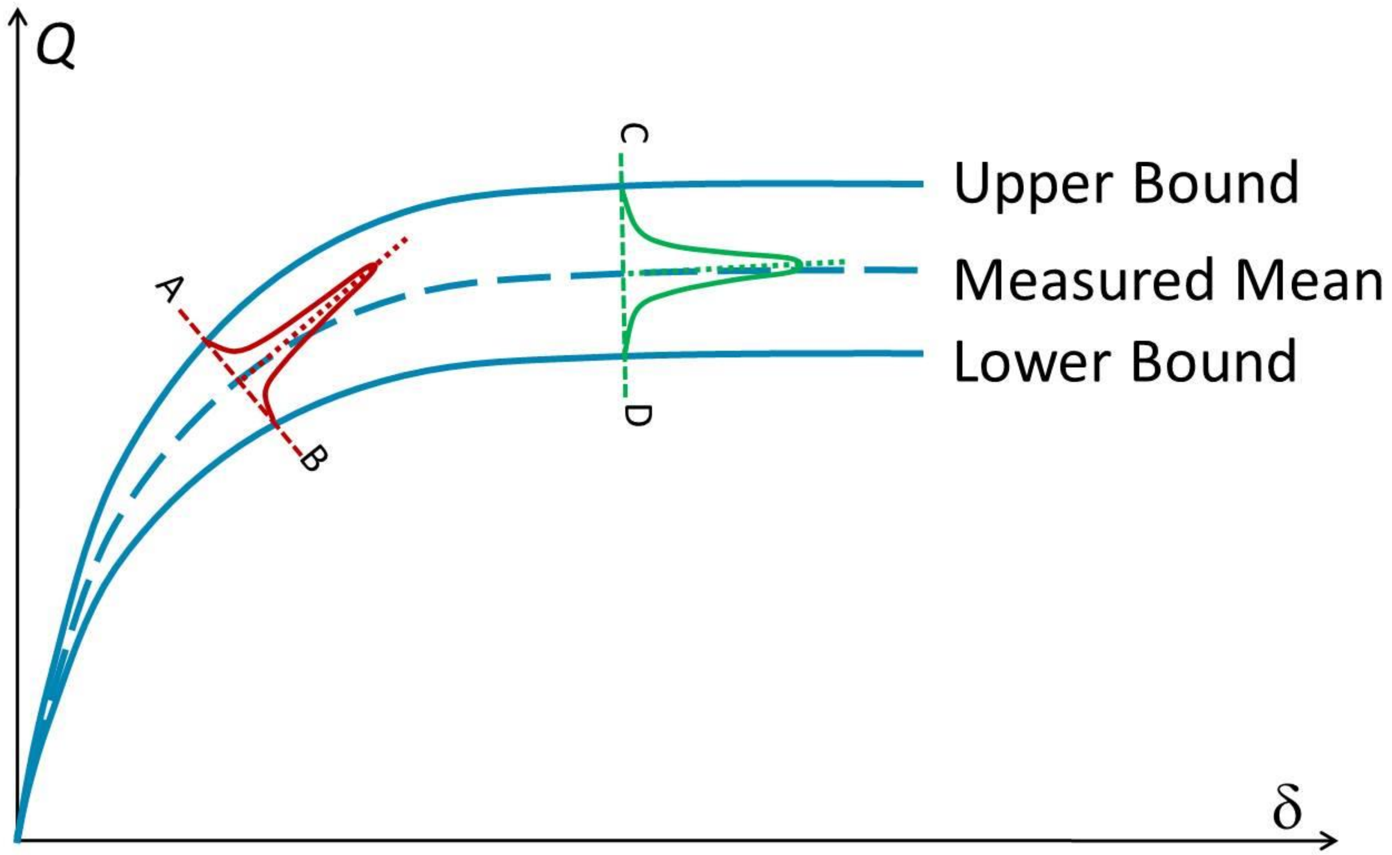
Q-δ Model and Limit States

WP Figure 8-3



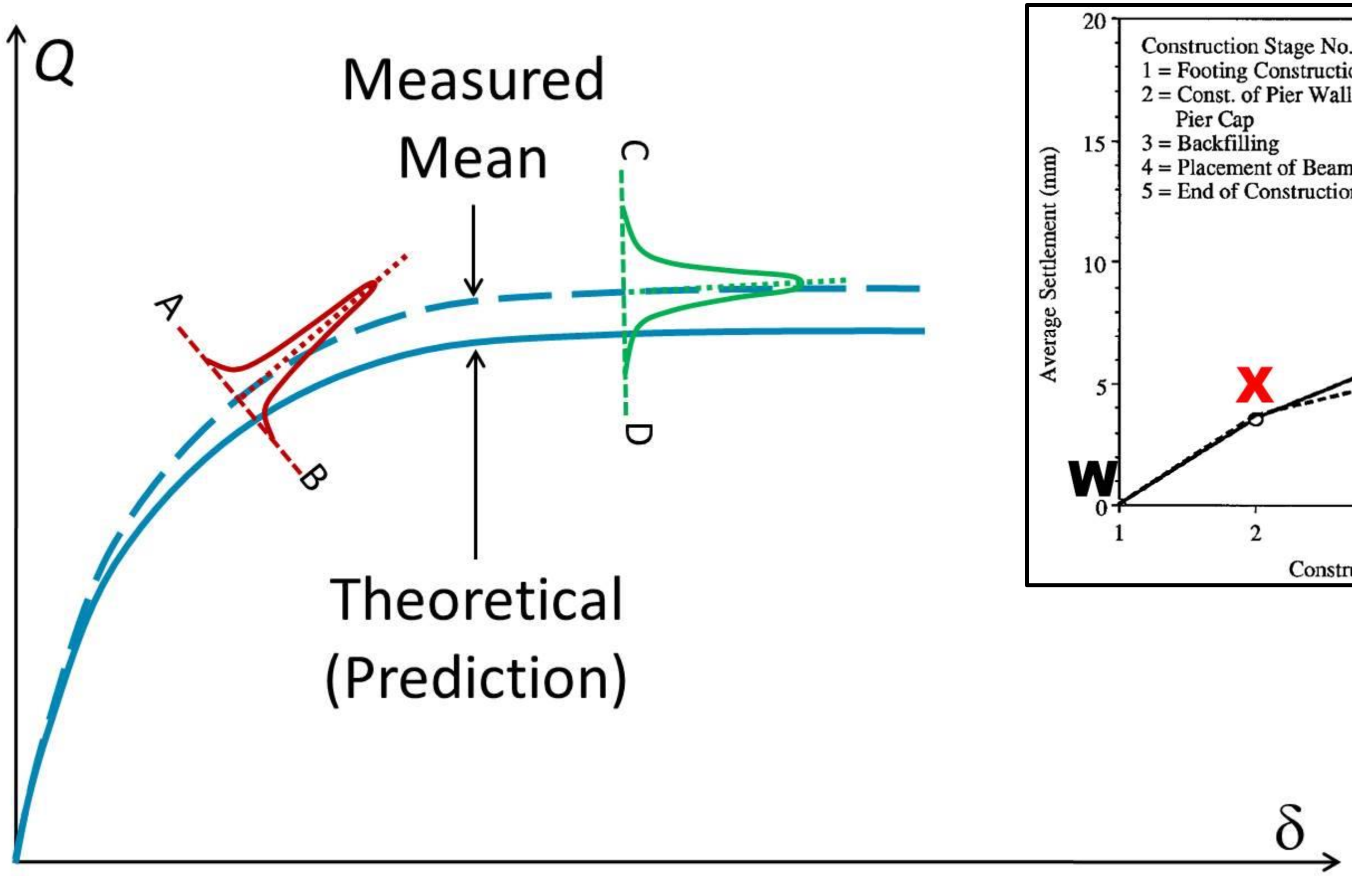
Range and Distribution of Q-δ

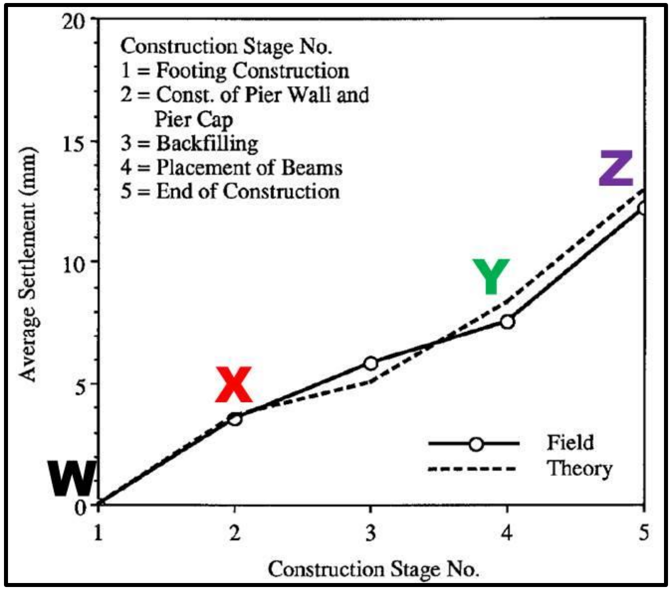
WP Figure 8-4



Correlation of Measured Mean With Theoretical Prediction

WP Figure 8-5





Serviceability Limit State(s)

- For strength limit state, common expression is $g = R - Q$

- For service limit state, the expression can be:

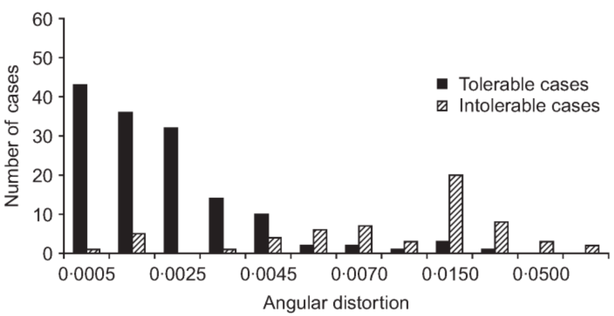
$$g = \delta_T - \delta_P \quad \begin{cases} \delta_T = \text{target (design or tolerable)} \\ \delta_P = \text{predicted (estimated)} \end{cases}$$

- δ_T is Resistance and δ_P is Load

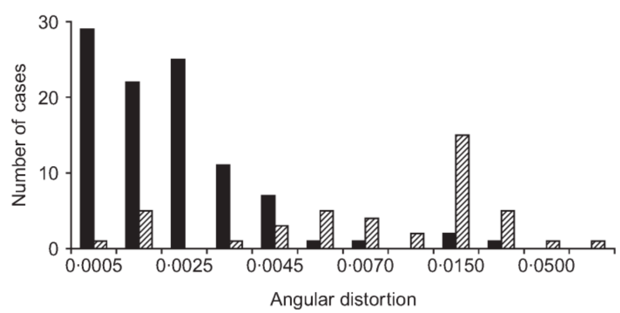
- Need statistics for δ_T and δ_P

Data from Moulton et al. (1985)

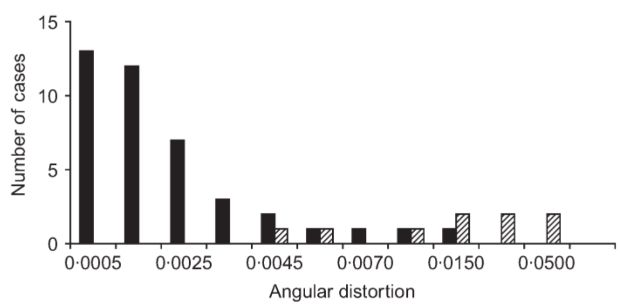
Angular distortion interval	All bridges*		Steel bridges		Concrete bridges	
	Tolerable cases	Intolerable cases	Tolerable cases	Intolerable cases	Tolerable cases	Intolerable cases
0–0·001	43	1	29	1	13	0
0·0011–0·002	36	5	22	5	12	0
0·0021–0·003	32	0	25	0	7	0
0·0031–0·004	14	1	11	1	3	0
0·0041–0·005	10	4	7	3	2	1
0·0051–0·006	2	6	1	5	1	1
0·0061–0·008	2	7	1	4	1	0
0·0081–0·010	1	3	0	2	1	1
0·011–0·020	3	20	2	15	1	2
0·021–0·040	1	8	1	5	0	2
0·041–0·060	0	3	0	1	0	2
0·061–0·080	0	2	0	1	0	0
	144	60	99	43	41	9



All Bridges



Steel Bridges



Concrete Bridges

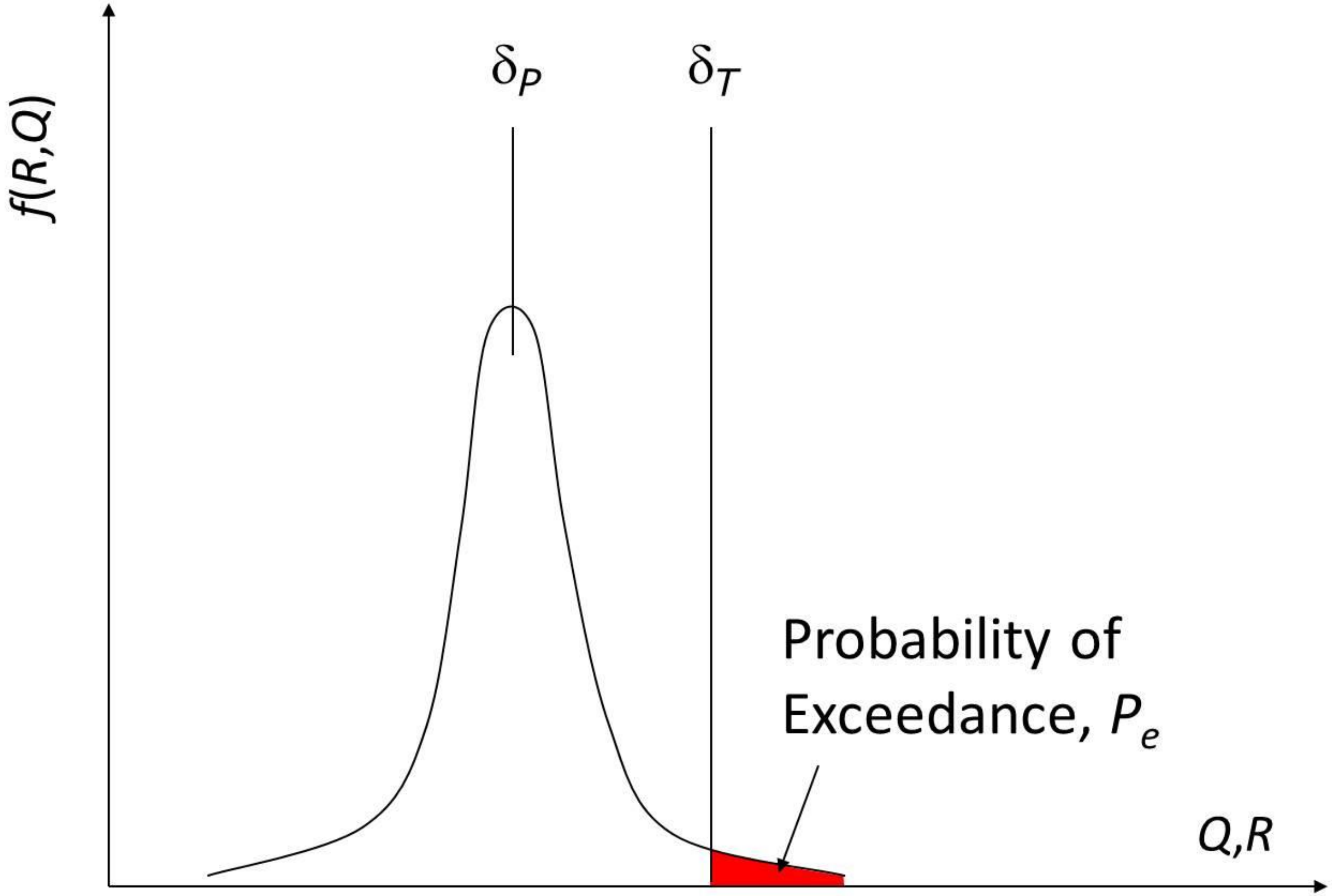
Reference: Zhang and Ng (2005)

Statistics for δ_T (Resistance)

- No consensus on δ_T
- No standard deviation (σ), Bias (or Accuracy) data available at this time using LRFD specifications
  - Long Term Bridge Performance Program (LTBPP) may offer future data
- Use of deterministic value of δ_T by bridge designer
  - Varies based on type of bridge structure, joints, design of specific component, ride quality, deck drainage, aesthetics, public perception, etc.

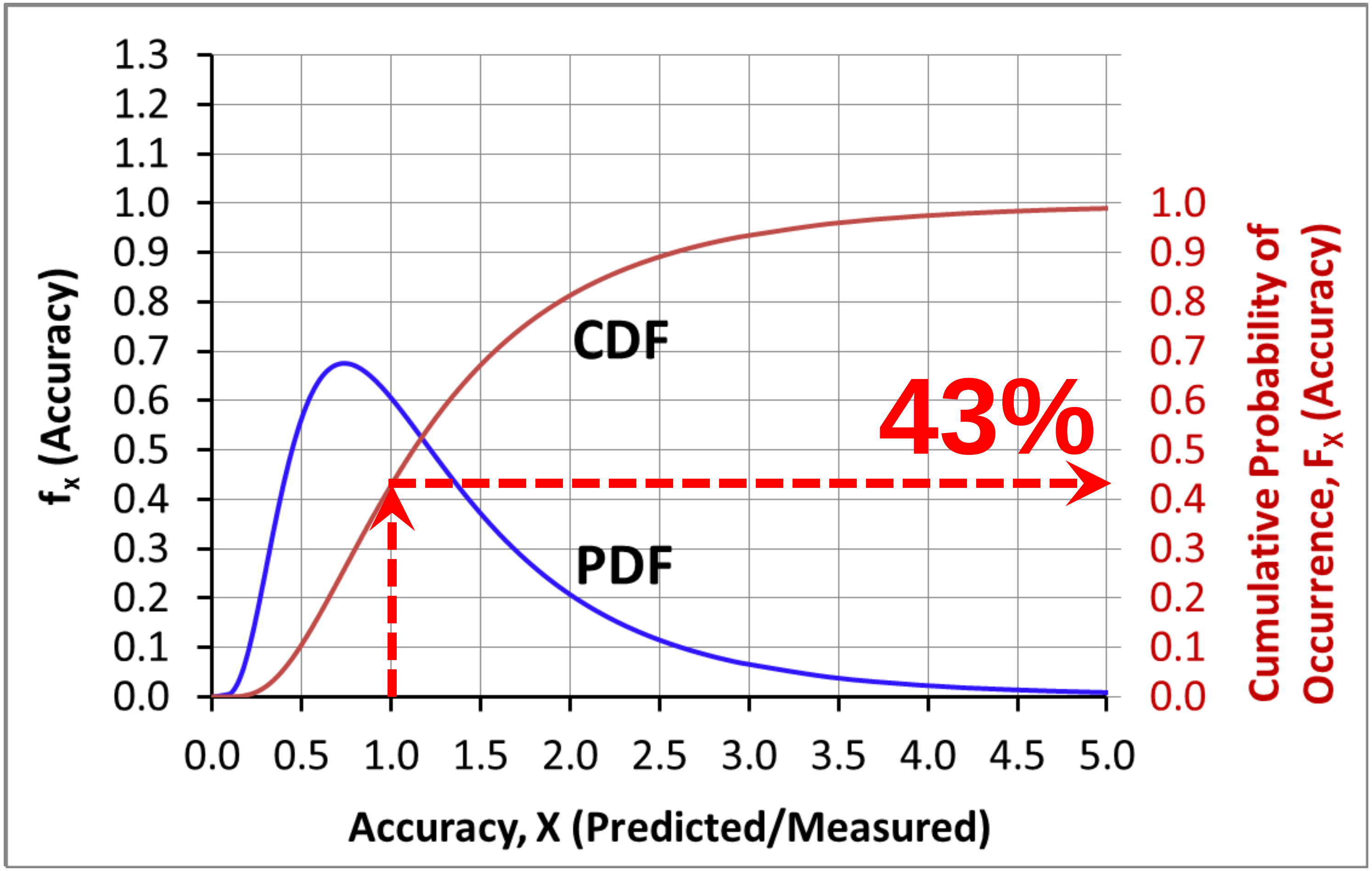
Adaptations

WP Figure 8-6



Convert PDF to CDF

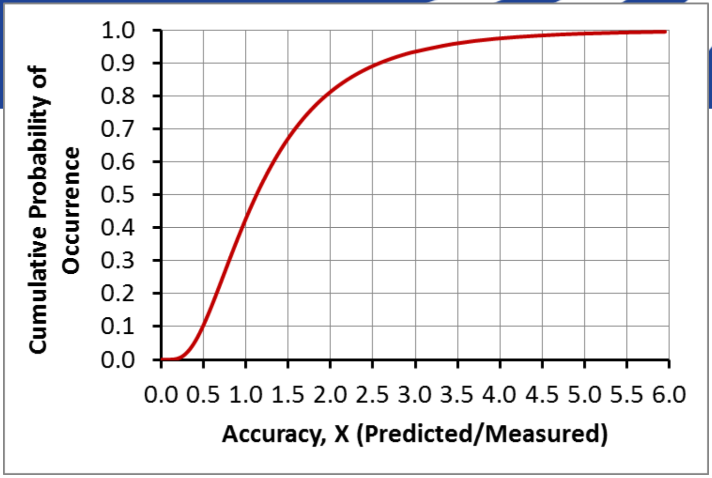
Example

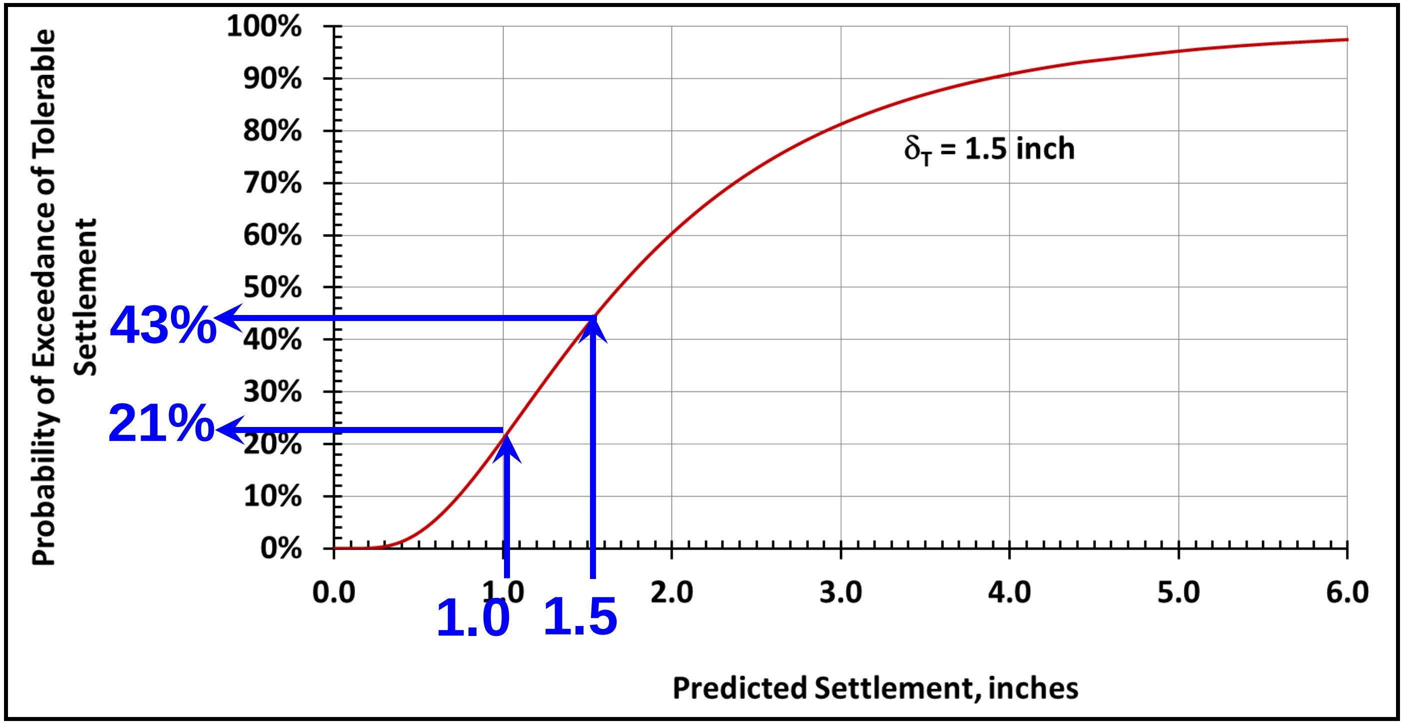


PDF: Probability Distribution Function; CDF: Cumulative Distribution Function

Generate Probability Exceedance Chart (PEC) from CDF

Example

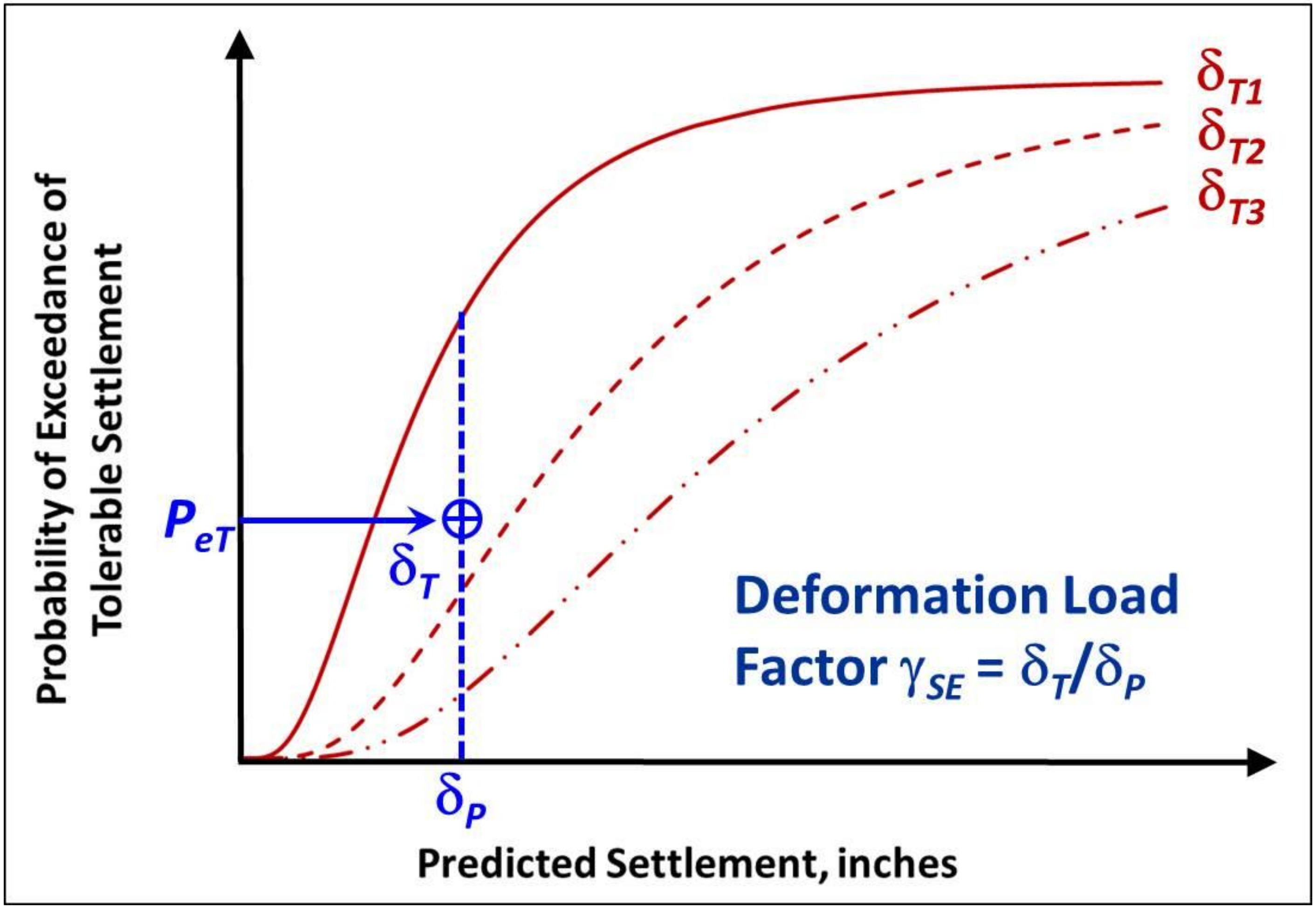




Determination of Load Factor for Deformations γ_{SE}

WP Figure 8-7

$\delta_{T1} < \delta_{T2} < \delta_{T3}$



Development of Deformation Load Factor, γ_{SE}

- Step-by-step approach to develop PEC for determination of load factor for deformation, γ_{SE}, is provided in WP Section 8.3.5
  - This approach is demonstrated by a numerical example in the next topic

Key Points

- There are overarching characteristics that apply to service limit states:
  1. Load-driven versus non-load-driven limit states
  2. Reversible versus irreversible limit states
  3. Consequences of exceeding deformation-related limit states and target reliability indices
  4. Calculation models
- Calibration of service limit state for foundation deformations require incorporation of load-deformation (Q-δ) behavior into the calculation models

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Foundation Deformations

Lecture 8

Review of White Paper, Chapter 9 to Chapter 11 and Appendix B

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June 28, 2017

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Foundation Deformations

Chapter 9 – Calibration Implementation

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June 28, 2017

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Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

Learning Outcomes

- Identify step-by-step process of implementation of calibration process for foundation deformations

- Learn the calibration process by a numerical example

Basic Framework for Calibration of Deformations

WP Table 9-1

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, e.g., 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 deformations. For each considered foundation type and deformation, the parameters to be calibrated must be determined, e.g., immediate settlement of a spread footing based on Hough method, lateral deflection of driven pile group at groundline based on P-y method.
4. Develop statistical models for load and resistance.	Gather statistical information about the performance of the considered deformation types and prediction models. Determine the accuracy (*X*) factor and statistics for loads based on prediction models. Resistance is often based on deterministic approach and its value will vary as a function of the considered structural limit state.
5. Apply the reliability analysis procedure.	Reliability can be calculated using the PEC method. In some cases, depending on the type of probability distribution function a closed form solution may be possible.
6. Review the results and develop the γ_{SE} load factors for target reliability indices.	Develop the γ_{SE} load factor for all applicable structural limits states and their corresponding target reliability indices and consideration of reversible and irreversible limit states
7. Select the γ_{SE} load factor.	Select an appropriate the γ_{SE} load factor based on owner criteria, e.g., reversible-irreversible condition.

Step 1

Formulate the Limit State functions and identify basic variables

- Limit State Function

  $g = R - Q$

  $g = \delta_T - \delta_P$ WP Equation 9-1

  where, δ_T is tolerable deformation (Resistance)

  and δ_P is predicted deformation (Load)

- For calibration of deformations, express g as a ratio

  $g = \delta_P / \delta_T$ WP Equation 9-2

Step 2

Identify and select representative structural types and design cases

- To demonstrate the calibration process and implementation, the following is used:
  - Structural type: Spread Footing
  - Design Case: Immediate Settlement

NOTE: Even though the example of immediate settlement of a spread footing has been selected, the calibration process illustrated by this example can be applied to calibrate vertical and lateral deformation for all structural foundation types (e.g., footings, drilled shafts, and driven piles) and retaining walls.

Step 3

Determine load and resistance parameters for the selected design cases

- Load Parameter
  - Predicted (or calculated) immediate settlement (vertical deformation), δ_P
- Resistance Parameter
  - Tolerable (or limiting or measured) immediate settlement (vertical deformation), δ_T

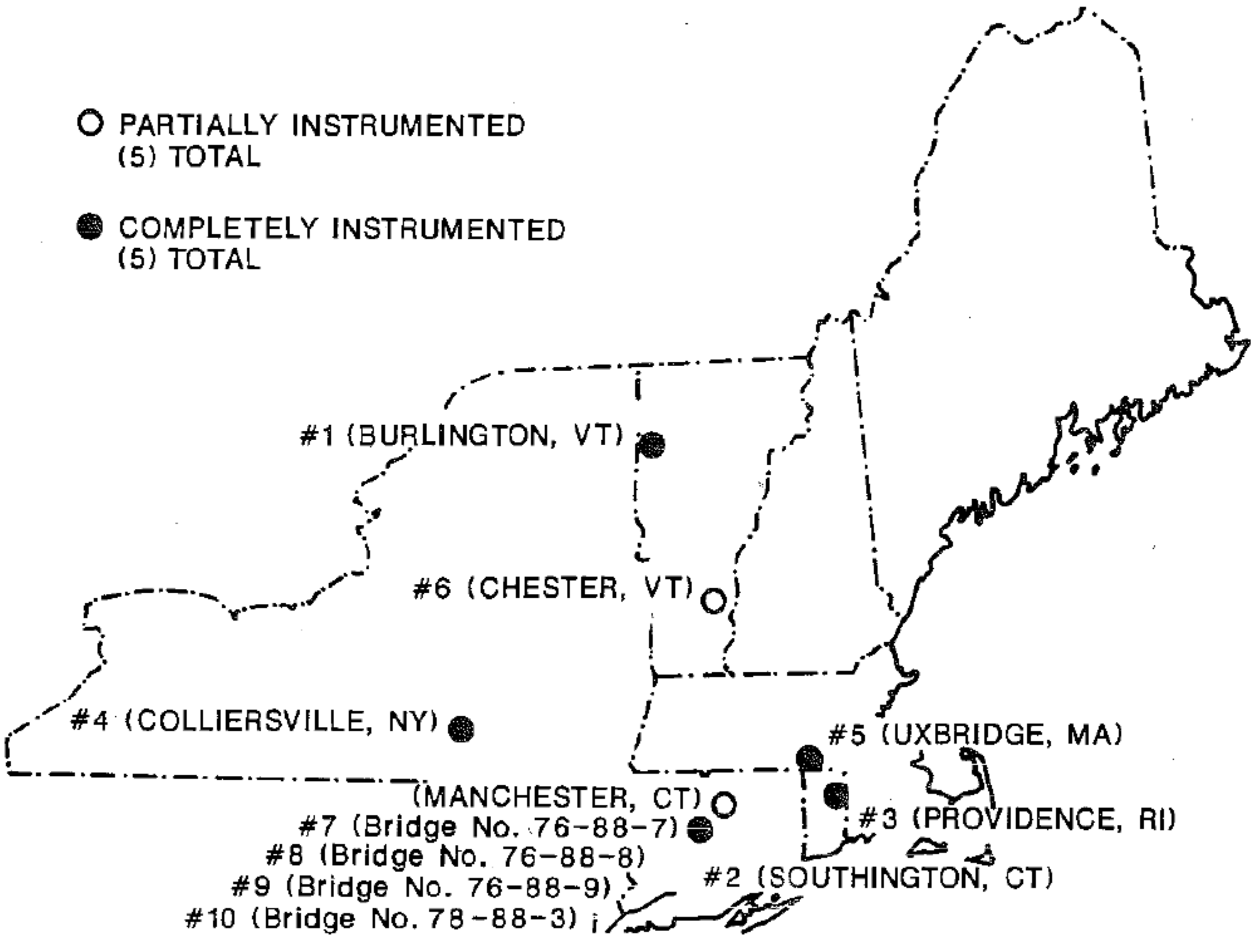
NOTE: *AASHTO LRFD* uses the symbol "*S*" for settlement. Therefore, for further discussions, the symbol *S* will be used instead of δ. Thus, S_P denotes predicted settlement and S_T denotes tolerable settlement.

Step 4

Develop statistical models for load and resistance

NOTE: A regional database from states in New England has been chosen for demonstration of the calibration process. This process has been applied to other regional databases from other DOTs (e.g., Washington State, Ohio, and South Carolina) as well as other databases (e.g., Texas A&M, Europe)

Example Database



Reference: Gifford et al. (1987)

Summary of Structures

- 20 footings
- Ten instrumented bridges in northeastern US
  - Five simple-span bridges
  - Five continuous-span bridges
  - Four 1-span, two 2-span, three 4-span, and one 5-span
    - Nine bridges were highway structures
    - One 4-span bridge carried railroad traffic across an interstate highway

Instrumentation

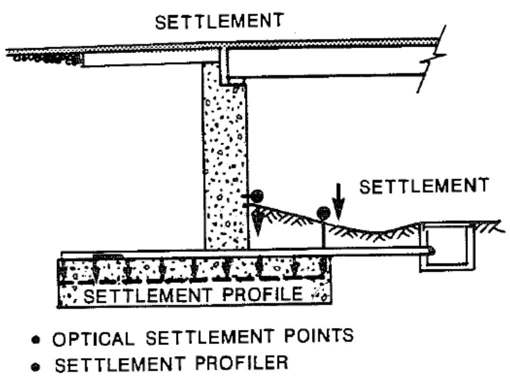


Figure 2. Settlement equipment for structures.

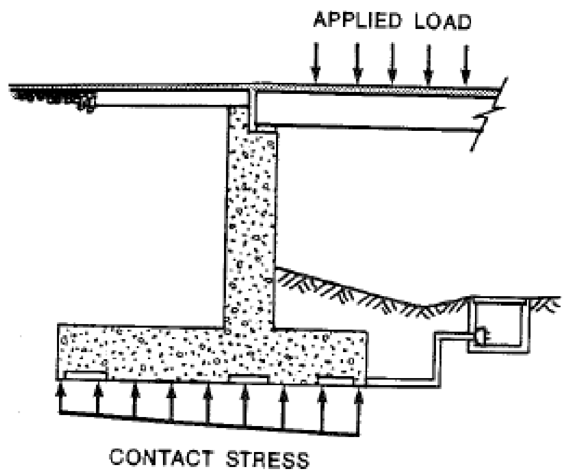


Figure 4 Contact stress and test loading.

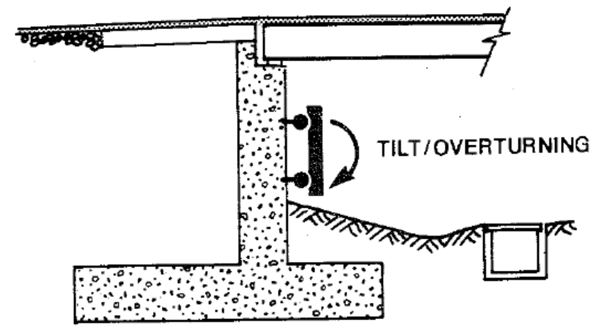


Figure 5. Tilt measurement equipment.

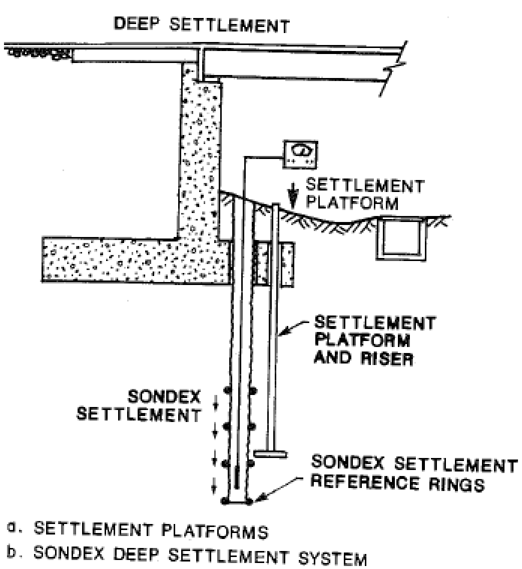


Figure 3. Settlement equipment in soil.

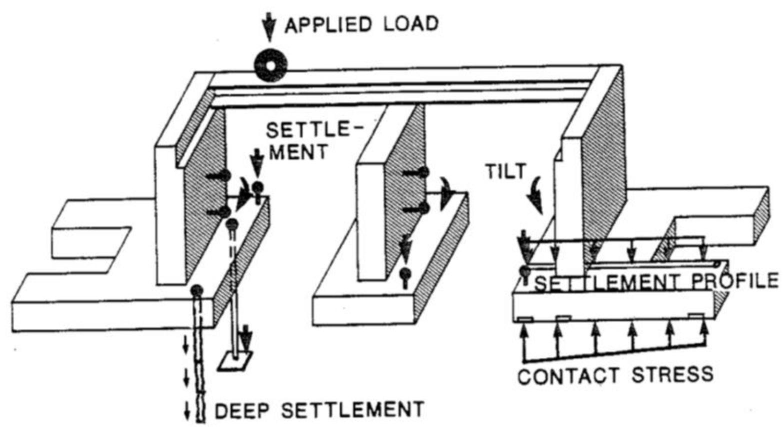


Figure 7. Conceptual plan: fully instrumented bridge.

Reference: Gifford et al. (1987)

Data for Measured and Predicted Settlement

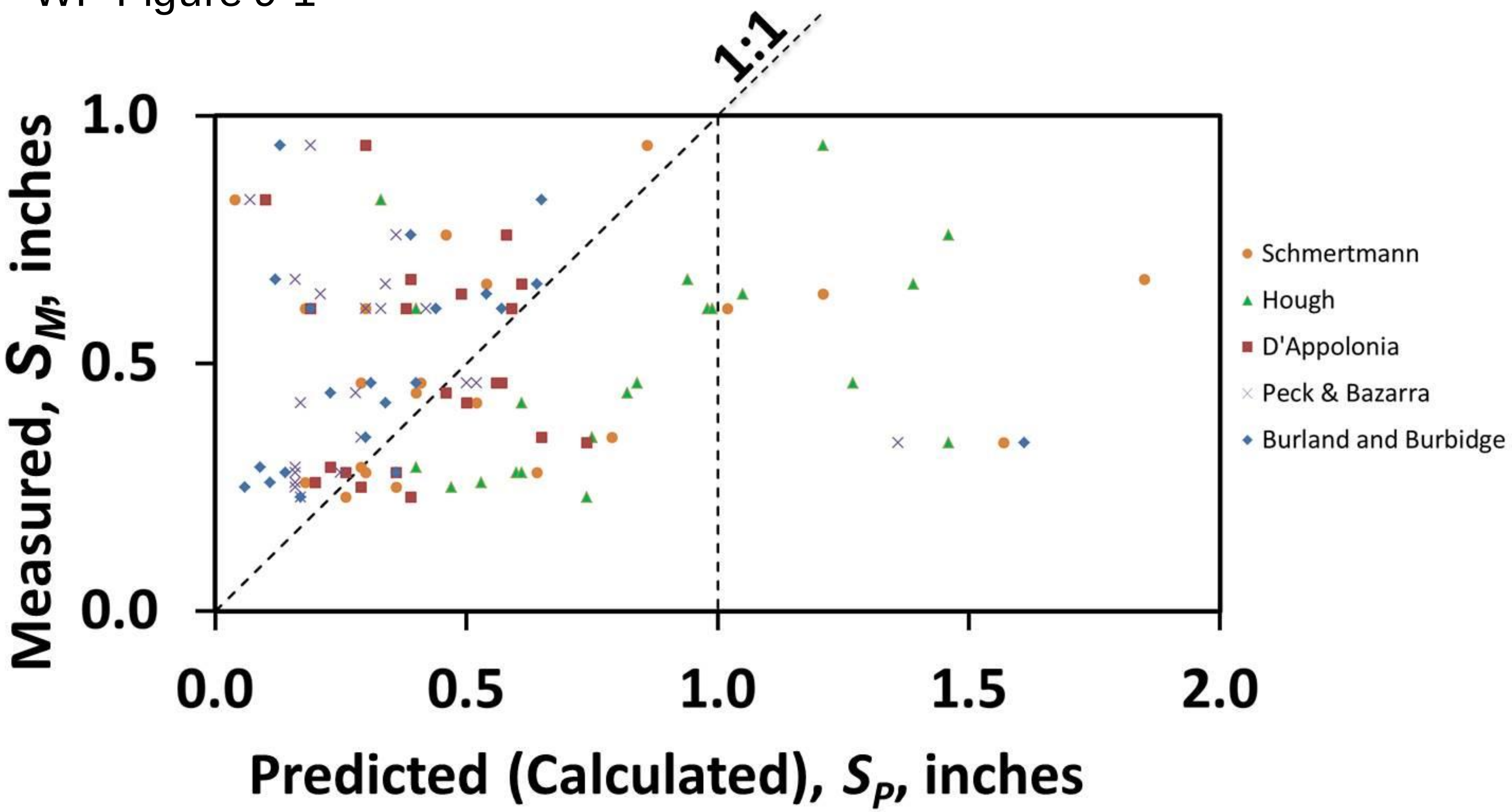
WP Table 9-2

Site	Settlement (in.)					
	Measured (S_M)	Predicted (Calculated) (S_P)				
		Schmertmann	Hough	D'Appolonia	Peck and Bazzara	Burland and 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	0.33	0.10	0.07	0.65
#21	0.46	0.29	1.05	0.49	0.21	0.54
#22	0.66	0.54	0.84	0.56	0.52	0.31
#23	0.61	1.02	1.39	0.61	0.34	0.64
#24	0.28	0.64	0.99	0.59	0.33	0.44

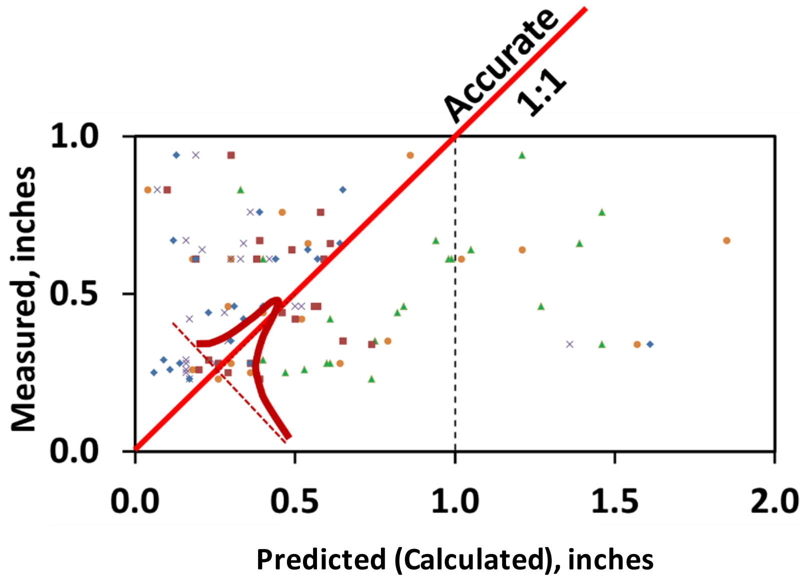
Note 1: Gifford, et al. (1987) notes that data for footings at Site #12, #13, and #18 were not included because construction problems at these sites resulted in disturbance of the subgrade soils and short term settlement was increased. Data for footing at Site #19 appears to be anomalous and have been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

Data for Immediate Settlement of Spread Footings

WP Figure 9-1



Concept of Accuracy and Bias



- Accurate method: $S_P = S_M \rightarrow S_P / S_M = 1.0$
- Accuracy, $X = S_P / S_M$ Bias, $\lambda = 1/X = S_M / S_P$
- Concept of Accuracy is used herein
- Accuracy, X, is a random variable

Data for Accuracy, X (= S_P / S_M)

WP Table 9-3

Site	Accuracy, X (= S_P / S_M)				
	Schmertmann	Hough	D'Appolonia	Peck and Bazzara	Burland and Burbridge
#1	2.257	2.143	1.857	0.829	0.857
#2	2.761	1.403	0.582	0.239	0.179
#3	0.915	1.287	0.319	0.202	0.138
#4	0.605	1.921	0.763	0.474	0.513
#5	0.492	1.607	0.623	0.689	0.934
#6	1.238	1.452	1.190	0.405	0.810
#7	0.295	0.656	0.311	0.492	0.311
#8	1.071	2.143	0.929	0.571	0.500
#9	0.692	2.038	0.769	0.615	0.423
#10	1.000	1.379	0.793	0.552	0.310
#11	1.440	1.880	1.160	0.640	0.240
#14	0.891	2.761	1.239	1.087	0.870
#15	4.618	4.294	2.176	4.000	4.735
#16	1.130	3.217	1.696	0.739	0.739
#17	0.909	1.864	1.045	0.636	0.523
#20	1.891	1.641	0.766	0.328	0.844
#21	0.630	1.826	1.217	1.130	0.674
#22	0.818	2.106	0.924	0.515	0.970
#23	1.672	1.623	0.967	0.541	0.721
#24	2.286	2.179	1.286	0.893	1.286

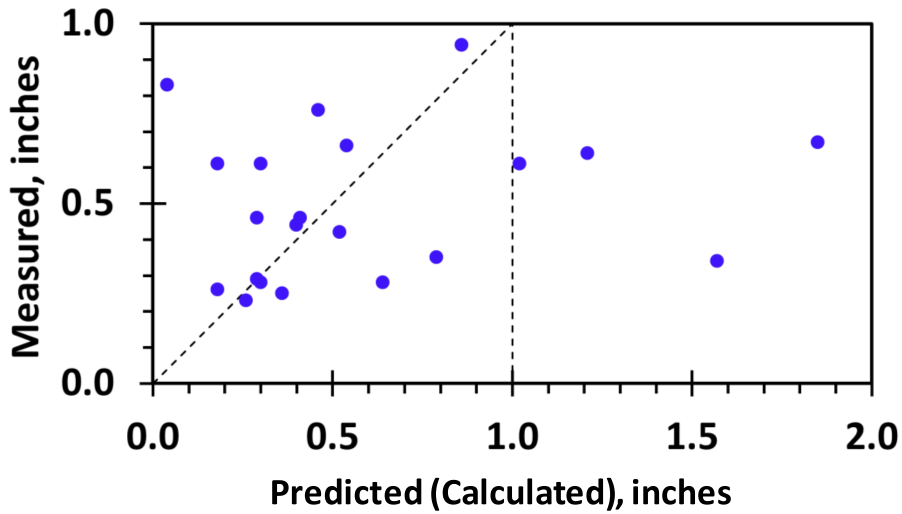
Statistics of Accuracy, *X* (= S_P / S_M)

WP Table 9-4

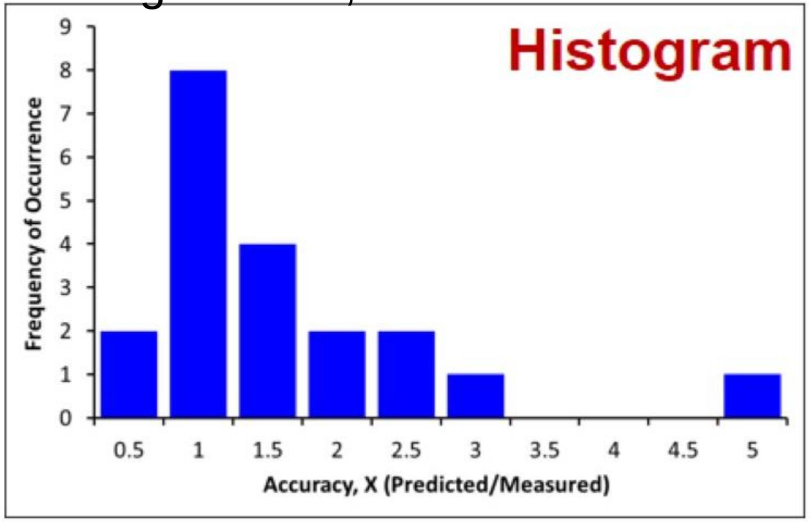
Statistic	Schmertmann	Hough	D'Appolonia	Peck & Bazzara	Burland & Burbridge
Count	20	20	20	20	20
Min	0.295	0.656	0.311	0.202	0.138
Max	4.618	4.294	2.176	4.000	4.735
μ	1.381	1.971	1.031	0.779	0.829
σ	1.006	0.769	0.476	0.796	0.968
CV	0.729	0.390	0.462	1.022	1.168

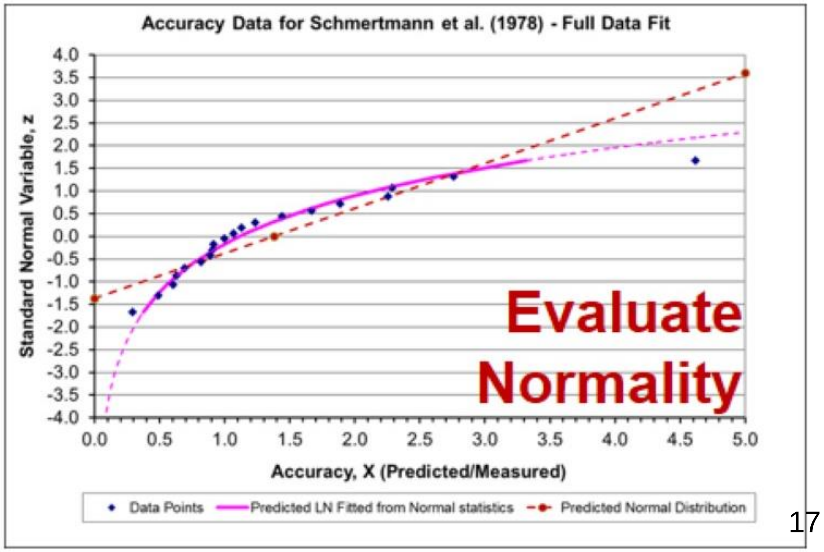
Legend:
μ = Mean
σ = Standard Deviation
CV = Coefficient of Variation (= σ/μ)

Schmertmann Data



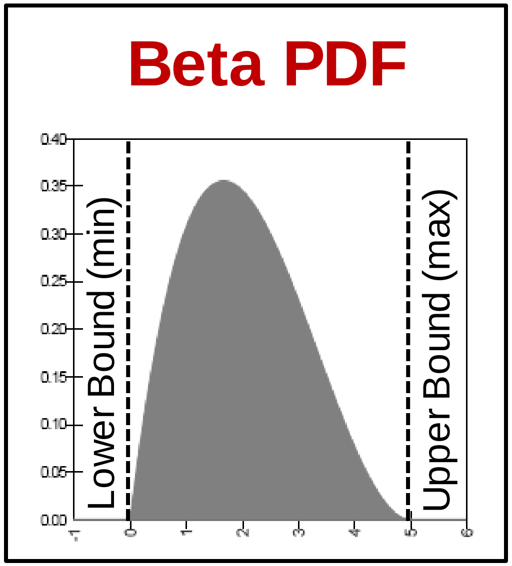
WP Figure 9-2a,b

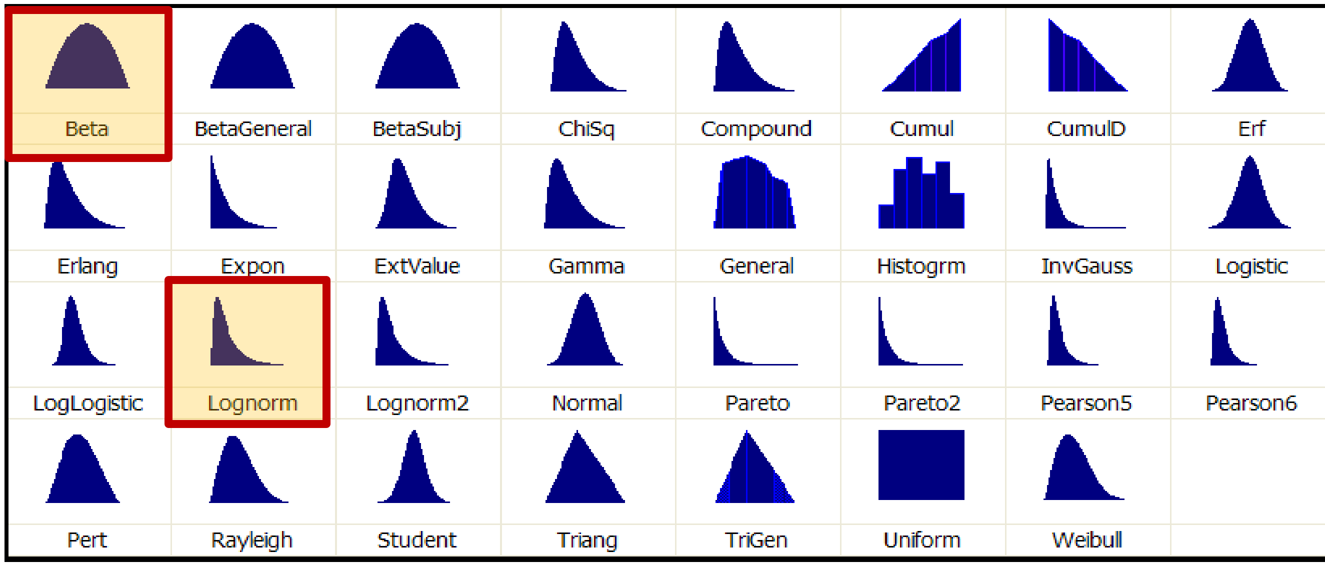




- Data are non-normal
- Which **Probability Distribution Function** (PDF) is the best to represent non-normal data?

Probability Distribution Functions (PDFs)





Reference for PDF Schematics: @Risk by Palisade Corporation

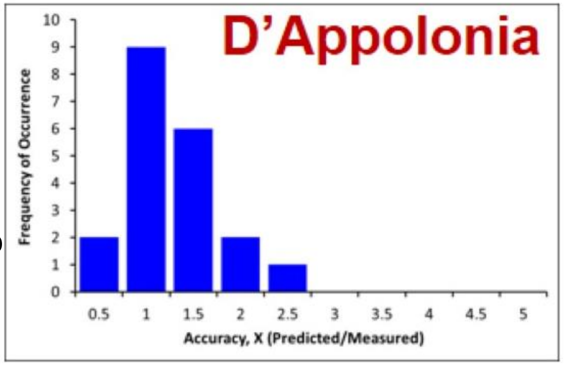
Calibration concept applies regardless of PDF chosen

Non-Normal Data

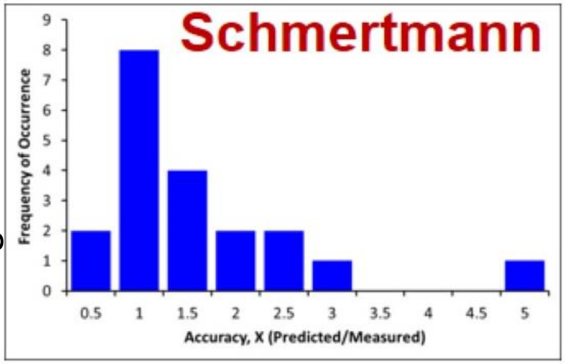
Histograms for Various Methods

Skewed to the right

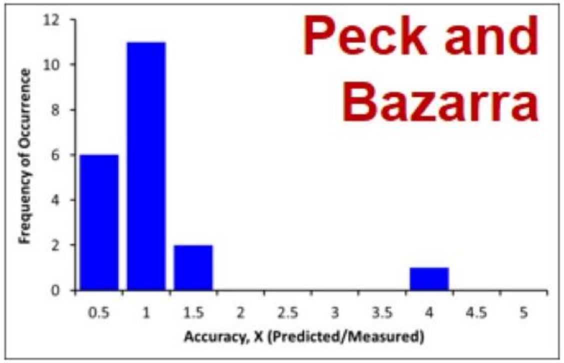
WP Figure 9-4a



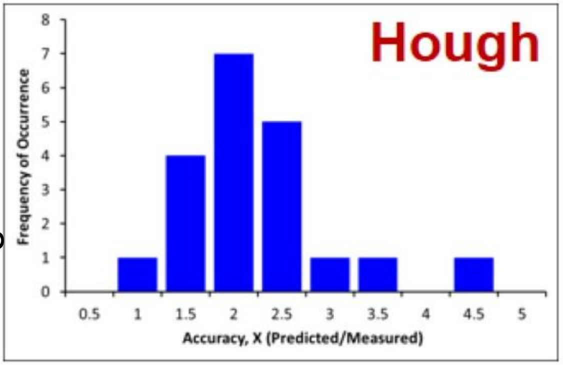
WP Figure 9-2a



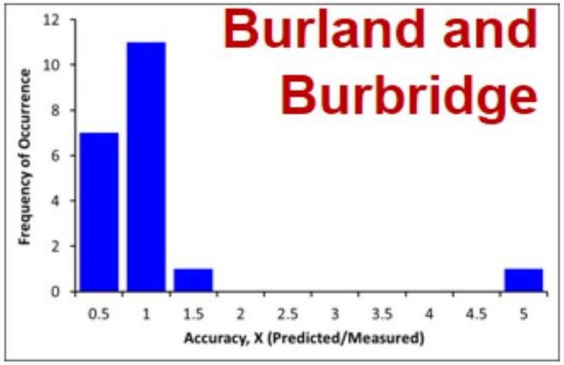
WP Figure 9-5a



WP Figure 9-3a



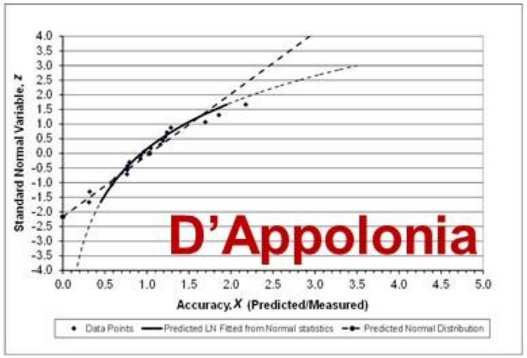
WP Figure 9-6a



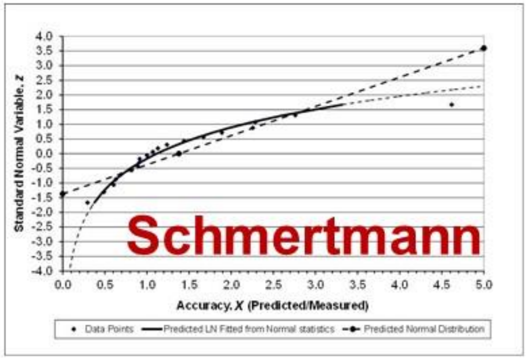
Evaluate Normality

Plot Standard Normal Variable (z) as a Function of Accuracy (X)

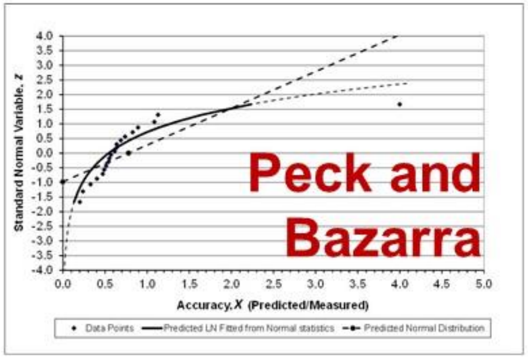
WP Figure 9-4b



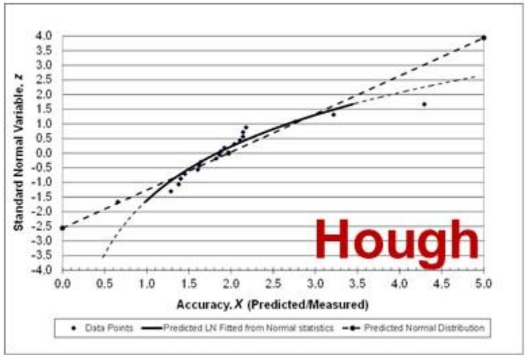
WP Figure 9-2b



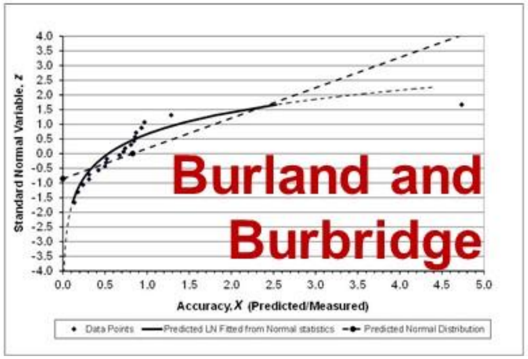
WP Figure 9-5b



WP Figure 9-3b



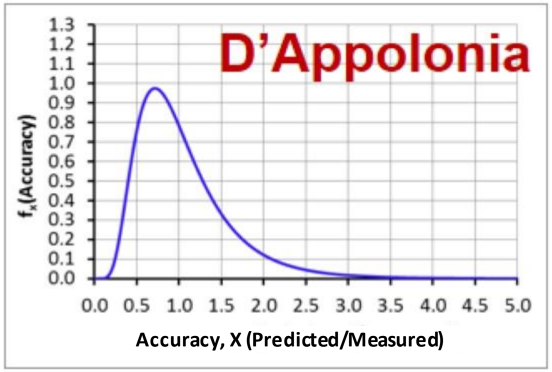
WP Figure 9-6b

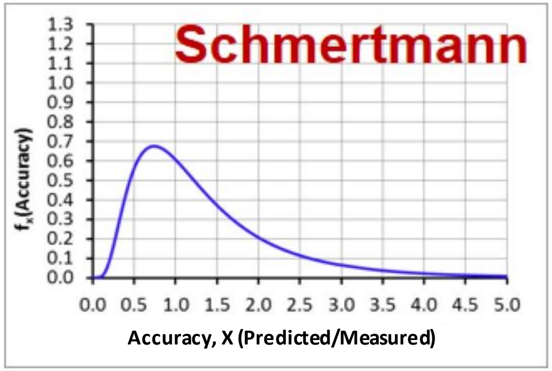


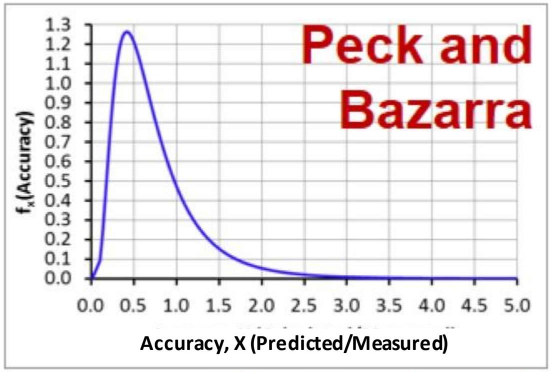
Non-Normal Data

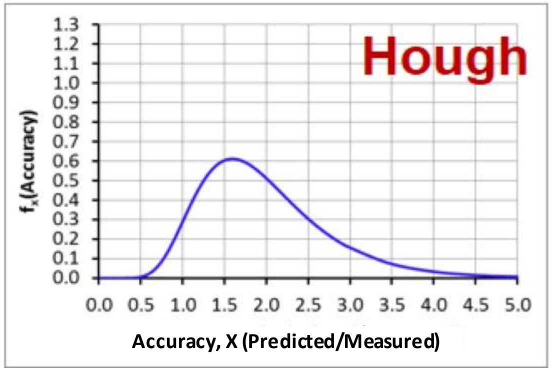
PDF's skewed to the right (reverse-J)

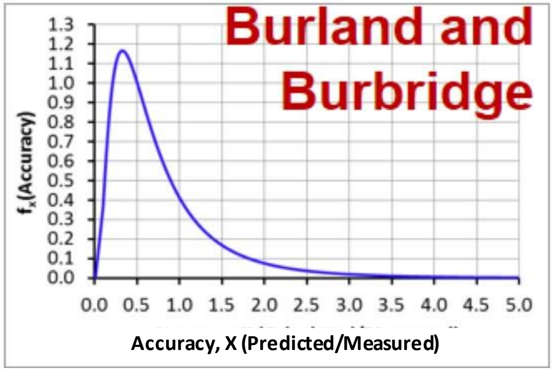
Use Lognormal PDF to be consistent with LRFD practice thus far





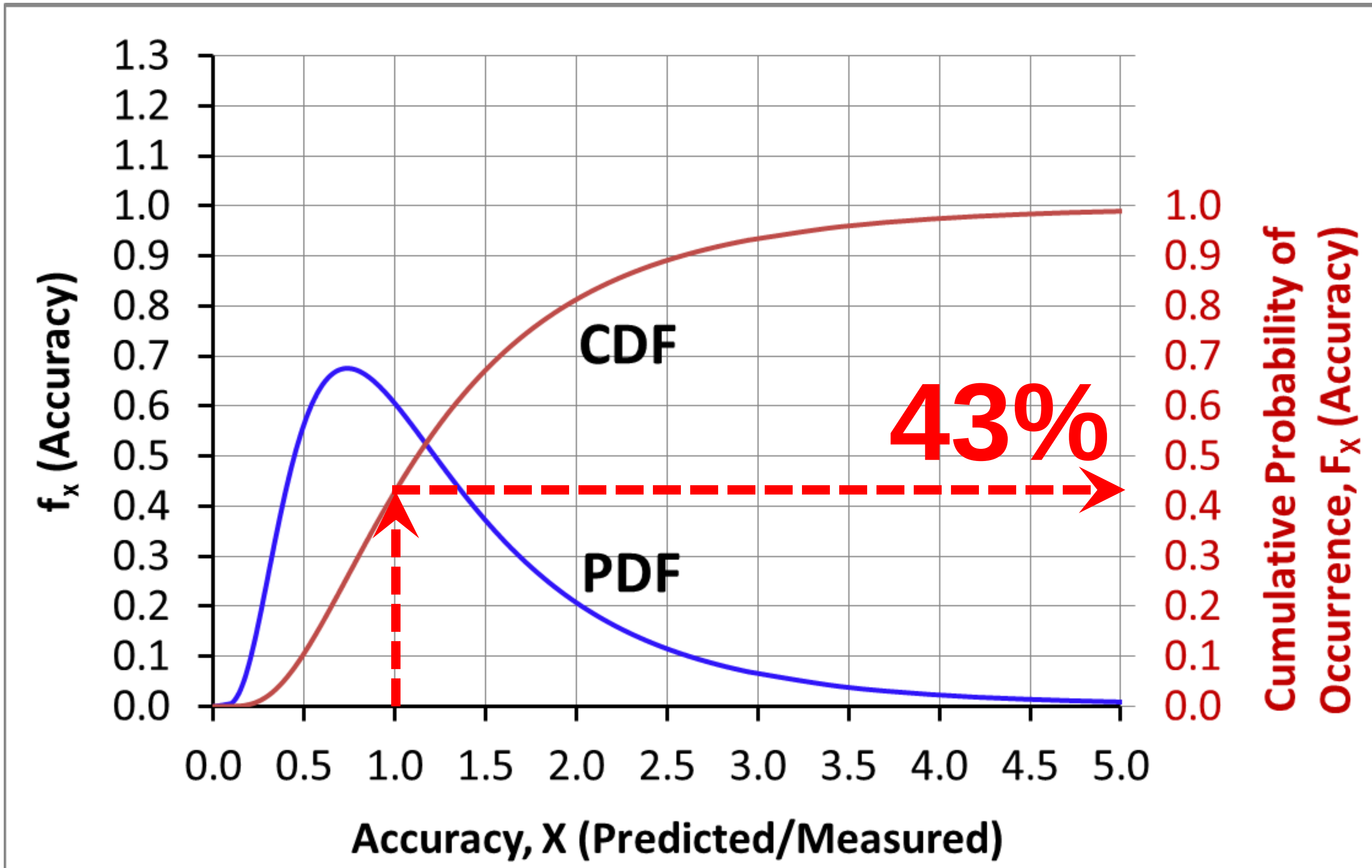






Convert PDF to CDF

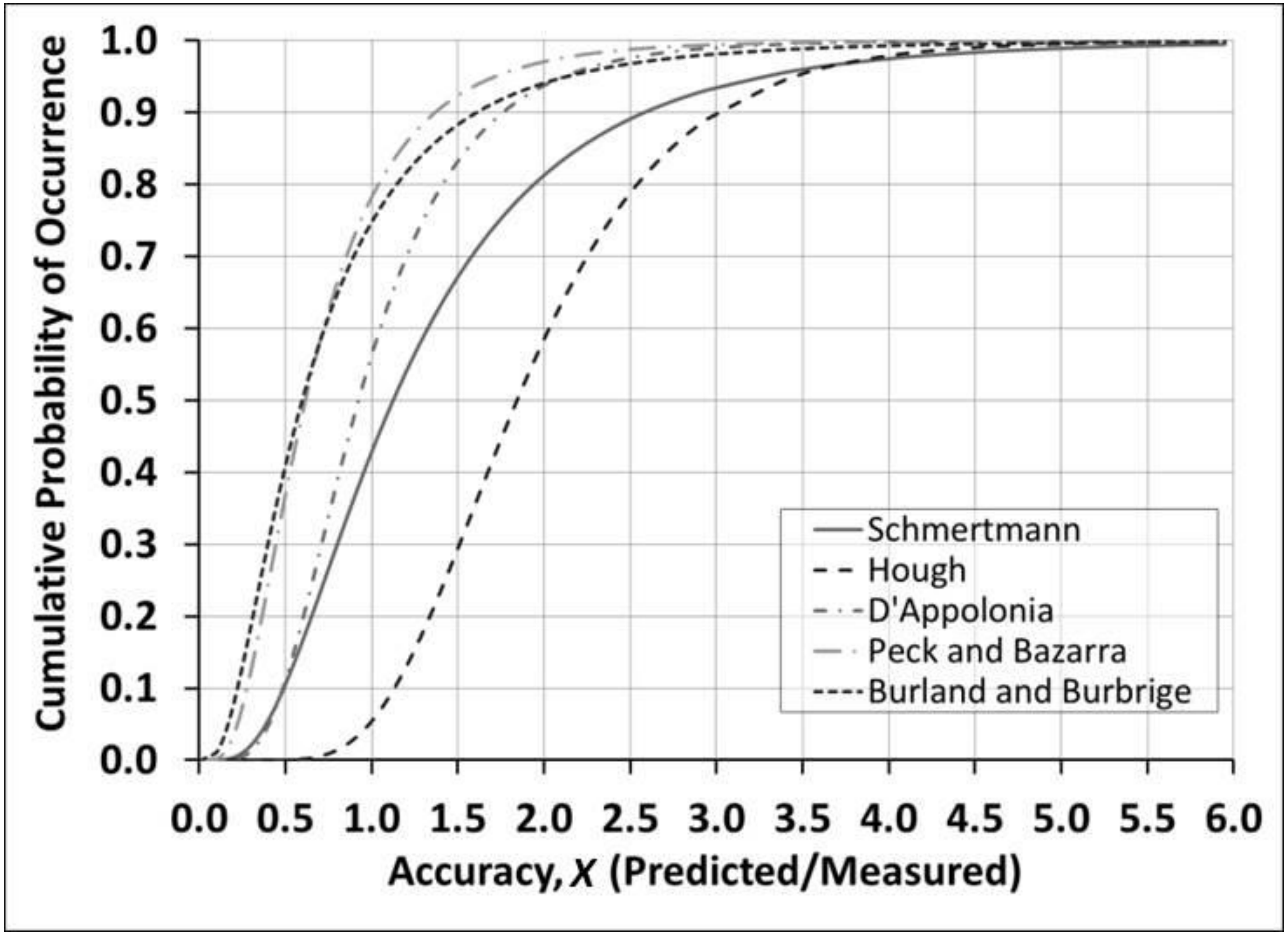
Example: Schmertmann



PDF: Probability Distribution Function; CDF: Cumulative Distribution Function

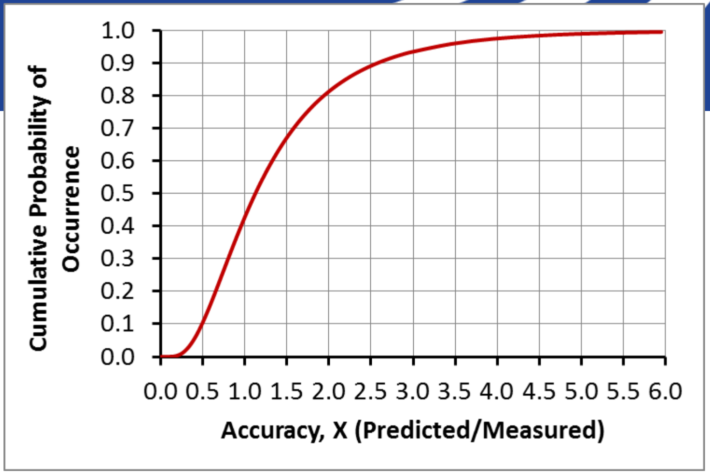
CDFs for Different Prediction Methods

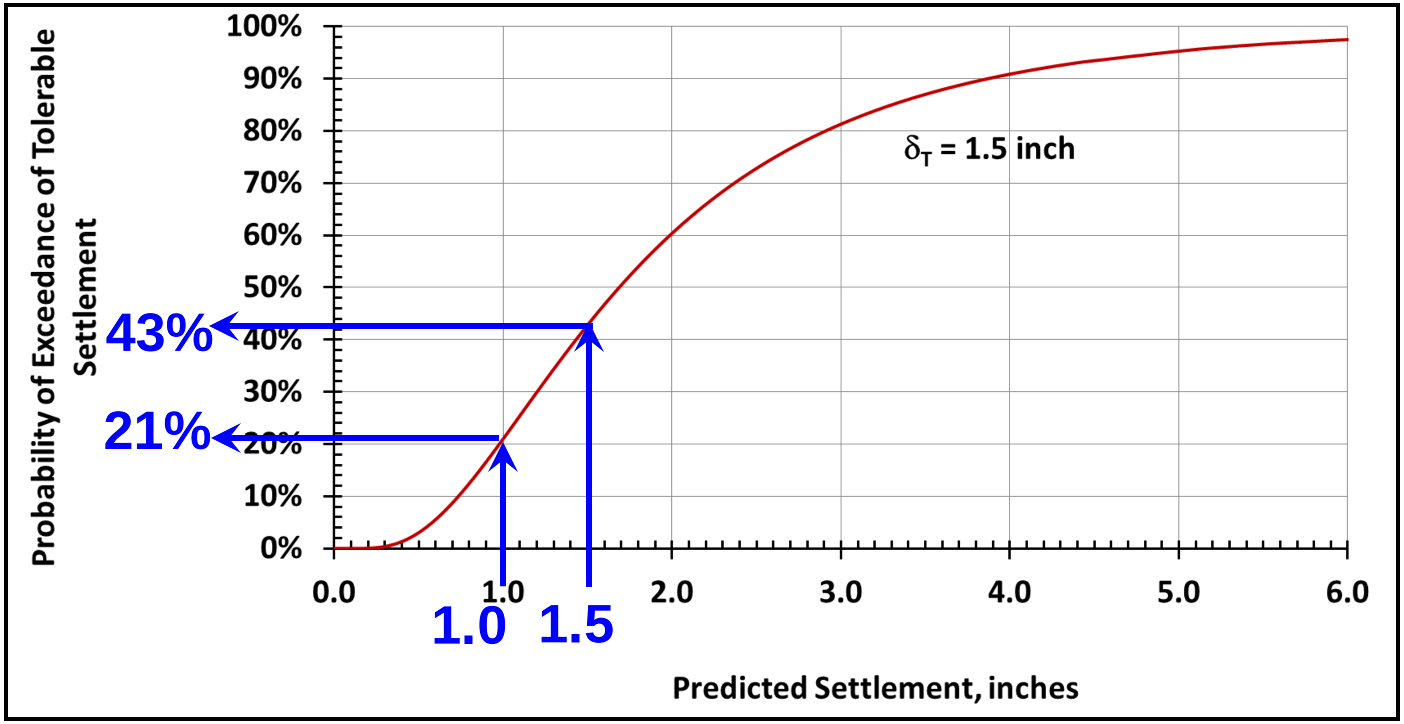
WP Figure 9-7



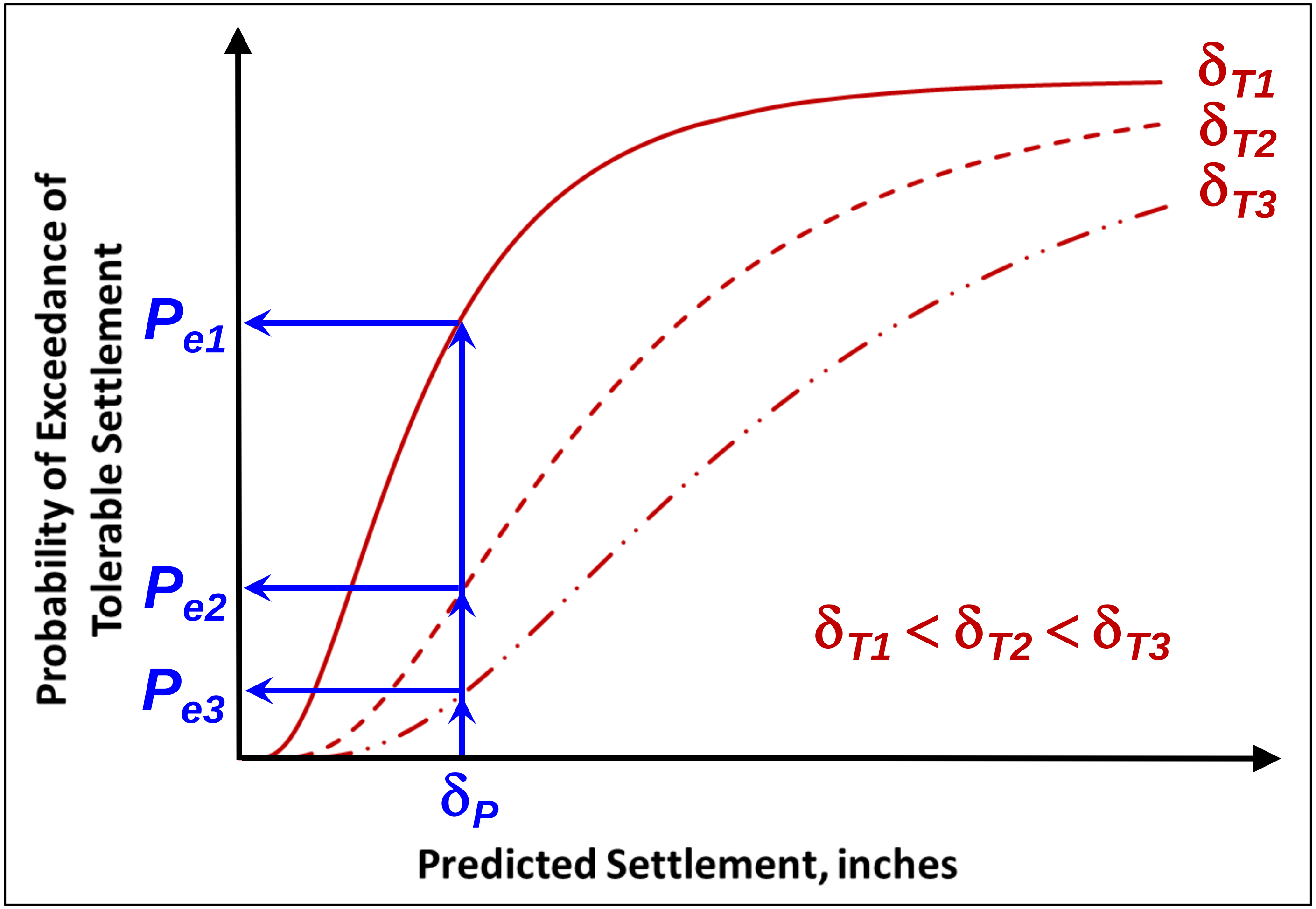
Generate Probability Exceedance Chart (PEC) from CDF

Example: Schmertmann





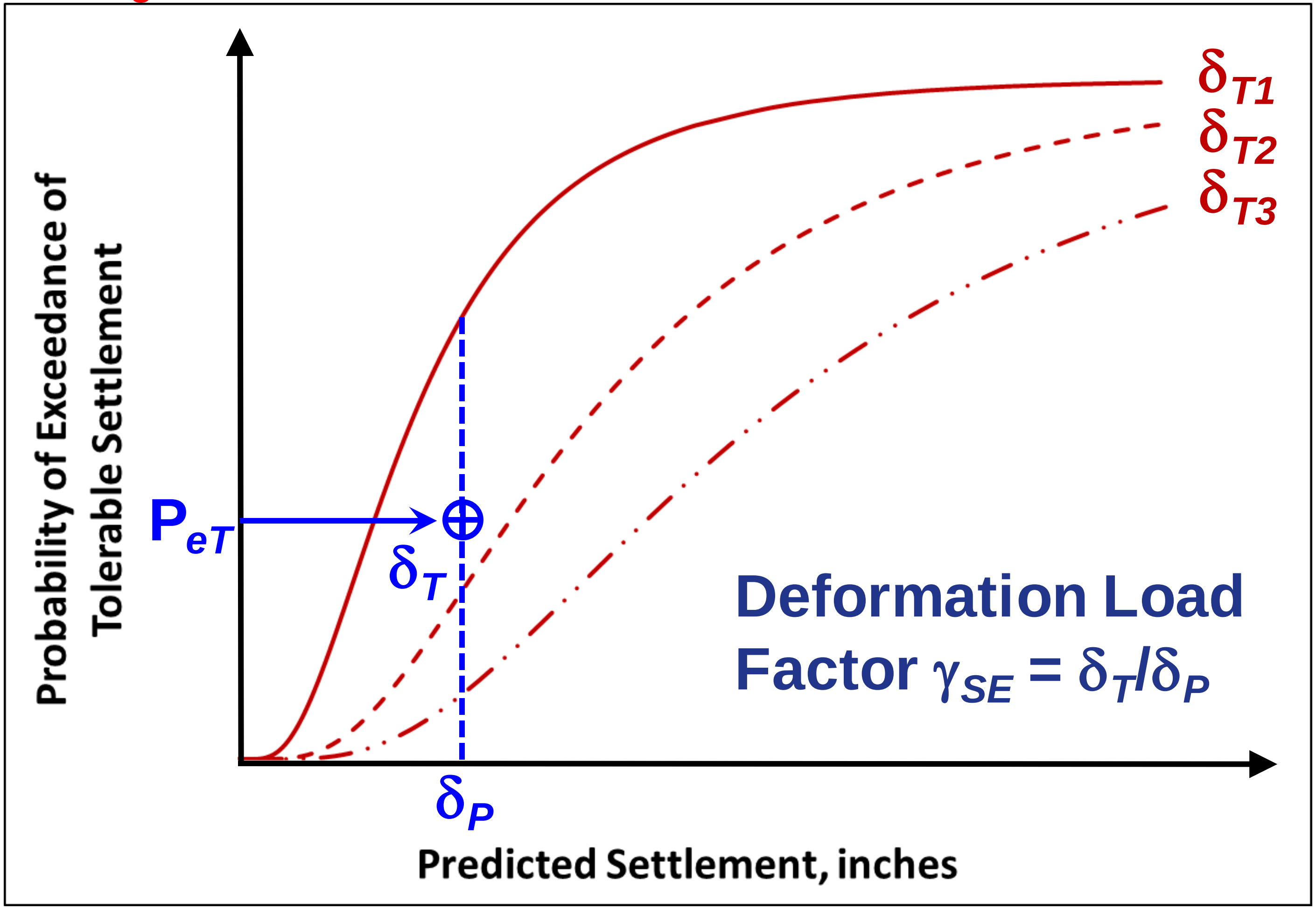
PEC with Family of Curves



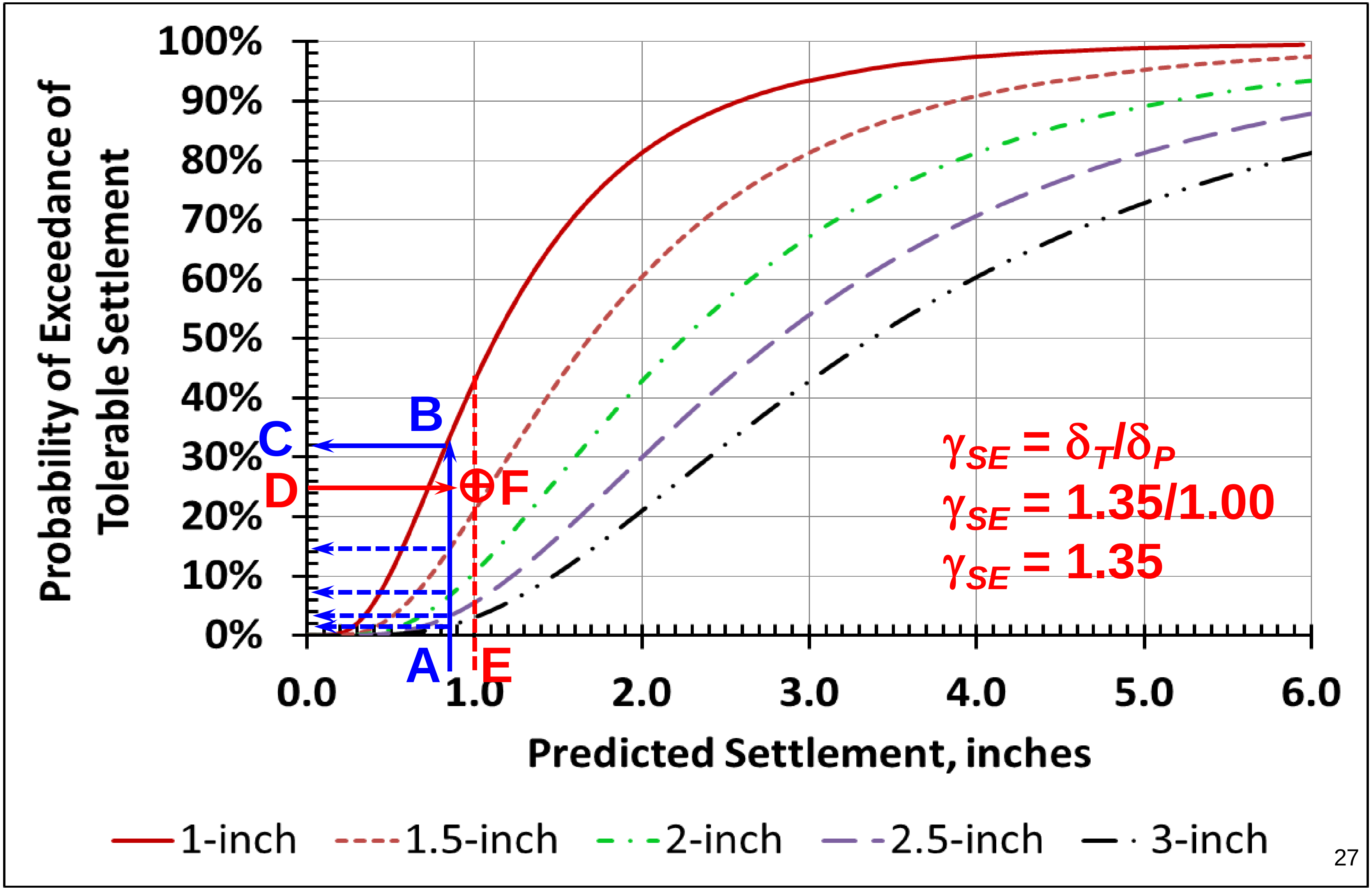
Load Factor γ_{SE}

WP Figure 8-7

$\delta_{T1} < \delta_{T2} < \delta_{T3}$



For Schmertmann Method (WP Figure 9-8)



Probability of Exceedance, P_e, For Structural Limit States

WP Table 9-11

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*	1.6 (Class 1) 1.0 (Class 2)	5% 16%
Service II limit state for yielding of steel and for bolt slip*	1.8	4%

Note 1: P_e is based on "Normal" Distribution

* No desire to change

Step 5

Apply the Reliability Analysis Procedure

- Express probability of exceedance, P_e in terms of reliability index, β

Express β in Terms of P_e

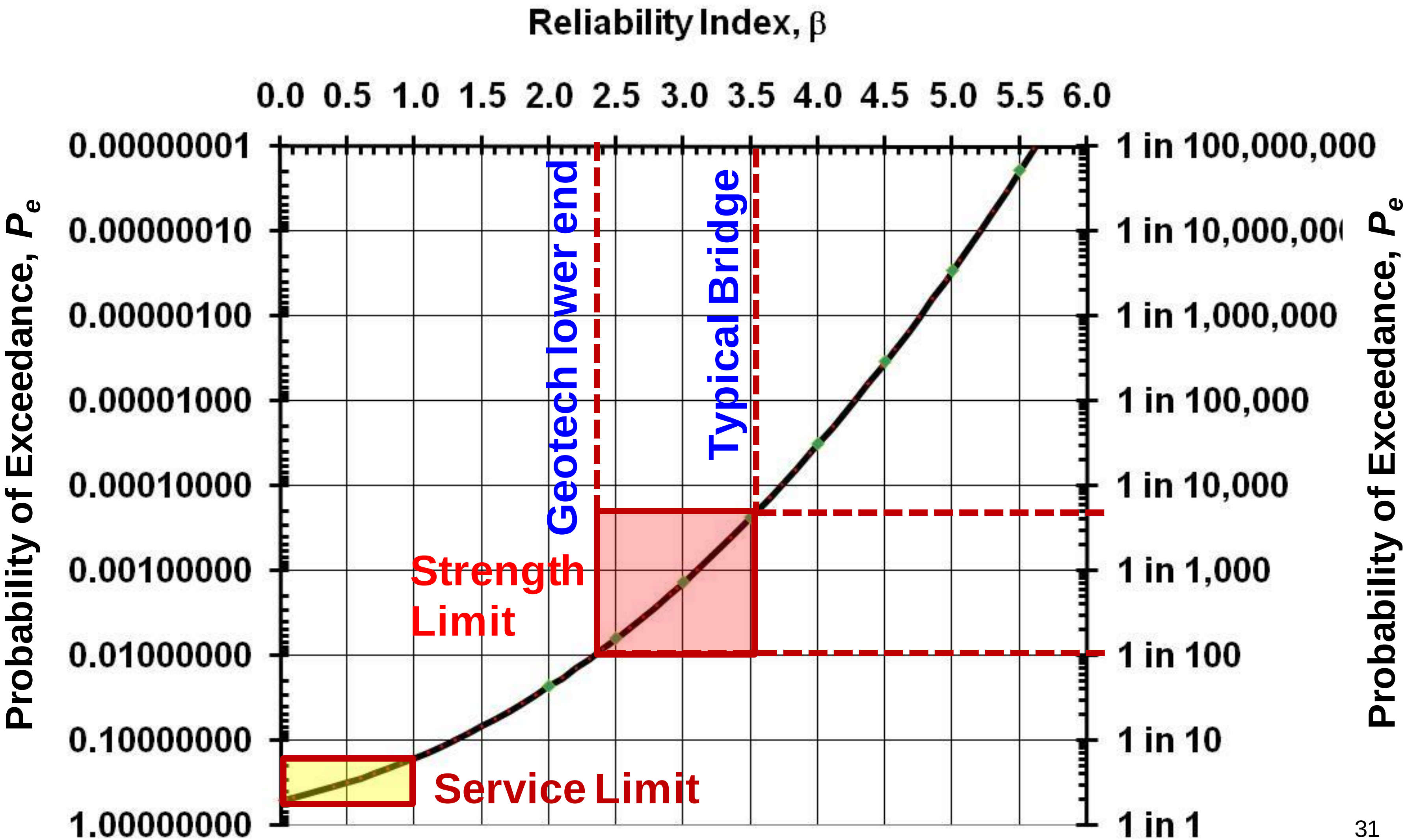
- Conventional definition of β

$$\beta = \frac{R_{mean} - Q_{mean}}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

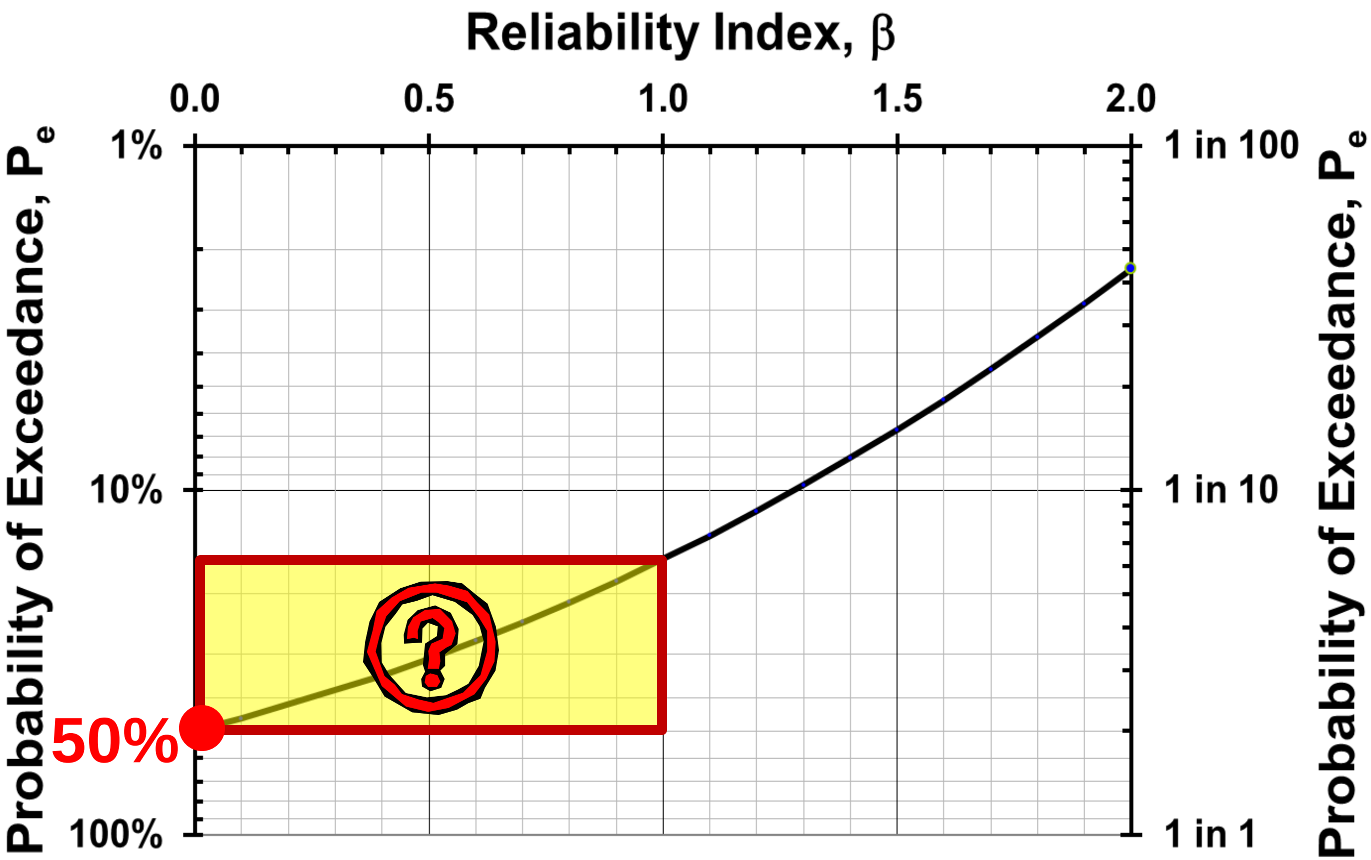
- Using Microsoft Excel, the relationship can be expressed as follows:

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

Reliability Index β vs P_e for "Normal" Distribution



What Value of β to Use? (WP Figure 9-13)



What about consequences?

What Value of β to Use?

P_e, %	β	P_e, %	β	P_e, %	β	P_e, %	β
0.01	3.719	11	1.227	25	0.674	39	0.279
0.02	3.540	12	1.175	26	0.643	40	0.253
0.05	3.291	13	1.126	27	0.613	41	0.228
0.1	3.090	14	1.080	28	0.583	42	0.202
1	2.326	15	1.036	29	0.553	43	0.176
2	2.050	16	0.994	30	0.524	44	0.151
3	1.875	17	0.954	31	0.496	45	0.126
4	1.750	[illegible]	[illegible]	32	0.468	46	0.100
5	1.645	[illegible]	[illegible]	33	0.440	47	0.075
6	1.555	20	0.842	[illegible]	[illegible]	48	0.050
7	1.476	21	0.806	[illegible]	[illegible]	49	0.025
8	1.405	22	0.772	36	0.358	50	0.000
9	1.341	23	0.739	37	0.332		
10	1.282	24	0.706	38	0.305		

B

G

Irreversible

Reversible
Irreversible

β Versus γ_{SE} for Various Methods

WP Table 9-10

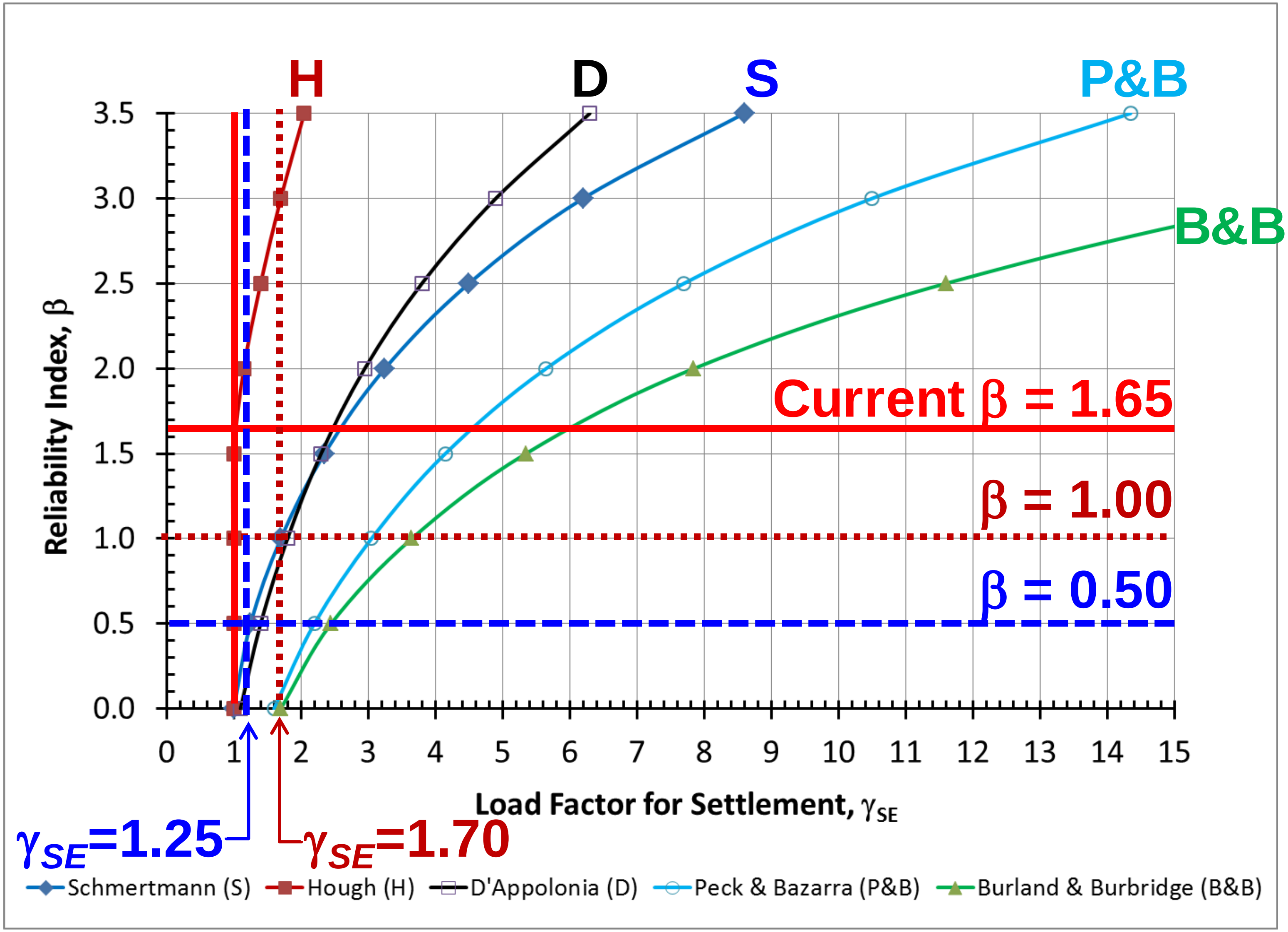
β	γ_{SE}				
	S	H	D	P&B	B&B
0.00	1.00	1.00	1.10	1.60	1.70
0.50	1.25	1.00	1.40	2.20	2.45
1.00	1.70	1.00	1.80	3.05	3.65
1.50	2.35	1.00	2.30	4.15	5.35
2.00	3.25	1.15	2.95	5.65	7.85
2.50	4.50	1.40	3.80	7.70	11.60
3.00	6.20	1.70	4.90	10.50	17.05
3.50	8.60	2.05	6.30	14.35	25.10
Legend: S: Schmertmann, H: Hough, D: D'Appolonia, P&B: Peck and Bazarra, B&B: Burland and Burbridge					

Step 6

Review of Results and Development of Load Factor for Settlement, γ_{SE}

- Plot the results and observe the trends

Development of γ_{SE} Based on β Value (WP Figure 9-14)



Step 7

Select Value of γ_{SE}

- Based on consideration of irreversible (β = 1.00) and reversible-irreversible (β = 0.50) limit states or any other owner specified value of β based on local practice as appropriate

Key Points

- A step-by-step process for implementation of calibration process for foundation deformations is available

- Microsoft Excel® can be used for the calibration process
  – See example in Section 9.2.5 (Step 5) of White Paper




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Foundation Deformations

Chapter 10 – Meaning and Effect of γ_{SE} in Bridge Design Process

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NCS GeoResources, LLC

June 28, 2017

U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO

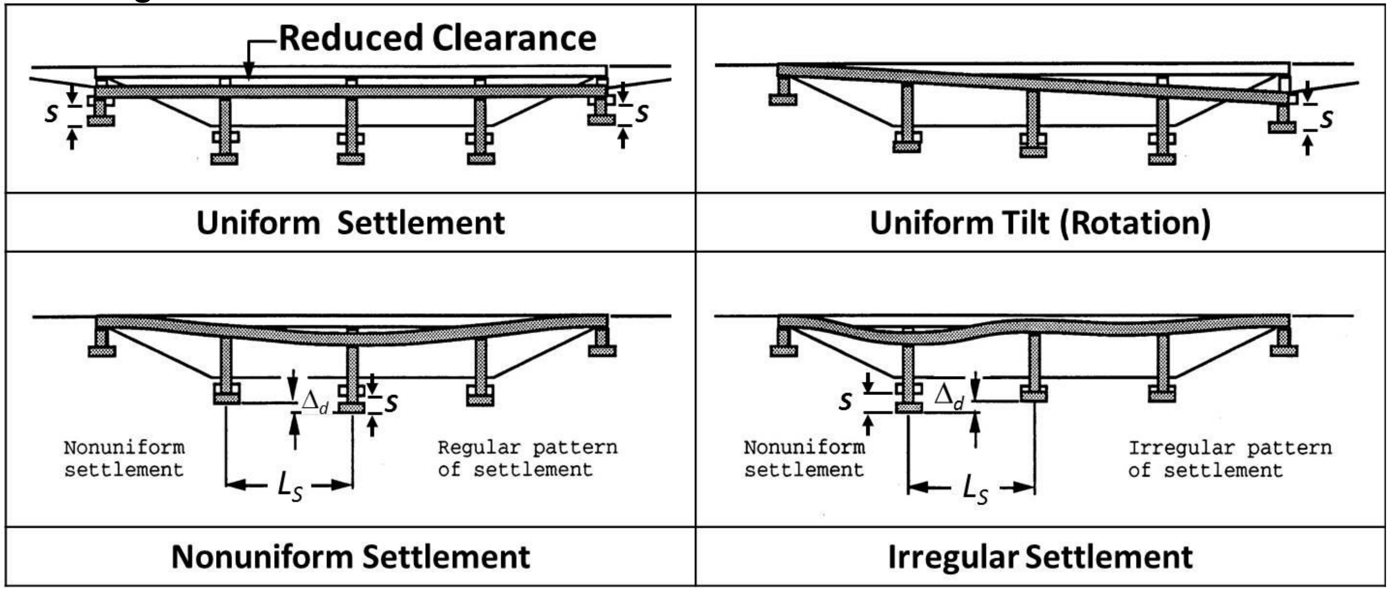
Learning Outcomes

- Understand the meaning of γ_{SE} load factor in context of bridge design

- Understand the effect of γ_{SE} load factor in context of bridge design

Meaning and Use of γ_{SE}

WP Figure 4-1



- Bridge deck (superstructure) implications
  - Force effect (e.g., moment) = *func* (EI/L_S, Δ_d/L_S)
- Implications for facilities at abutments (e.g., joints, approach slabs, utilities, etc.), roadway grade, and vertical clearance

Effect of Foundation Deformations On Superstructures

- For all bridges, stiffness should be appropriate to considered limit state
- The effect of continuity with the substructure should be considered
- Consider all viable deformation shapes
- For concrete bridges, the determination of the stiffness of the bridge components should consider the effect of cracking, creep, and other inelastic responses

Some Observations

- Deformations generate additional force effects
  - Load factor of *SE* is similar to *PS*, *CR*, *SH*, *TU*, and *TG*
- The value of γ_{SE} must not be taken literally:
  - $\gamma_{SE} = 1.25$ does not mean that the total force effects will increase by 25%
  - γ_{SE} is only one component in a load combination
- Use of construction point concept in conjunction with γ_{SE} incorporates force effects related to expected sequence of construction along with quantification of uncertainty in predicted deformations

Some Observations

- In general, the factored design force effects for shorter spans will be affected by the proposed provisions more than longer spans
- The additional moments due to effect of deformations are very dependent on the stiffness of the bridge (EI/L_S) as well as the angular distortion (Δ_d/L_S)
- In performing the design, if including the settlement decreases a certain force effect at a section, the force effect calculated ignoring the effect of the settlement should be used for the design

Results of Initial Limited Parametric Study

- Several 2- and 3-span steel and pre-stressed concrete continuous bridges from NCHRP Project 12-78
  - Considered full angular distortion (Moulton’s criteria)
- Finding: An increase in factored Strength I moments on the order of as little as 10% for the more flexible units to more than double the moment from only factored dead and live load moments for the stiffer units
  - Finding is based on elastic analysis and without consideration of creep, which could significantly reduce the moments, especially for relatively stiff concrete bridges
  - Additional examples were developed to study effects

Key Points

- The γ_{SE} load factor is just one of the several load factors in a load combination

- Use of construction point concept in conjunction with γ_{SE} incorporates force effects related to expected sequence of construction, along with quantification of uncertainty in predicted deformations




SHRP2 SOLUTIONS
STRATEGIC HIGHWAY RESEARCH PROGRAM

Foundation Deformations

Chapter 11 – Incorporating Values of of γ_{SE} in AASHTO LRFD

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NCS GeoResources, LLC

June 28, 2017

U.S. Department of Transportation
Federal Highway Administration

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
AASHTO

Learning Outcomes

- Understand how the γ_{SE} load factor is proposed to be incorporated into *AASHTO LRFD*

Proposed Modifications to AASHTO

Article 10.5.2 – “Service Limit States”

- Article 10.5.2 is cross-referenced in articles for various foundation types such as spread footings, driven piles, drilled shafts, micropiles, retaining walls, joints, etc.
- Making change in Article 10.5.2 will permeate through all the relevant sections of *AASHTO LRFD*

Section 3, Table 3.4.1-3

WP Figure 3-3

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 • using $I_{effective}$	 0.5 1.0	 0.5 1.0
Steel Substructures	1.0	1.0

- Include the γ_{SE} in above table or develop a similar table

Section 3, New Table 3.4.1-4 For γ_{SE}

WP Table 11-1

Deformation	*SE*
Immediate Settlement • Hough method • Schmertmann method • Local method	 1.00 1.25 *
Consolidation settlement	1.00
Lateral deformation • *P-y* or SWM soil-structure interaction method • Local method	 1.00 *
*To be determined by the Owner based on local geologic conditions	

Key Points

- The γ_{SE} load factor is proposed to be incorporated into *AASHTO LRFD* using treatment similar to those for other superimposed deformations

- Making a change in **Article 10.5.2** will permeate through all the relevant sections of *AASHTO LRFD*




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Foundation Deformations

Appendix B – Application of γ_{SE} Load Factor

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June 28, 2017

U.S. Department of Transportation
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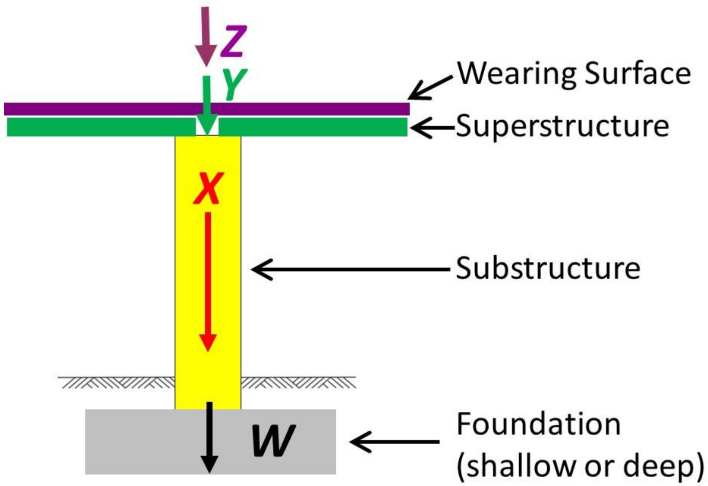
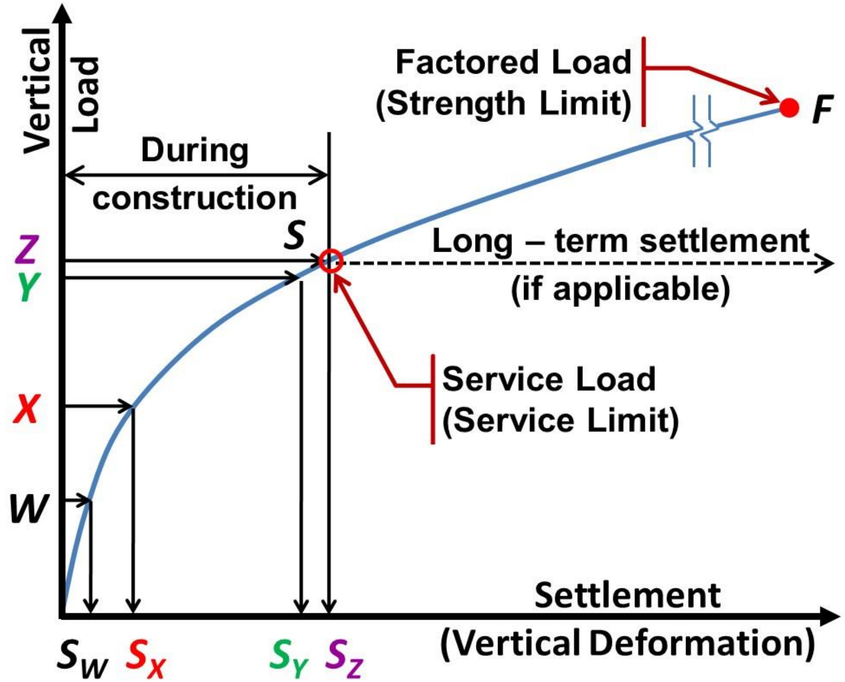
Learning Outcomes

- Learn the application of the γ_{SE} load factor for computation of factored deformations through a numerical example

Application of γ_{SE} Load Factor – Appendix B

WP Figure 6-1 (a) and (b)





- Implications for facilities at abutments (e.g., joints, approach slabs, utilities, etc.), roadway grade, and vertical clearance

Application of γ_{SE} Load Factor – Appendix B, Example

- Four-span bridge
- Immediate settlement
  - Two methods:
    - Hough → γ_{SE} = 1.00
    - Schmertmann → γ_{SE} = 1.25
- Consolidation (long-term) settlement → γ_{SE} = 1.00

Application of γ_{SE} Load Factor – Appendix B, Example

WP Table B-1

Support Element	Unfactored Predicted Settlements				
	Immediate Settlement (NOTE 1)			Consolidation Settlement (in.) (NOTE 2)	Total Relevant Settlement, S_{tr} (in.) (NOTE 3)
	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

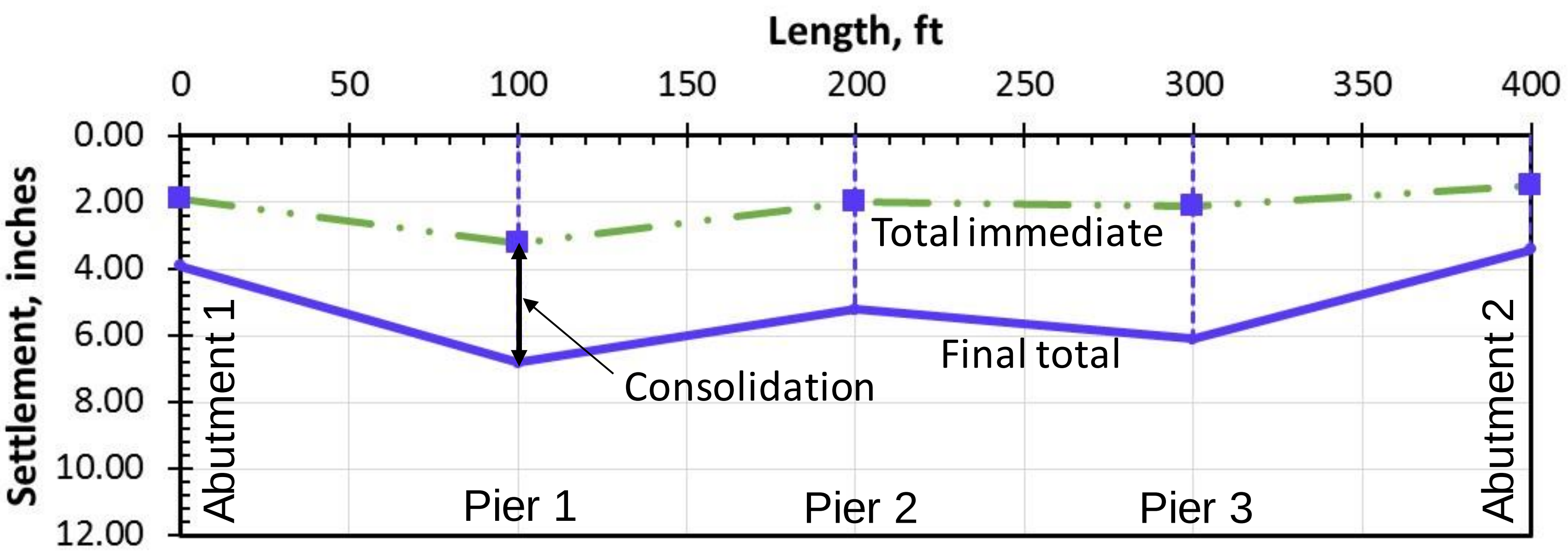
NOTE 1: The total immediate settlement is based on the assumption of instantaneous application of all loads while the relevant settlement is based on the assumption of loads due to superstructure only. With respect to Figure 6.1, the relevant immediate settlement is based on loads after the completion 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.

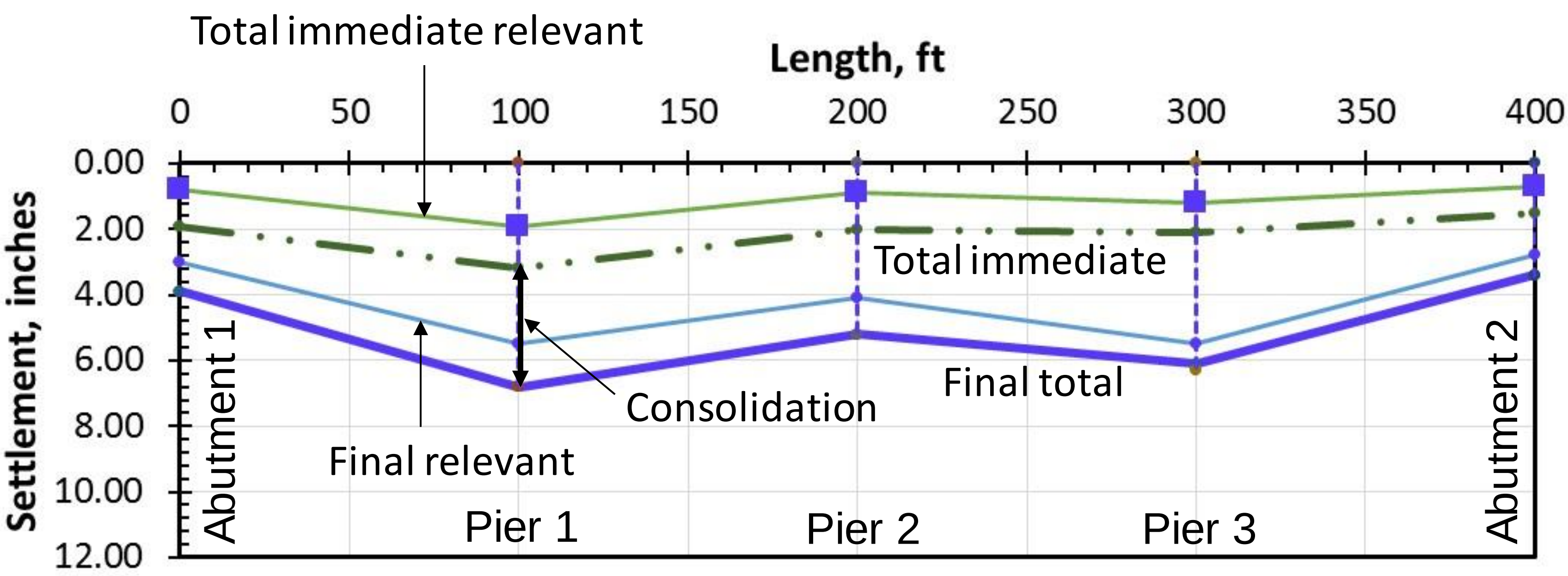
Settlement Profiles

- Profiles of total immediate and final settlement
- Consolidation settlement = Final settlement – Total immediate settlement



Settlement Profiles

- Total relevant immediate profile
- Final relevant settlement



Application of γ_{SE} Load Factor – Appendix B, Example

- For the data in the four previous slides, develop the factored total relevant settlement, S_f, values that will be used for bridge structural analysis

Application of γ_{SE} Load Factor – Appendix B, Example

- **Abutment 1**
  - γ_{SE} = 1.25 for Schmertmann method
  - γ_{SE} = 1.00 for consolidation settlement
  - Thus, S_f = (1.25)(0.80 in.) + (1.00)(2.00 in.) = 3.00 in.

Application of γ_{SE} Load Factor – Appendix B, Example

- **Pier 1**
  - γ_{SE} = 1.00 for Hough method
  - γ_{SE} = 1.00 for consolidation settlement
  - Thus, S_f = (1.00)(1.90 in.) + (1.00)(3.60 in.) = 5.50 in.

Application of γ_{SE} Load Factor – Appendix B, Example

- **Pier 2**
  - γ_{SE} = 1.00 for Hough method
  - γ_{SE} = 1.00 for consolidation settlement
  - Thus, S_f = (1.00)(0.90 in.) + (1.00)(3.20 in.) = 4.10 in.

Application of γ_{SE} Load Factor – Appendix B, Example

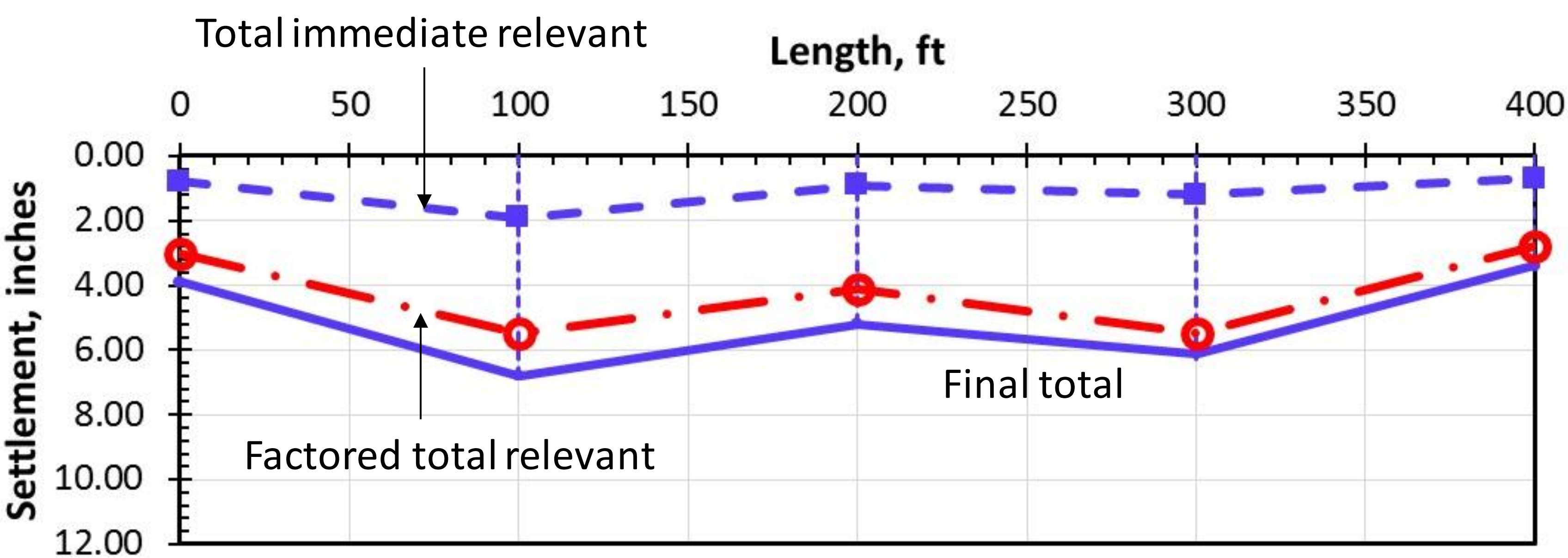
- **Pier 3**
  - γ_{SE} = 1.25 for Schmertmann method
  - γ_{SE} = 1.00 for consolidation settlement
  - Thus, S_f = (1.25)(1.20 in.) + (1.00)(4.00 in.) = 5.50 in.

Application of γ_{SE} Load Factor – Appendix B, Example

- **Abutment 2**
  - γ_{SE} = 1.25 for Schmertmann method
  - γ_{SE} = 1.00 for consolidation settlement
  - Thus, S_f = (1.25)(0.70 in.) + (1.00)(1.90 in.) = 2.78 in.

Application of γ_{SE} Load Factor – Appendix B, Example

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



Key Points

- Different values of the γ_{SE} load factor along a bridge structure depending on the method of analysis can be easily incorporated into the bridge design process

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Foundation Deformations

Lecture 9
White Paper Chapter 12 to Chapter 15, Appendix D, and Appendix E

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Foundation Deformations

Chapter 12 – The “S_f-0” Concept

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Learning Outcomes

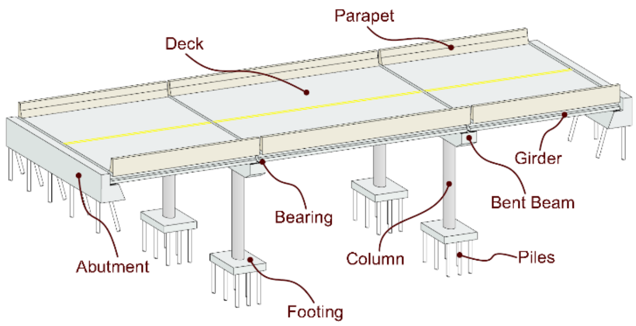
- Learn how to incorporate the concept of extreme values of differential settlements into bridge design process

- Introduce and explain the S_f-0 concept

Superimposed Deformations – Article 3.12.6

Article 3.12.6 – Settlement

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



Commentary

- "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."

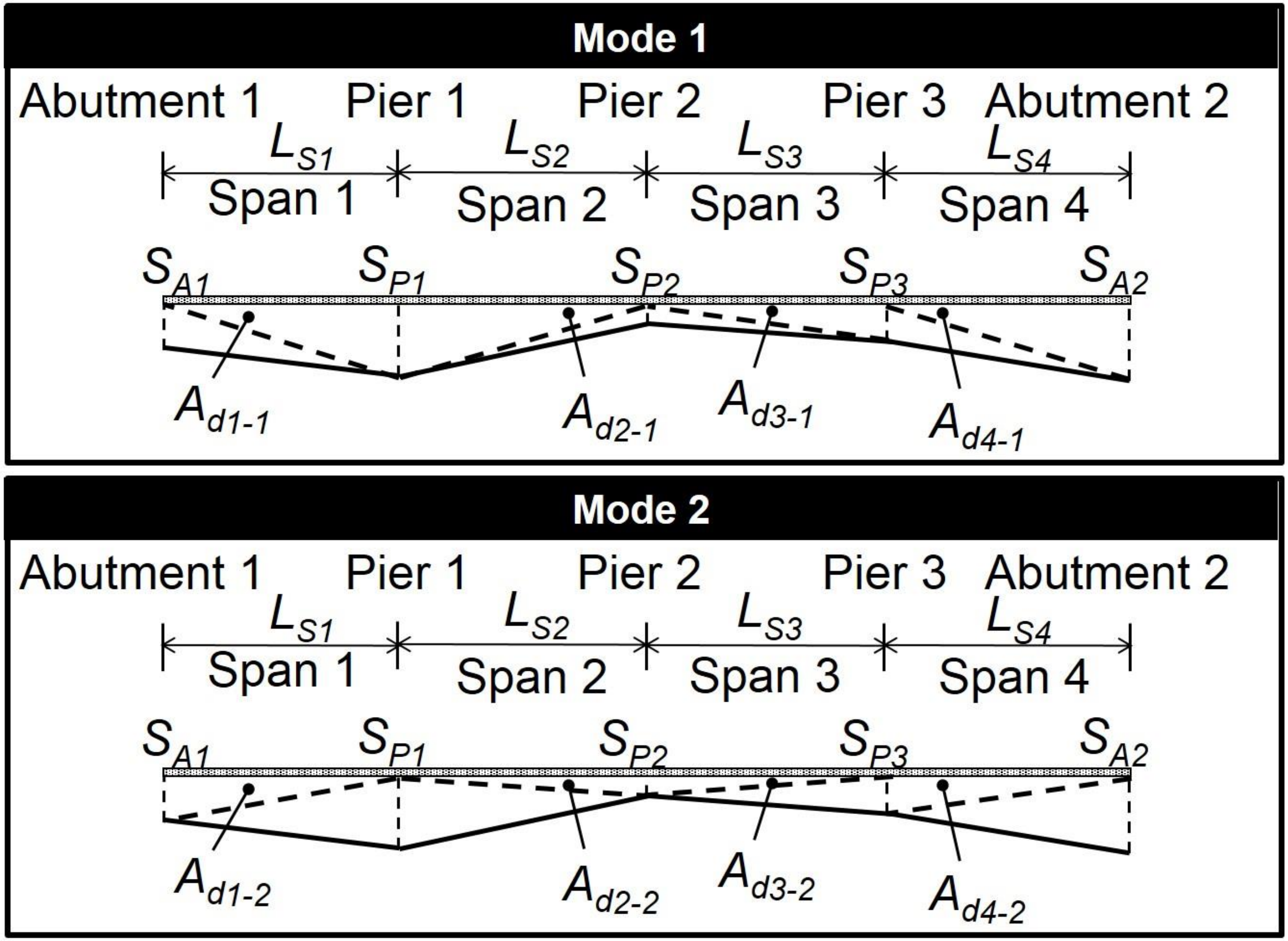
Underlying Basis for Use of Extreme Differential Settlement

- 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 the differential settlement

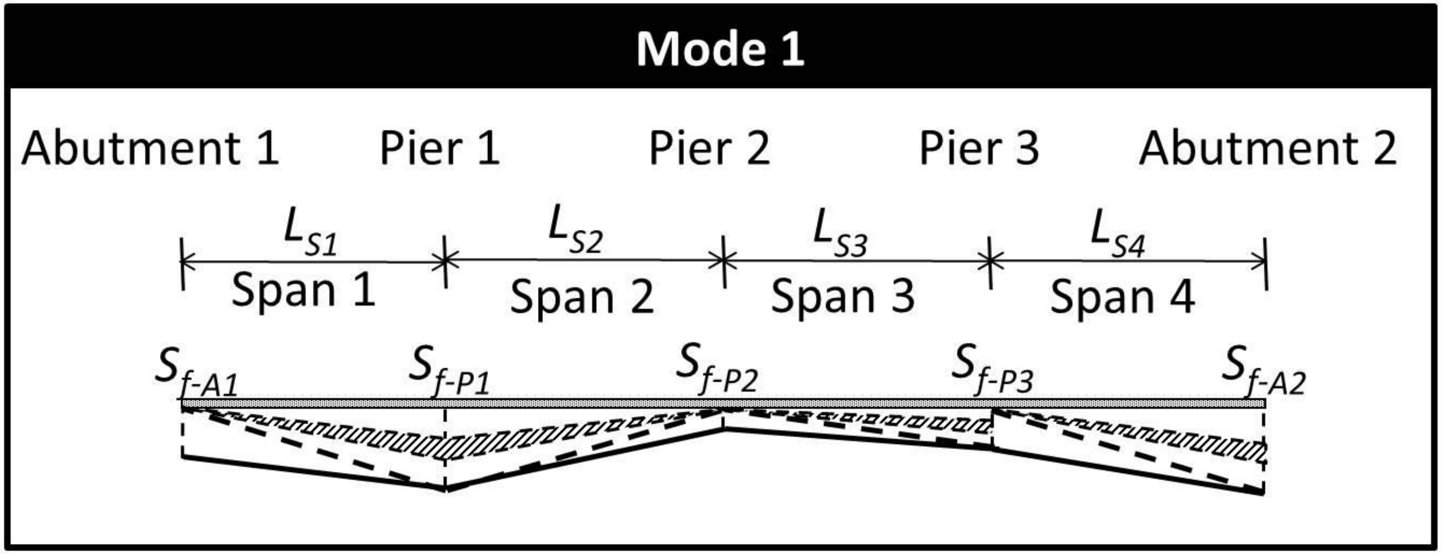
- Consideration of temporal and spatial uncertainties

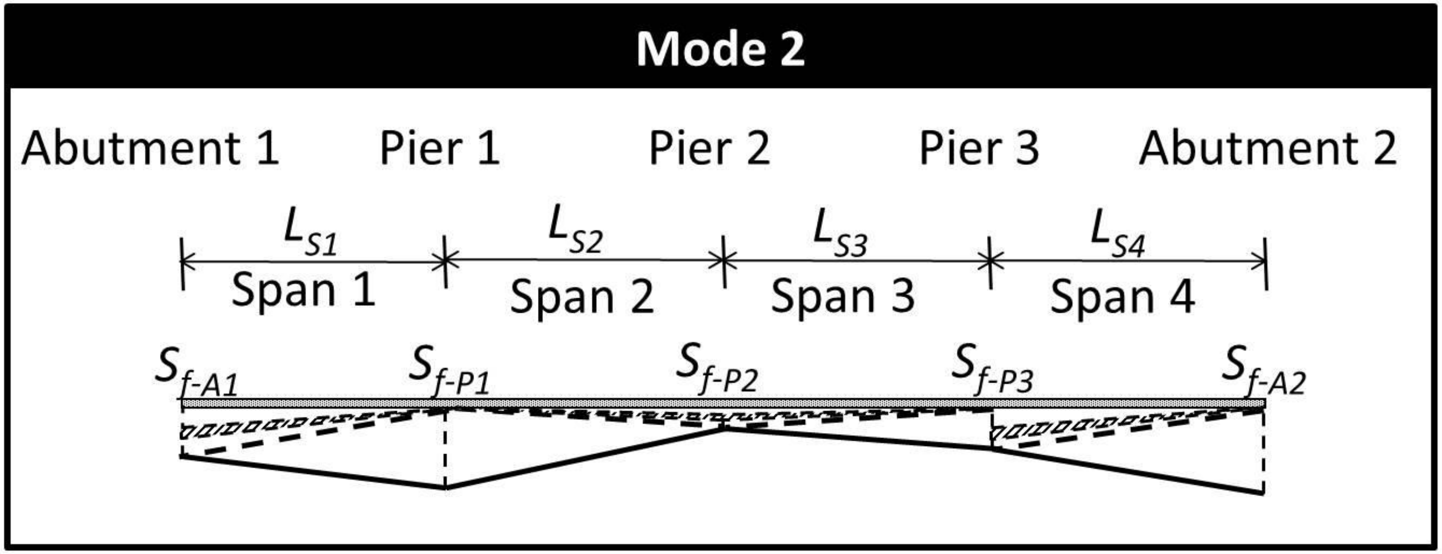
- Not all uncertainties associated with foundation deformations can be accounted for by a single load factor γ_{SE} for a certain model for prediction of deformation

Article 3.12.6 – Extreme Values and Combinations



Factored Angular Distortions Based on Construction-Point Concept





Key Points

- The *AASHTO LRFD* requirement to consider extreme values of differential settlements into bridge design process can be considered through the S_f-0 concept

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Foundation Deformations

Chapter 13 – Flow Chart to Consider Foundation Deformations in Bridge Design Process

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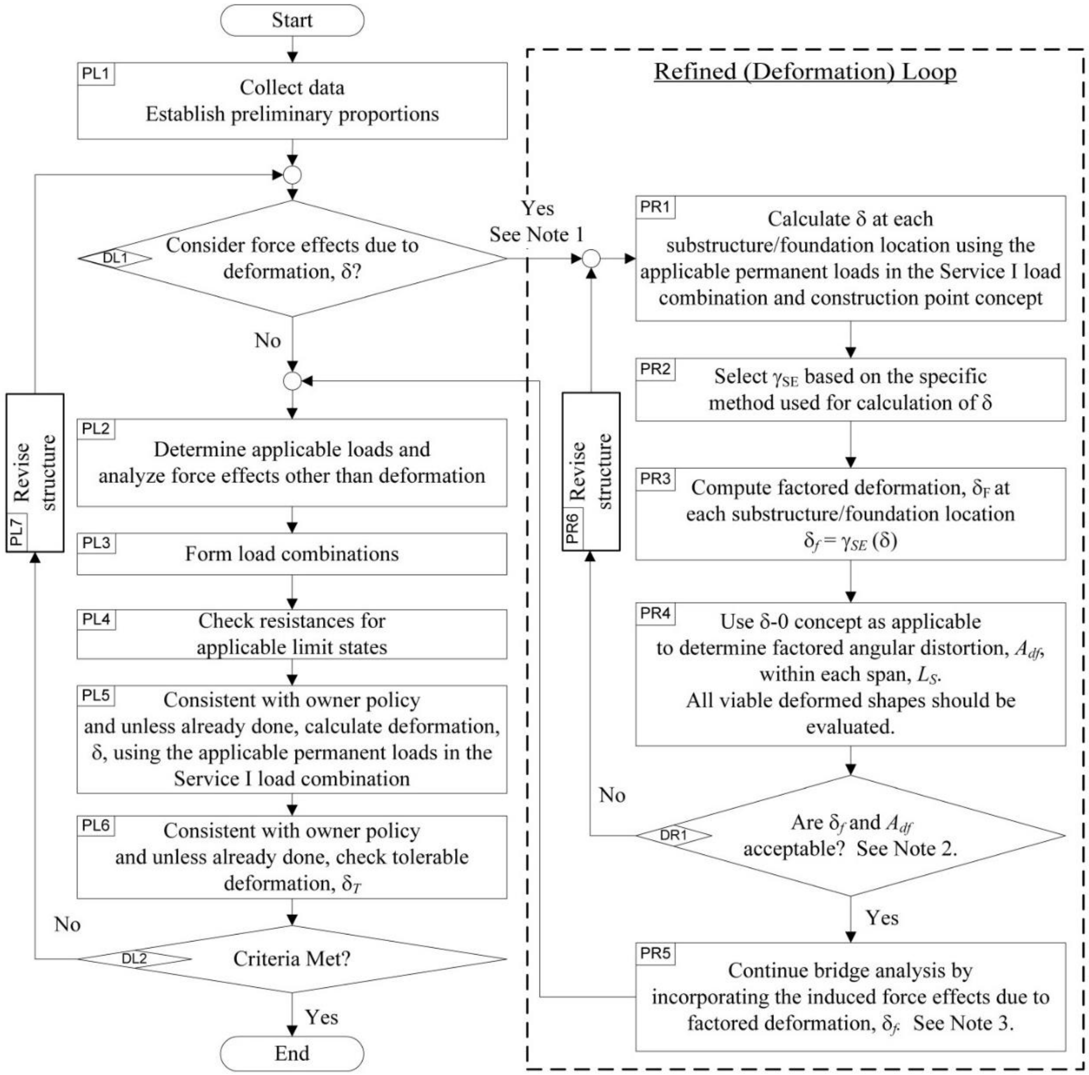
Learning Outcomes

- Introduce and explain a flow chart to incorporate the γ_{SE} load factor into the *AASHTO LRFD* bridge design proces.

Flow Chart

WP Figure 13-1



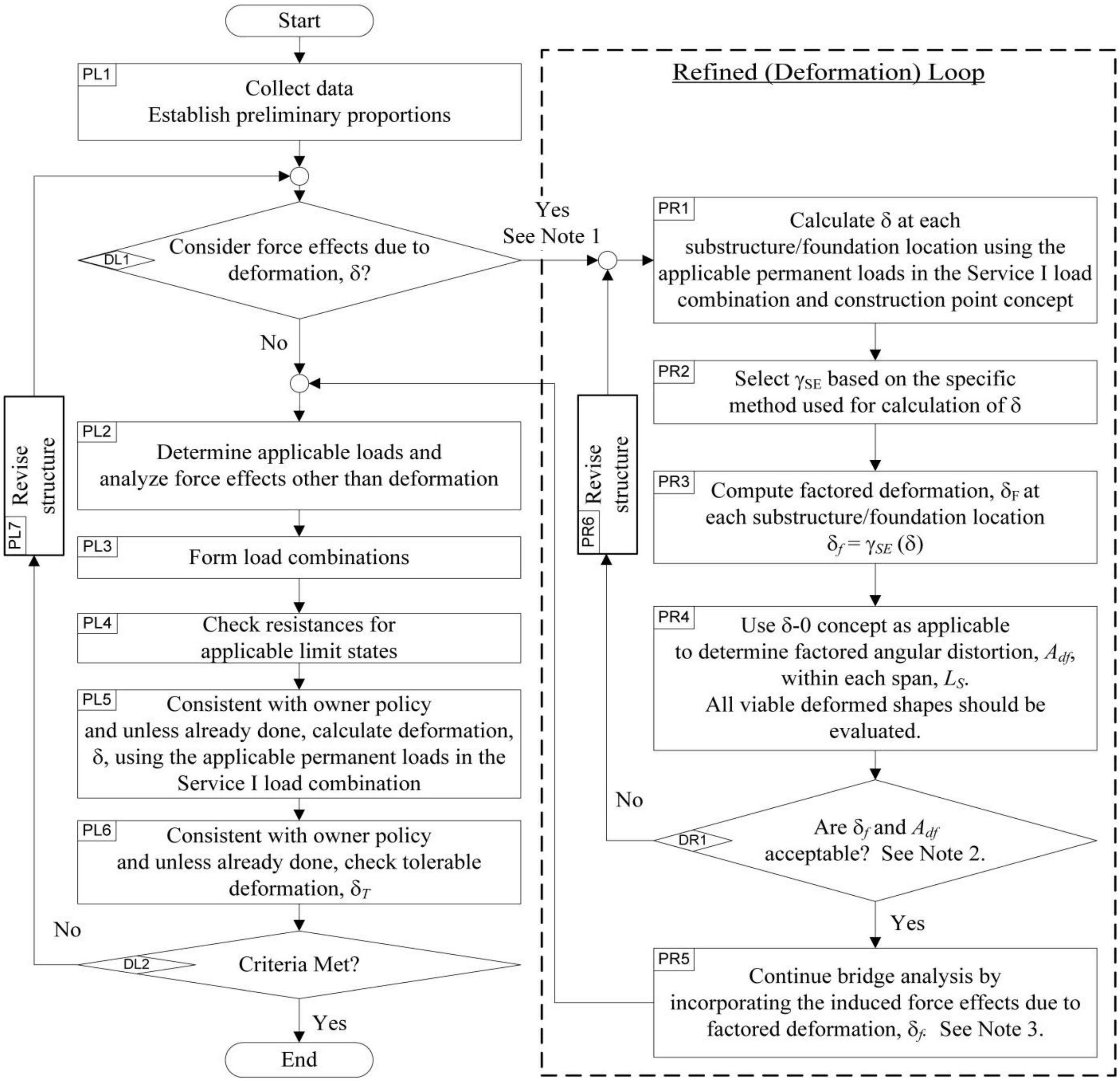
Note 1: It may be efficient to run some early design iterations without including this loop until the proportions of the bridge are well developed, and then include this loop to consider the force effects from differential deformations.

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.

Flow Chart

WP Figure 13-1



Key Points

- The γ_{SE} load factor can be incorporated into the *AASHTO LRFD* bridge design process in a streamlined manner

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Foundation Deformations

Chapter 14 – Modifications to AASHTO LRFD Bridge Design Specifications

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Learning Outcomes

- Discuss and understand the proposed changes to Section 3 of *AASHTO LRFD*

- Discuss and understand the proposed changes to Section 10 of *AASHTO LRFD*

Applicable *AASHTO LRFD* Sections

- Applicable sections in *AASHTO LRFD*
  - Section 3: Loads and Load Factors
  - Section 10: Foundations

- Section 3: Loads and Load Factors
  - Articles 3.4.1 and 3.12.6

- Section 10: Foundations
  - Article 10.5.2

Applicable AASHTO SCOBS Technical Committees

- SCOBS: Subcommittee on Bridges and Structures

- Applicable technical committees
  - T-5: Loads and load distribution
    - Responsible for Section 3 in *AASHTO LRFD*
  - T-15: Substructures and Retaining Walls
    - Responsible for Section 10 in *AASHTO LRFD*

Proposed Agenda Items for Balloting

- For T-5 technical committee
  - See Appendix D in White Paper for modifications to Section 3 of *AASHTO LRFD*
- For T-15 technical committee
  - See Appendix E in White Paper for modifications to Section 10 of *AASHTO LRFD*

Appendix D: Proposed Modifications To Section 3 of *AASHTO LRFD*

- Highlights
  - Modifications to Article 3.4.1 (“Load Factors and Load Combinations”)
    - New table of γ_{SE} load factors (Table 3.4.1-5)
    - Additional specifications
    - Additional commentaries

Appendix E: Proposed Modifications To Section 10 of *AASHTO LRFD*

- Highlights
  - Modifications to Article 10.5.2 (“Service Limit State”)
    - Additional specifications
    - Additional commentaries
  - Modifications to Article 10.6.2 (“Service Limit State Design”)
    - Add Schmertmann method
    - Additional specifications
    - Additional commentaries
  - Additional appendices
    - Appendix B10: Explain bridge design process with new provisions through use of a flow chart
    - Appendix C10: Explain construction-point and S_f-0 concepts

Key Points

- Changes will be proposed to Section 3 and Section 10 of *AASHTO LRFD*

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Foundation Deformations

Chapter 15 – Application of Calibration Procedures

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Learning Outcomes

- Discuss and understand the classes of problems that can be tackled by the calibration procedures in the White Paper

Application of Calibration Procedures

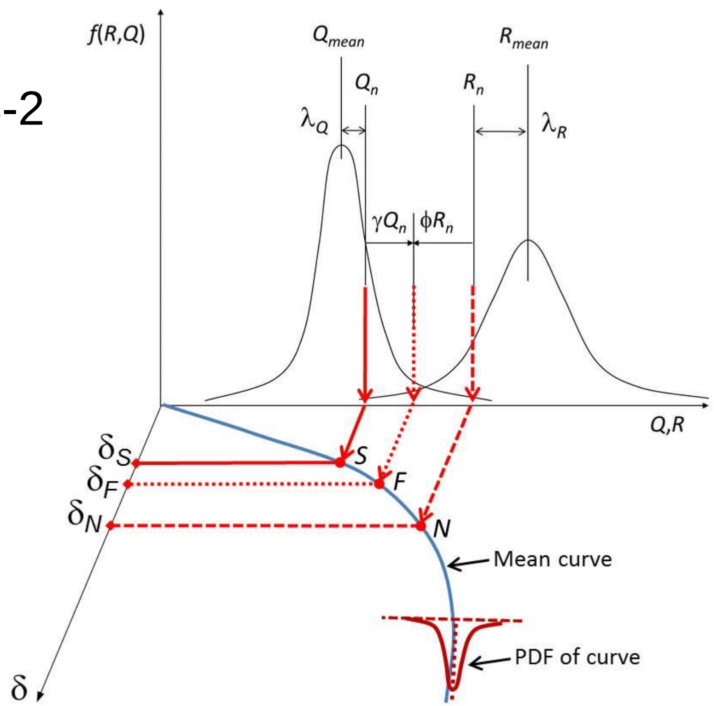
- Although the focus of the work is on calibration of foundation deformations, the calibration procedures are general and can be considered for calibration of any civil engineering feature

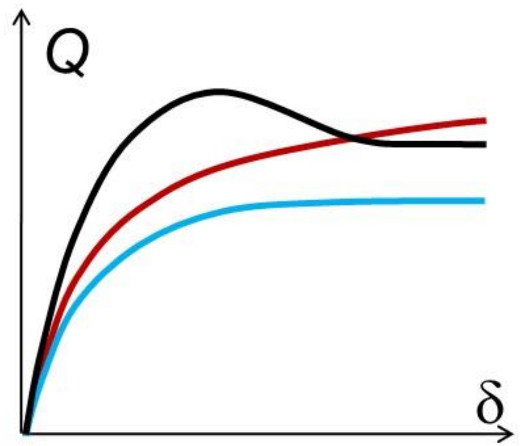
- Two classes of problems that can be treated using the calibration procedures for foundation deformations are:
  - Class A
  - Class B

Application of Calibration Procedures – Class A Problems

- Situations where consideration of deformations is required to inform the “two-hump” distributions of load and resistance

WP Figure 8-2

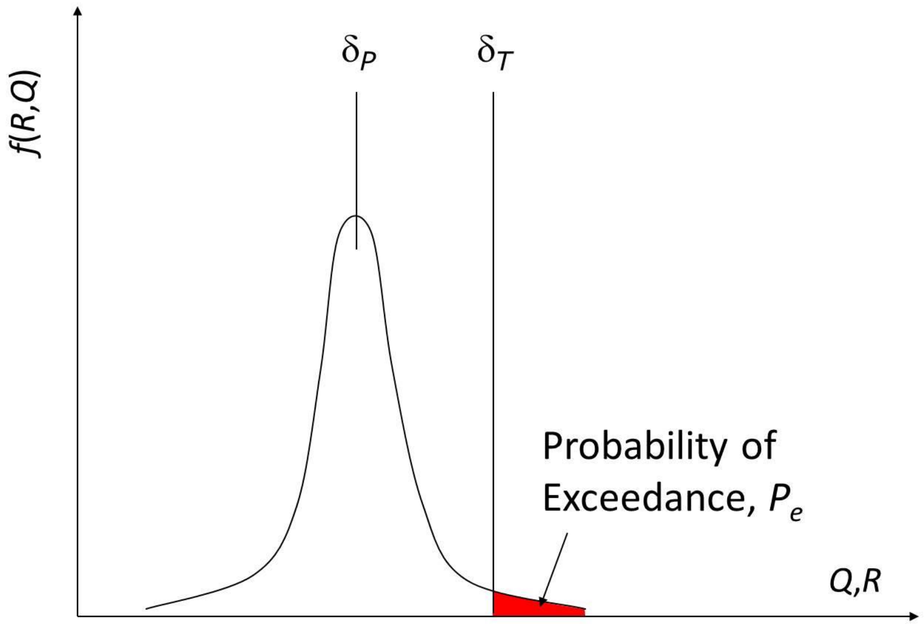




Application of Calibration Procedures – Class B Problems

- Situations where there is so little data on the distribution of either loads or resistances, or their proxies, that one needs to be considered as determinant, where there is no variability and Monte-Carlo simulation is unstable

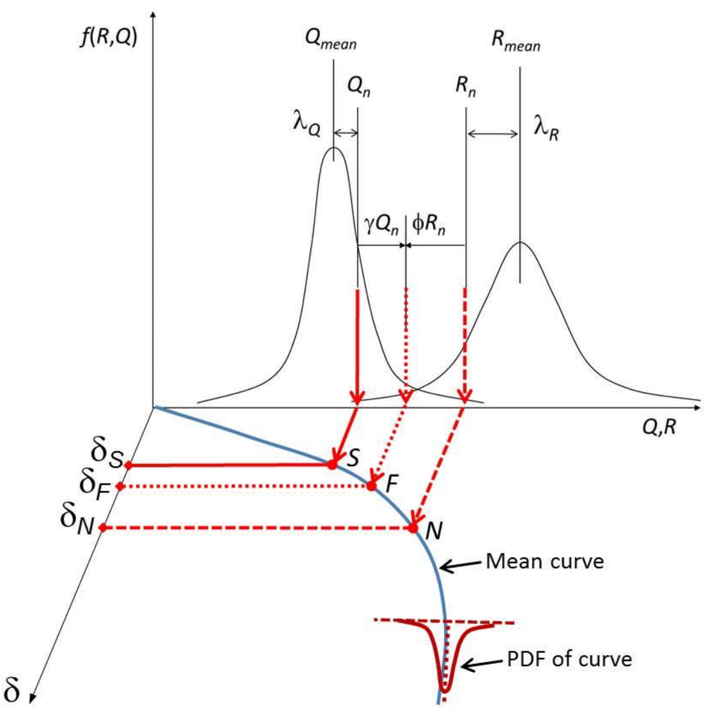
WP Figure 8-6



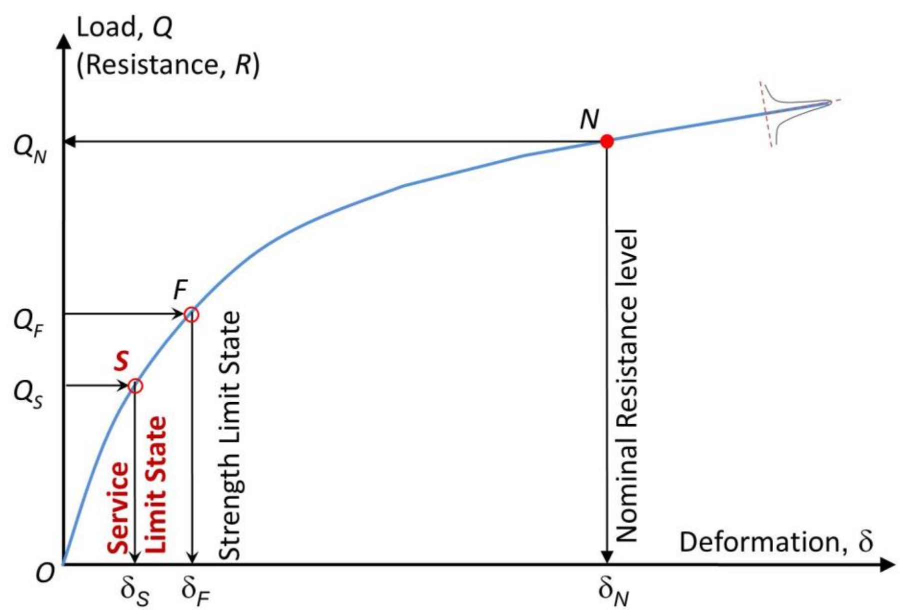
Application of Calibration Procedures

- Extension to strength limit state is also possible

WP Figure 8-2



WP Figure 8-3



Application of Calibration Procedures

- For development of γ_{SE} load factors for other types of deformations

- Some examples:
  - Lateral deformation of deep foundations
  - Face movements of MSE walls
  - Pullout resistance of soil reinforcements

Key Points

- The calibration procedures in the White Paper can be applied to problems beyond foundation deformations

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Foundation Deformations

Lecture 10
White Paper Appendix C - Examples

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Foundation Deformations

Example Problems for Foundation Deformations and Cost Considerations

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Learning Outcomes

- Demonstrate the application of the proposed changes in *AASHTO LRFD* by example problem(s)

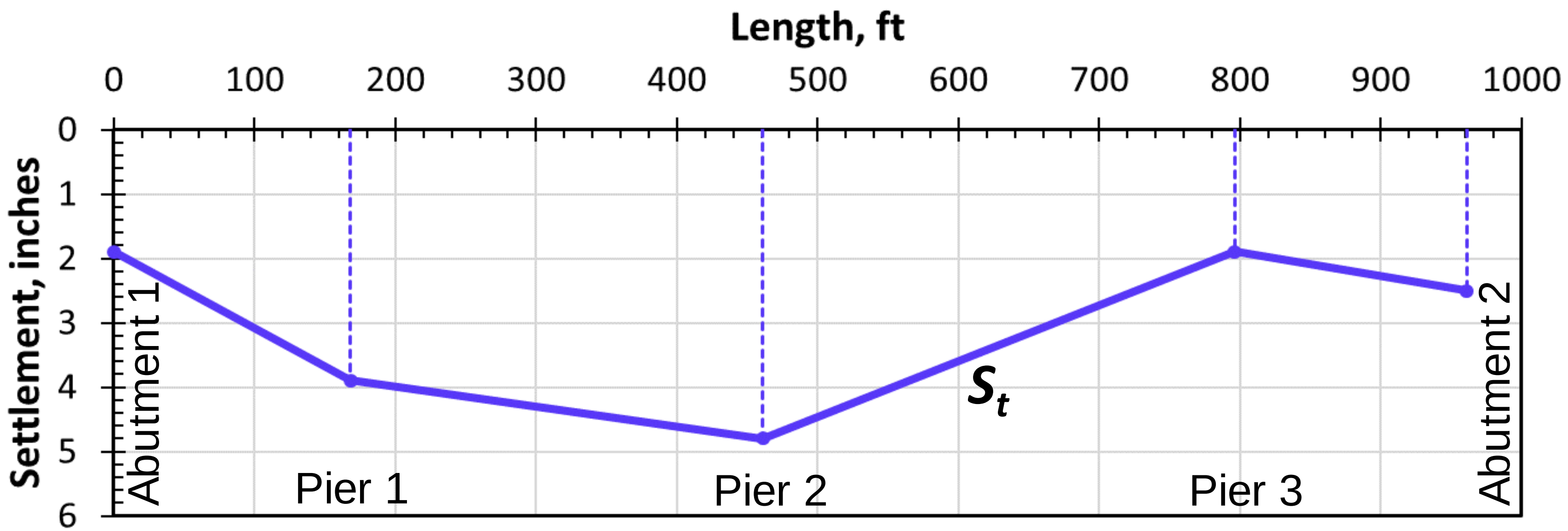
Impact on Bridge Design

- Three examples in Appendix C of White Paper
  - With input and assistance from Dr. Wagdy Wassef (AECOM)
- Example 1
  - Two span bridge, 100 ft long
  - Span lengths: 50 ft, 50 ft
- Example 2
  - Four span bridge, 961 ft long
  - Span lengths: 168 ft, 293 ft, 335 ft, 165 ft
- Example 3
  - Five span bridge, 660 ft long
  - Span lengths: 120 ft, 140 ft, 140 ft, 140 ft, 120 ft

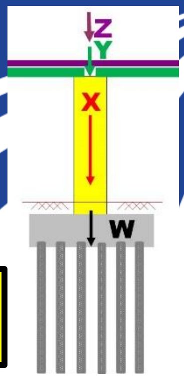
Predicted Unfactored Total Settlements, S_t (WP Table C-2)

S_t based on Service I load combination (TOTAL)

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

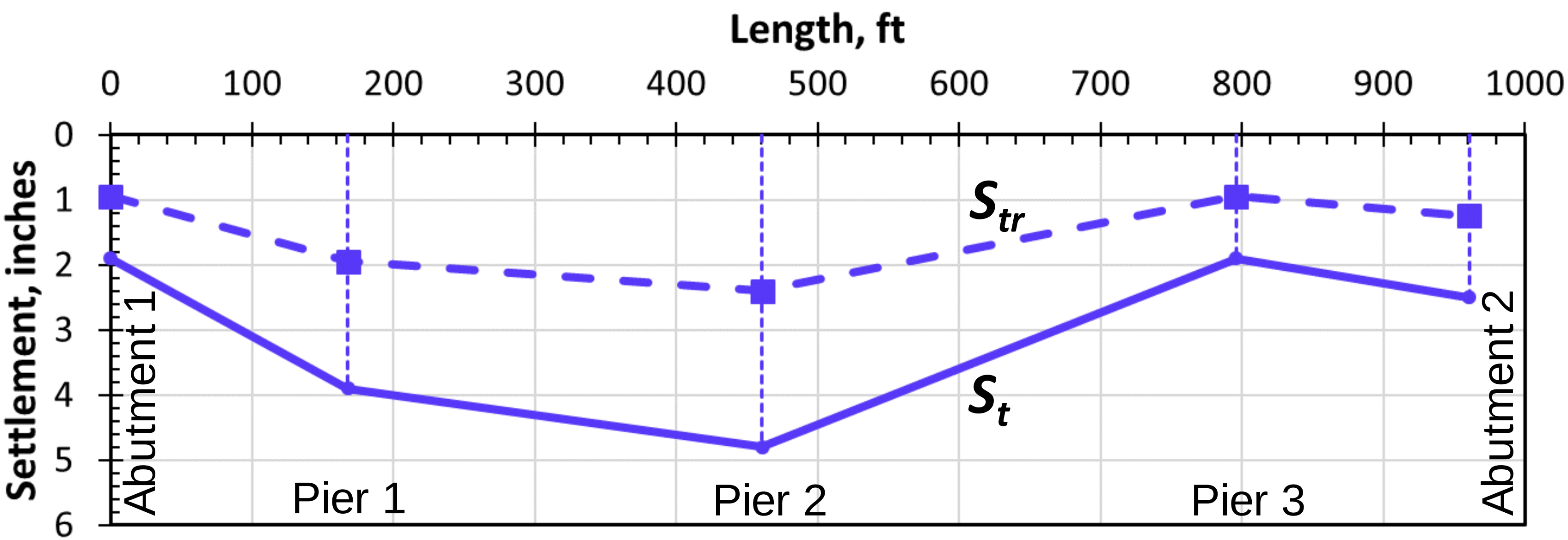


Estimated Unfactored Relevant Settlements, S_{tr} (WP Table C-3)



S_{tr} based on construction point concept

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

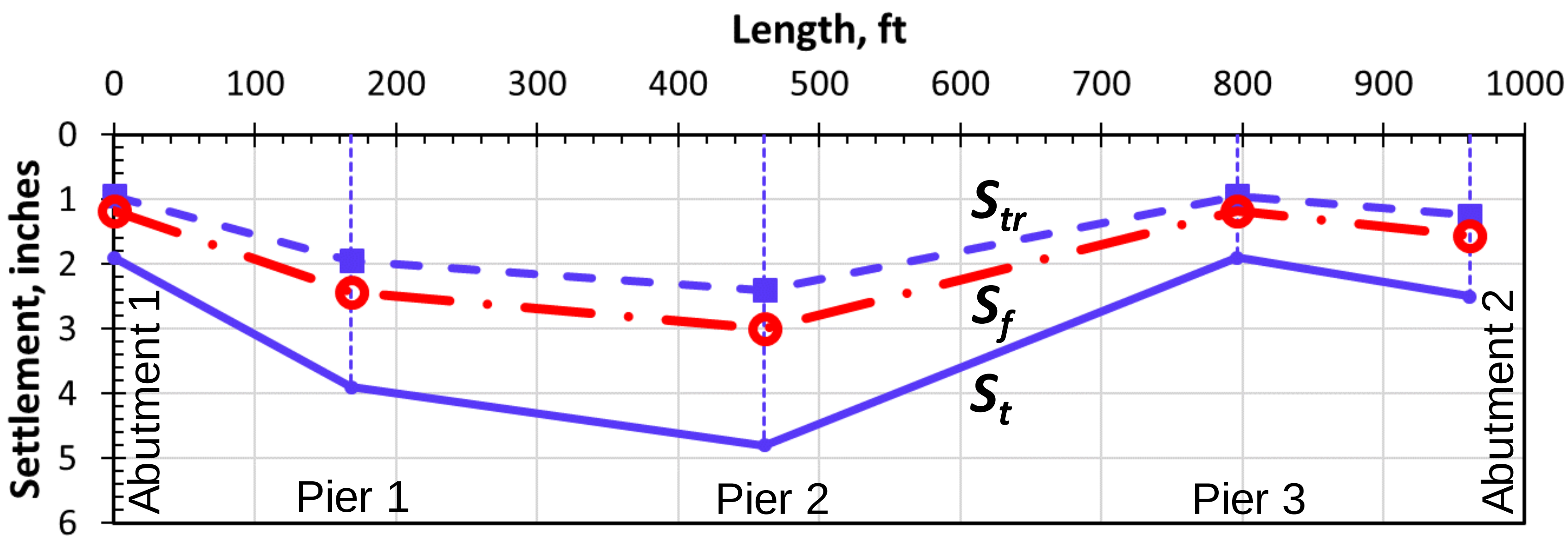


Note: For this example problem it is assumed that $S_{tr} = 0.5S_t$.

Factored Relevant Settlements, S_f (WP Table C-4)

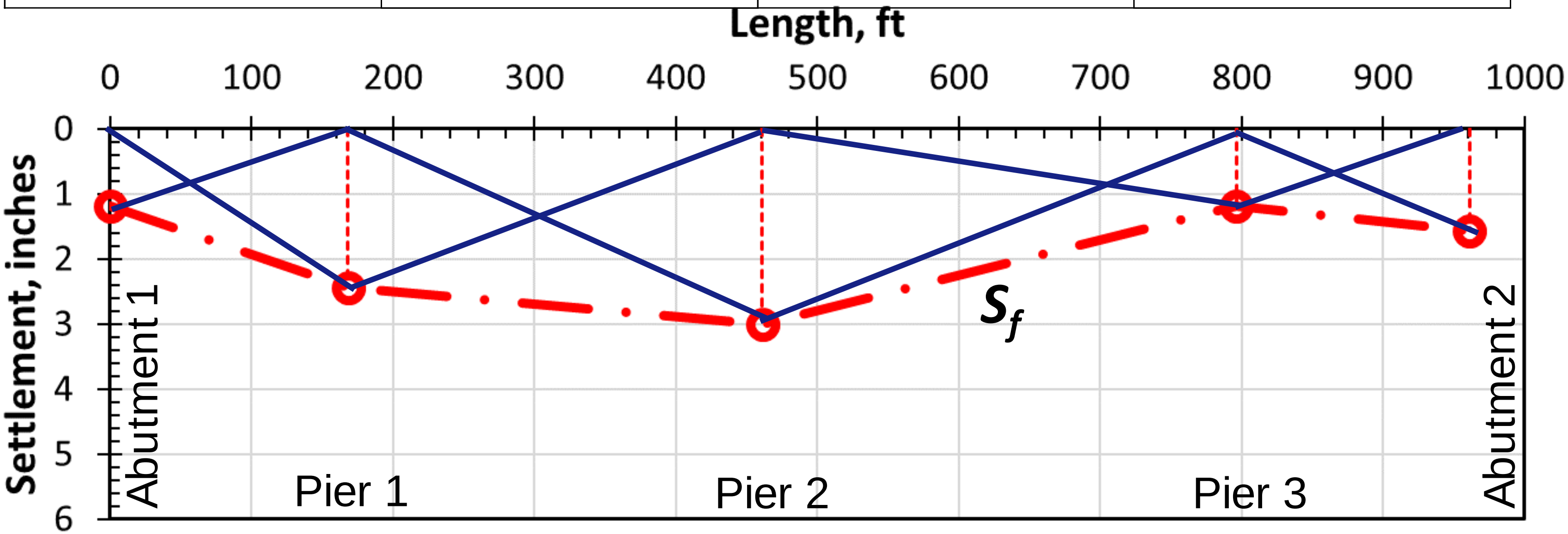
$$S_f = \gamma_{SE} (S_{tr})$$

Factored Relevant Settlements, S_f (in.) using γ_{SE} = 1.25				
Abutment 1	**Pier 1**	**Pier 2**	**Pier 3**	**Abutment 2**
1.19	2.44	3.00	1.19	1.56



Evaluate Factored Angular Distortions, A_{df} (WP Table C-4)

Factored Angular Distortion, A_{df} (rad.)			
Mode 1: S_f at the left end of the span divided by the span length			
Span 1	**Span 2**	**Span 3**	**Span 4**
0.0006	0.0007	0.0007	0.0006
Mode 2: S_f at the right end of the span divided by the span length			
Span 1	**Span 2**	**Span 3**	**Span 4**
0.0012	0.0009	0.0003	0.0008



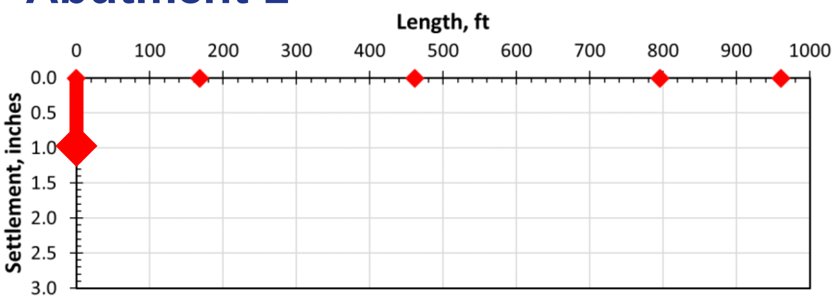
Example 2: Four-Span Bridge

WP Table E2-M1

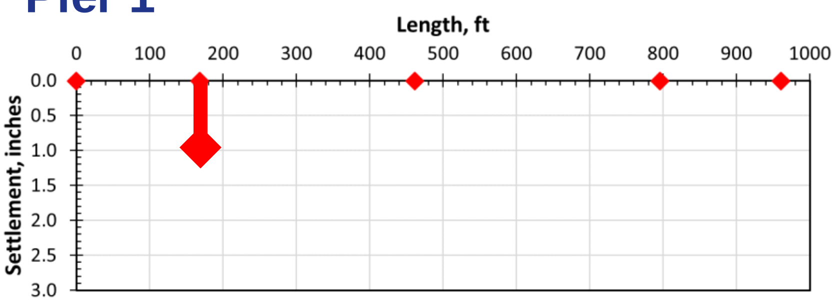
Moment (kip-ft)								
		Span 1 - 0.4L	Pier 1	Span 2 - 0.5L	Pier 2	Span 3 - 0.5L	Pier 3	Span 4 - 0.6L
Unfactored DL moment (No Settlement)		3884	-15561	8001	-33891	13513	-25824	1651
Unfactored LL moment	+ve	6401	2807	8639	1166	9741	2662	4379
	-ve	-3171	-10609	-3174	-13208	-2257	-14582	-2270
Unfactored 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

Unit Settlements at Supports

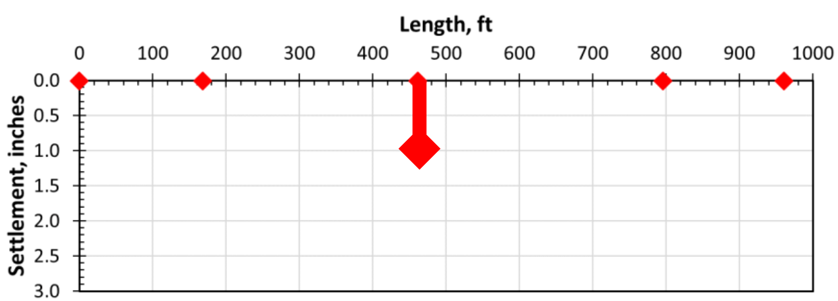
Abutment 1



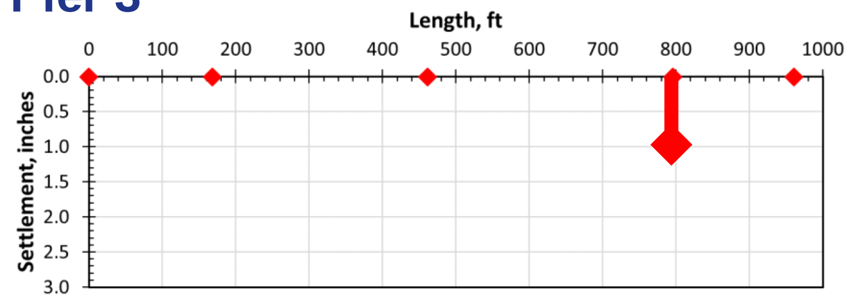
Pier 1



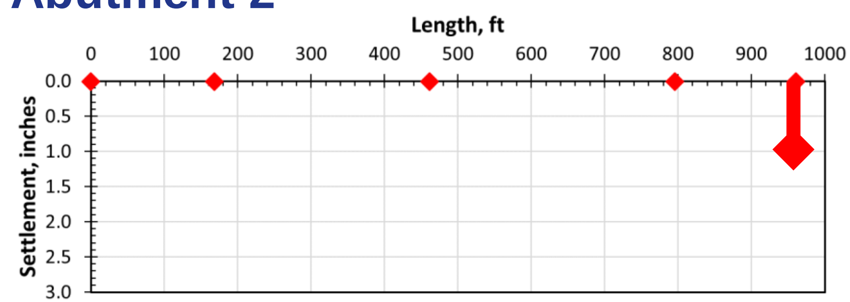
Pier 2



Pier 3



Abutment 2



- Use linear scaling and superposition to develop force effects (moments and shears) due to settlements

Example 2: Four-Span Bridge

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

Example 2: Four-Span Bridge
Linear Scaling of Values

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

S_{tr} at Abutment 1 = 0.95 in.
Unfactored effect of 1 in. settlement at Abutment 1 on Pier 1 = -822 kip-ft
Effect of unfactored S_{tr} at Abutment 1 on Pier 1 = (0.95 in./1.00 in/)(-822 kip-ft) = -781 kip-ft

Example 2: Four-Span Bridge
Linear Scaling of Values

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

S_{tr} at Pier 2 = 2.40 in.
Unfactored effect of 1 in. settlement at Pier 2 on Span 3-0.5L = 344 kip-ft
Effect of unfactored S_{tr} at Pier 2 on Span 3-0.5L = (2.40 in./1.00 in/)(344 kip-ft) = 826 kip-ft

Example 2: Four-Span Bridge
Linear Scaling of Values

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

S_{tr} at __________ = ______ in.
Unfactored effect of 1 in. settlement at _____________ on Pier 2 = _______ kip-ft
Effect of unfactored S_{tr} at ___________ on Pier 2 = (___ in./1.00 in/)(___ kip-ft) = ___ kip-ft

Example 2: Four-Span Bridge

Total +ve Effect Due to γ_{SE} =1.00 and S_{tr}

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

For total +ve effect, sum only the +ve values at each support, i.e., do not consider –ve values
+ve values at Pier 1 occur due to effect of settlement at Pier 1 and Pier 3
+ve value: 3418 kip-ft + 429 kip-ft = 3848 kip-ft

Example 2: Four-Span Bridge

Total -ve Effect Due to γ_{SE} =1.00 and S_{tr}

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

For total -ve effect, sum only the -ve values at each support, i.e., do not consider +ve values

-ve values at Span 3-0.5L occur due to effect of settlement at Pier 1 and Abutment 2

-ve value: -314 kip-ft - 734 kip-ft = -1048 kip-ft

Example 2: Four-Span Bridge

Total +ve Effect Due to γ_{SE} =1.00 and S_t

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

<u>For total +ve effect, sum only the +ve values at each support, i.e., do not consider –ve values</u>

+ve values at Pier 1 occur due to effect of settlement at Pier 1 and Pier 3 based on S_t (= $2S_{tr}$)

+ve value: 2(3848 kip-ft) = 7696 kip-ft

Example 2: Four-Span Bridge

Total +ve Effect Due to γ_{SE} =1.25 and S_{tr}

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

For total +ve effect, sum only the +ve values at each support, i.e., do not consider –ve values
+ve values at Pier 1 occur due to effect of sett at Pier 1 and Pier 3 based on γ_{SE}=1.25 and S_{tr}
+ve value: 1.25(3848 kip-ft) = 4810 kip-ft

Cases for Evaluation

- Case 1
  - Not consider settlement
- Case 2:
  - Current AASHTO
  - Consider full settlement with $\gamma_{SE} = 1.0$
- Case 3
  - Proposed specifications
  - Consider uncertainty in settlement (i.e., use appropriate γ_{SE}) and construction-point concept

Example 2: Four-Span Bridge

Case 1: Not Consider Settlement

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

1.00 DL + 1.00 LL = 3884 kip-ft + 6401 kip-ft = 10285 kip-ft

Example 2: Four-Span Bridge

Case 2: S_t with γ_{SE} = 1.00

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

1.00 DL + 1.00 LL + 1.00 using S_t = 3884 kip-ft + 6401 kip-ft + 3103 kip-ft = 13388 kip-ft

Example 2: Four-Span Bridge

Case 3: S_{tr} with γ_{SE} = 1.25

WP Table E2-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 γ_{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 γ_{SE} = 1.25 and S_{tr}	+ve	*1939*	*4810*	*1830*	*4395*	*1513*	*2951*	*593*
	-ve	*-1926*	*-4823*	*-1130*	*-2975*	*-1310*	*-3966*	*-790*

1.00 DL + 1.00 LL + 1.25 using S_{tr} = 3884 kip-ft + 6401 kip-ft + 1939 kip-ft = 12224 kip-ft

Service I Comparison

Case 1: Not consider settlement
Case 2: Consider full settlement with γ_{SE} = 1.0 (current AASHTO)
Case 3: Consider uncertainty in settlement and construction-point concept

WP Table E2-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 + γ_{SE} SE (use γ_{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 + γ_{SE} SE (use γ_{SE} = 1.25 and S_{tr})	Max	12224	-7944	18470	-28330	24767	-20211	6623
	Min	-1213	-30993	3697	-50074	9946	-44372	-1409
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*

Comments

- Ratio of Case 3 to Case 2 represents the change in specifications
  - Force effects are reduced since ratio < 1

- Ratio of Case 3 to Case 1 represents considering settlement as proposed in contrast to not considering settlement

- **The governing case does not change**

Strength I Comparison

Case 1: Not consider settlement
Case 2: Consider full settlement with γ_{SE} = 1.0 (current AASHTO)
Case 3: Consider uncertainty in settlement and construction-point concept

WP Table E2-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 + γ_{SE} SE (use γ_{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 + γ_{SE} SE (use γ_{SE} = 1.25 and S_{tr})	Max	17996	-9729	26949	-35928	35451	-24670	10320
	Min	-2620	-42840	3317	-68453	11632	-61765	-2699
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*

Comments

- Ratio of Case 3 to Case 2 represents the change in specifications
  - Force effects are reduced since ratio < 1

- Ratio of Case 3 to Case 1 represents considering settlement as proposed in contrast to not considering settlement

- **The governing case does not change**

Benefits of Using Calibrated Foundation Deformations

- Consideration of calibrated foundation deformations in the bridge design process can lead to use of cost-effective structures with more efficient foundation systems
  – Permits enhanced use of cost-effective spread footings and true bridge abutments (spread footing on top of MSE wall)
- The proposed revisions provide a more rational basis on which to compare alternatives

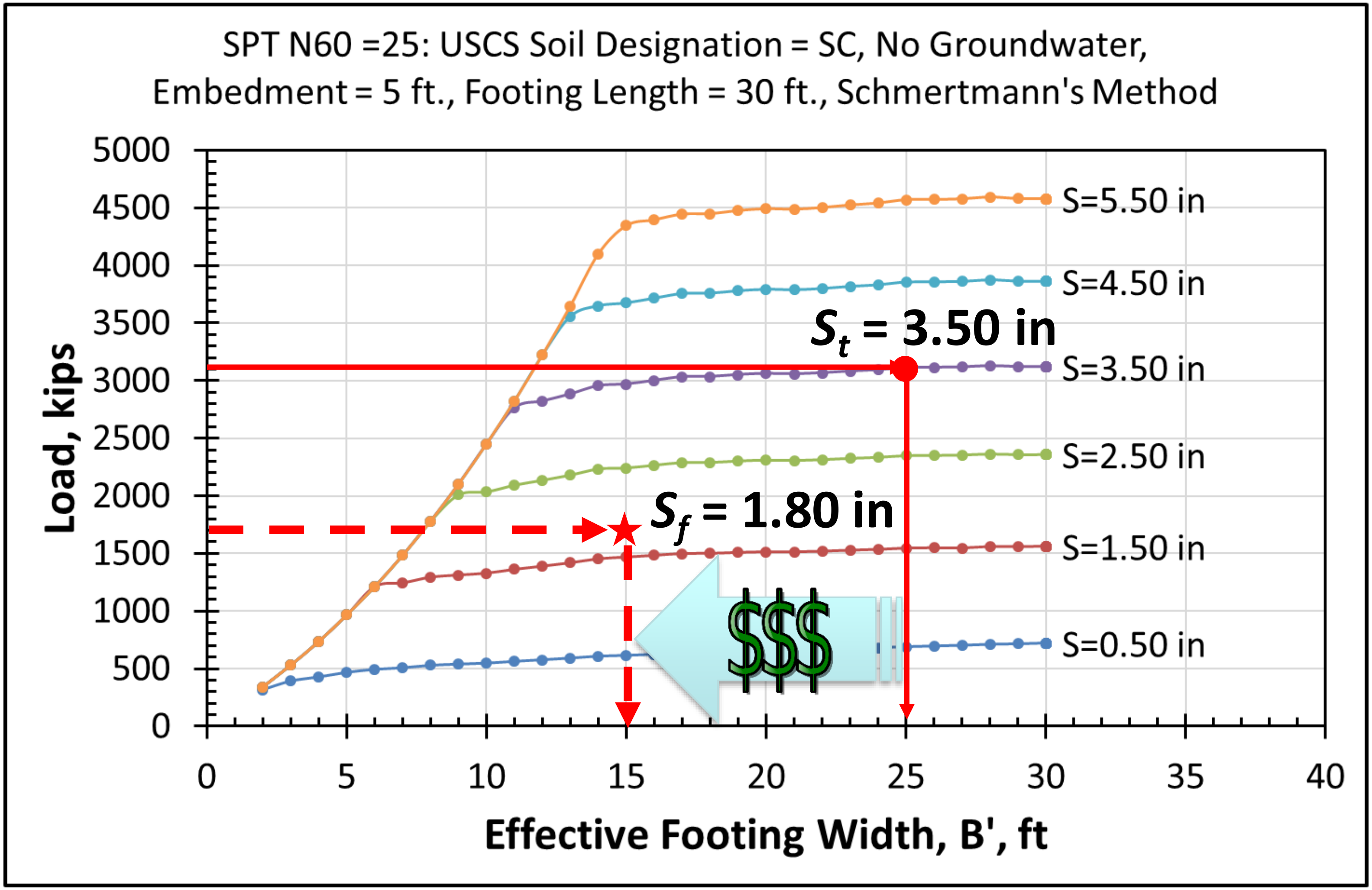
Benefits of Using Calibrated Foundation Deformations

- 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).

Example of Foundation Efficiency

- Subsurface conditions
  - Soil: Clayey Sand (USCS soil designation: SC)
  - No groundwater
  - SPT N60 value: 25
- Footing
  - Depth of embedment: 5 ft
  - Footing length: 30 ft
- Method of settlement analysis
  - Schmertmann
- Total load at bottom of footing: 3100 kips
- Load due to superstructure: 1700 kips

Example of Foundation Efficiency



FHWA Resources

APPENDIX E – LRFD GUIDANCE FOR SPREAD FOOTINGS

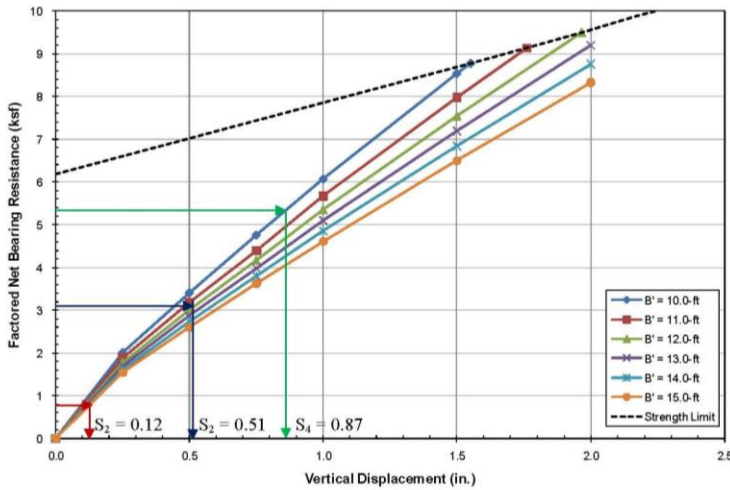


Figure E.4-2. Graph. Example of a factored bearing resistance chart in terms of stress-settlement curves for a range of effective footing widths.

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

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

SELECTION OF SPREAD FOOTINGS ON SOILS TO SUPPORT HIGHWAY BRIDGE STRUCTURES

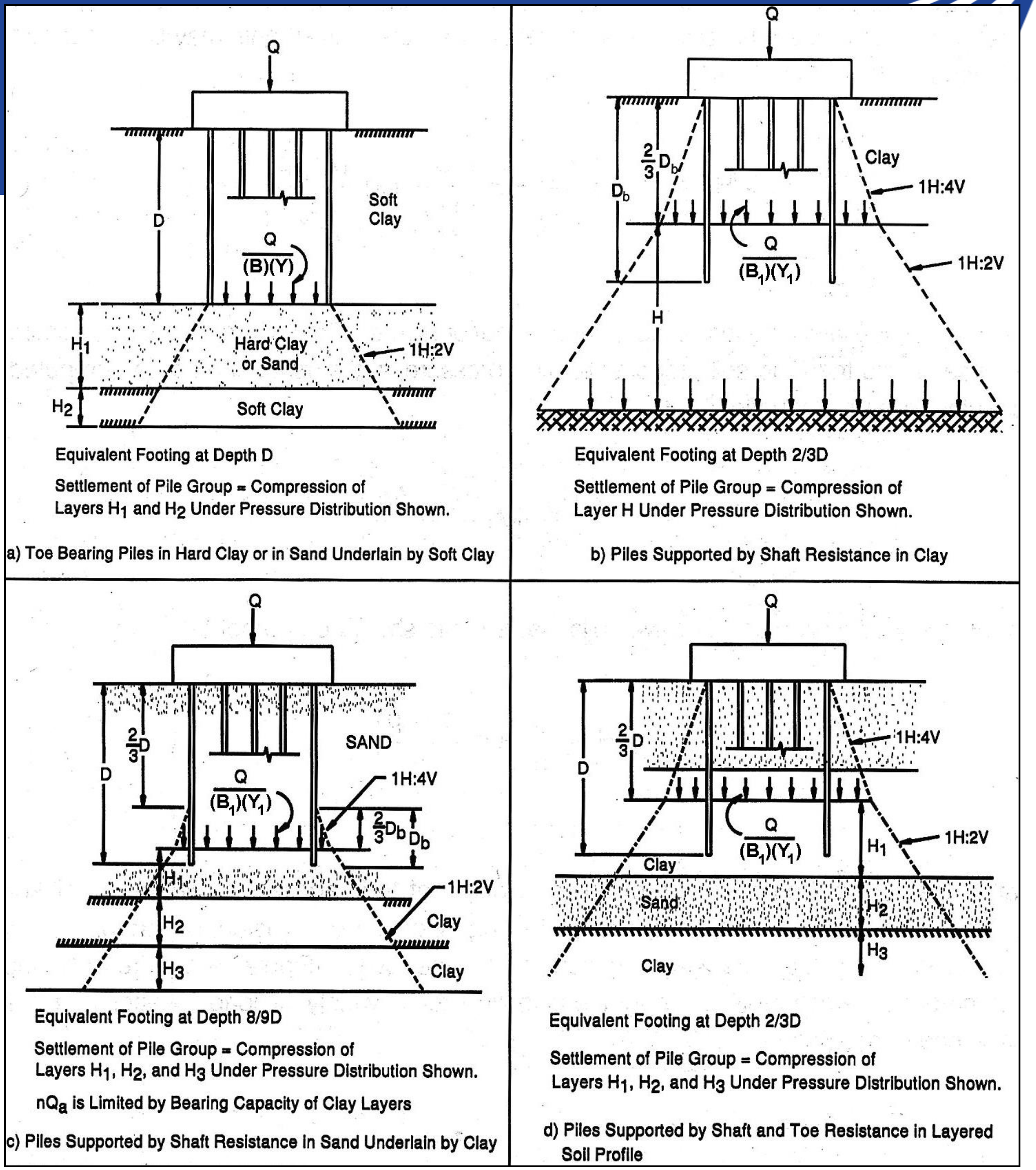
Publication No. FHWA-RC/TD-10-001 February 2010

U.S. Department of Transportation
Federal Highway Administration

http://www.ncsgeoresources.com/downloads/

Settlement of Deep Foundations

- Article 10.7.2.3
  - Use equivalent footing
- Can reduce:
  - length of deep foundations
  - plan size of deep foundation system
  - number of deep foundation elements in a group



Closing Comments

- Consideration of foundation deformations in bridge design is not new – **it is, in fact, required by specifications**
- The uncertainty in predicted deformations can now be quantified through the mechanism of SE load factor, γ_{SE}
- The calibration process is general and can be applied to any foundation or wall type and any type of deformation
  - Microsoft Excel®-based calibration processes have been developed
- Proposed LRFD specification revisions and commentaries have been developed
- Significant cost efficiencies can be realized

Key Points

- The proposed changes in *AASHTO LRFD* are unlikely to lead to significant changes in superstructure member sizes

- The application of the proposed changes are likely to lead to cost savings through use of cost-effective structures with more efficient foundation systems