Service Limit State Design for Bridges

Background information – Research Overview and Implementation into the AASHTO Specifications

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AECOM

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U.S. Department of Transportation
Federal Highway Administration

American Association of State Highway and Transportation Officials
AASHTO
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TRB

Initial SHRP2 Research

- General calibration process was developed for Service Limit State (SLS) Design
- Research conducted under auspices of Transportation Research Board (TRB)
  - Through second Strategic Highway Research Program (SHRP2)
  - Through National Cooperative Highway Research Program (NCHRP)
- Research Title: *Durable Bridges for Service Life beyond 100 Years: Service Limit State Design* (R19B)

TRB SHRP2 Research Team

SHRP2 Research Leads Working on Service Limit State Calibration:

- John Kulicki, Ph.D., P.E. (PI, SHRP R19B)
- Wagdy Wassef, Ph.D., P.E. (PI NCHRP 12-83)
- Dennis Mertz, Ph.D., P.E.
- Andy Nowak, Ph.D.
- Naresh Samtani, Ph.D., P.E. (R19B only)
- Hani Nassif, Ph.D., P.E. (NCHRP 12-83 only)

Initial Work under Research Phase

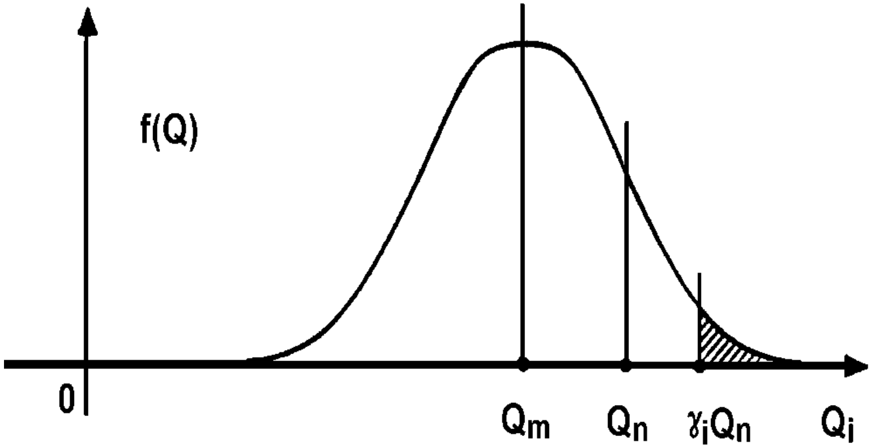
- General calibration process was developed for SLS and was revised to fit specific requirements for different limit states.
- The following limit states were calibrated:
  - Fatigue I and Fatigue II limit states for steel components.
  - Fatigue I for compression in concrete and tension in the reinforcement.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

General Calibration Process

- Loads and resistance vary, as well as most other things in life.
- Failure is assumed when the loads exceed the resistance.
- Low probability of failure constitutes acceptable design.
- For design purposes, assumptions regarding the value of the loads, the resistance (load capacity) and the acceptable margin of safety are made to produce acceptable designs (low probability of failure).

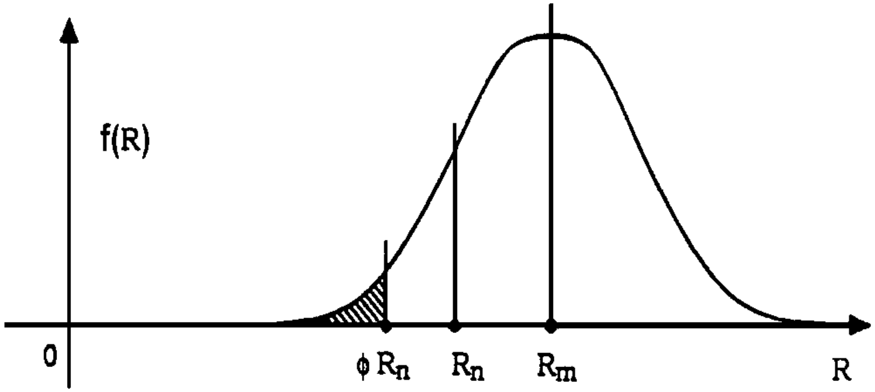
General Calibration Process

- Typically, normal distribution is used.
- For design purposes to achieve low probability of the design loads being exceeded:
  - o Nominal value is assumed higher than the mean.
  - o The loads are then factored by a "load factor" generally greater than 1.0.



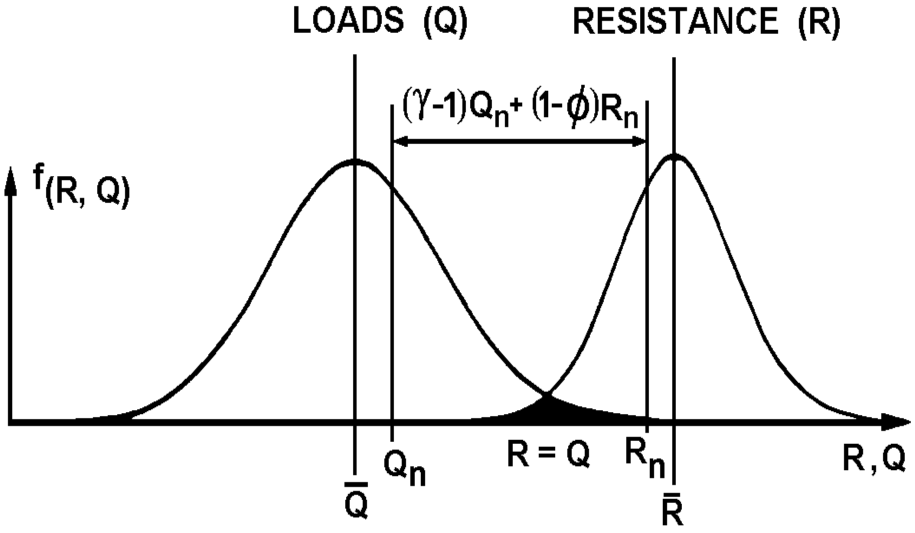
General Calibration Process

- The resistance is treated similar to the loads, but opposite in logic.
  - Nominal value is assumed lower than the mean.
  - The resistance is then factored by a "resistance factor" generally less than 1.0.



General Calibration Process

- The nominal values and the load and resistance factors are selected such that the probability of the factored loads exceeding the factored resistance, as represented by the shaded area, is acceptably small.



General Calibration Process

For the **Strength** limit state:

- The consequences of loads exceeding the resistance are relatively clear and typically make the bridge unsafe for use.
- The frequency of exceedance should be kept to an extreme minimum.

General Calibration Process

For the **Service** limit state:

- The consequences of the loads exceeding the resistance are not well defined (exceeding deflection limit, wider cracks in reinforced concrete (RC), exceeding stress limit in PSC, etc.).
- Limit states may be exceeded but the acceptable frequency of exceedance is not known.

Research, *continued*

- The general calibration process and procedure were revised to fit specific requirements for different limit states.
- A study of weight-in-motion (WIM) data from different sources was performed to determine the appropriate loads.

Review of Live Load Model Development

- 60 million truck WIM data records were obtained. After filtering, about 35 million records were used.
- Eliminated records included erroneous records, light vehicles, a site that has a large number of extremely heavy vehicles and one state that used different format.
- Three studies:
  - Fatigue I
  - Fatigue II
  - Other limit states

Conclusion For Non-Fatigue Service Limit States

- For all limit states except fatigue: Not necessary to envelope all trucks – SLS is expected to be exceeded occasionally.
- Site/region specific live load should be accommodated.
- Some states with less weight enforcement may need additional consideration.
- **Other than for fatigue loading, results have been generally accepted by the bridge community.**

Design Cycles Per Truck (Fatigue II)

Current

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

Proposed

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

Fatigue I

- Usually assumed that constant-amplitude fatigue threshold (CAFT) can be exceeded by 1/10,000 of the stress cycles.
- 99.99% inclusion of normal random variables requires mean plus 3.8 standard deviation.

Fatigue

- From the WIM data study, proposed load factors for live load:

Limit State	Mean	COV
Fatigue I	2.0 (currently 1.5)	0.12
Fatigue II	0.8 (currently 0.75)	0.07

Fatigue

- Uniform reliability can be achieved using variable resistance factors or change in constants used in the calculations.

- The latter approach is consistent with the philosophy of the specifications.

Calibration For Fatigue In Structural Steel

- Uniform reliability can be achieved using variable resistance factors or change in constants used in calculations as shown below.
- The latter approach is consistent with the philosophy of the specifications.

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	11
E′	3.9	**3.5**

Calibration For Fatigue In Structural Steel

Detail Category	Current Constant-Amplitude Fatigue Threshold (ksi)	Proposed Constant-Amplitude Fatigue Threshold (ksi)
A	24	24
B	16	16
B′	12	12
C	10	10
C′	12	12
D	7.0	**8.0**
E	4.5	4.5
E′	2.6	**3.1**

Calibration For Fatigue In Concrete

Proposed Fatigue I Resistance Factors
For Concrete and Reinforcement

Type	Proposed	
	Resistance Factor	**Reliability Index**
Reinforcement in tension	**1.25**	1.1
Concrete in compression	1.0	0.9

Uniform reliability can be achieved using variable resistance factors as shown above or change in constants used in calculations on next slide.

Calibration For Fatigue In Concrete and Reinforcement (Fatigue I)

- **Current** threshold equation:

For reinforcement $$(\Delta F)_{TH} = 24 - 20 f_{min} / f_y$$

For welded wire fabric $$(\Delta F)_{TH} = 16 - 0.33 f_{min}$$

- **Recommended** threshold equation

For reinforcement $$(\Delta F)_{TH} = 30 - 25 f_{min} / f_y$$

For welded wire fabric $$(\Delta F)_{TH} = 20 - 0.41 f_{\min}$$

f_{min} = minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

Work Done With Fatigue Limit States

For Steel Components

- Some design trials were performed by the industry.
- The proposed revisions resulted in making some fatigue details to be more critical than before.
- T14 opted to delay making a decision on the implementation until some issues are resolved.
- General agreement exists on the correctness of the approach except that a shift of 1.5 standard deviations is thought to be excessive.
- More work is currently underway to reduce the inherent conservatism in the process.

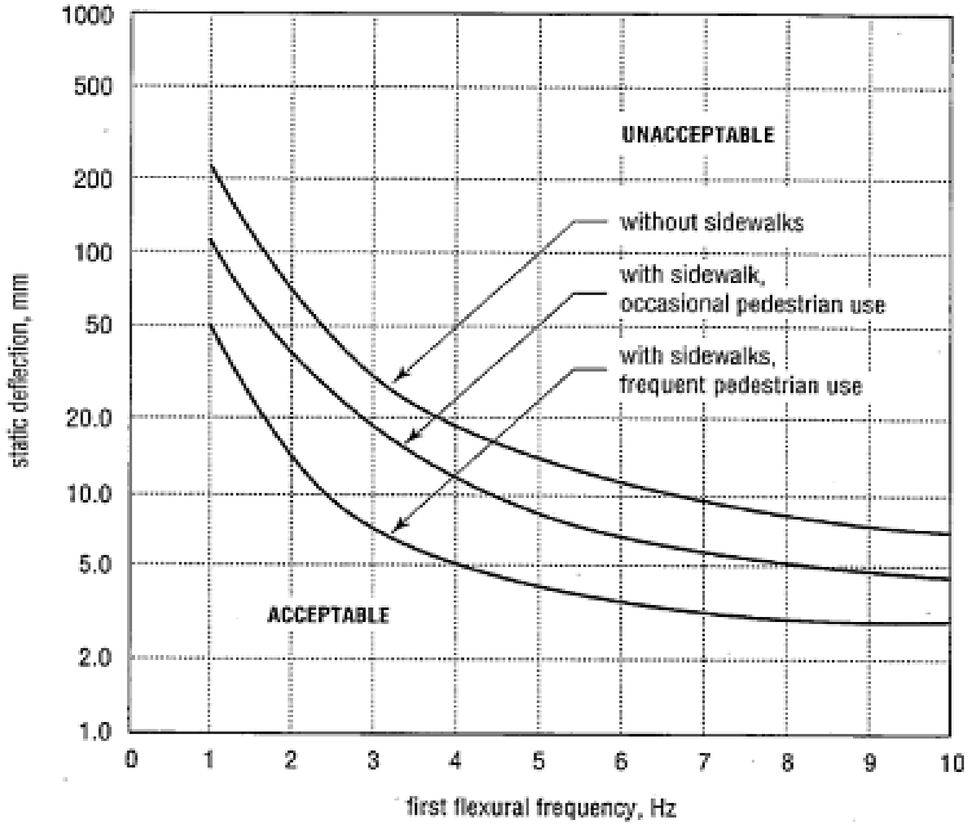
For Concrete Components: Wait for Steel

Live Load (LL)-Deflections in AASHTO LRFD

- Humans do not feel deflections, they feel the accelerations associated with the deflections and vibrations.
- Deflection limits were first used by railroads, then found their way to highway design.
- AASHTO limits on deflections are a fraction of the span length.

LL-Deflections – Service I

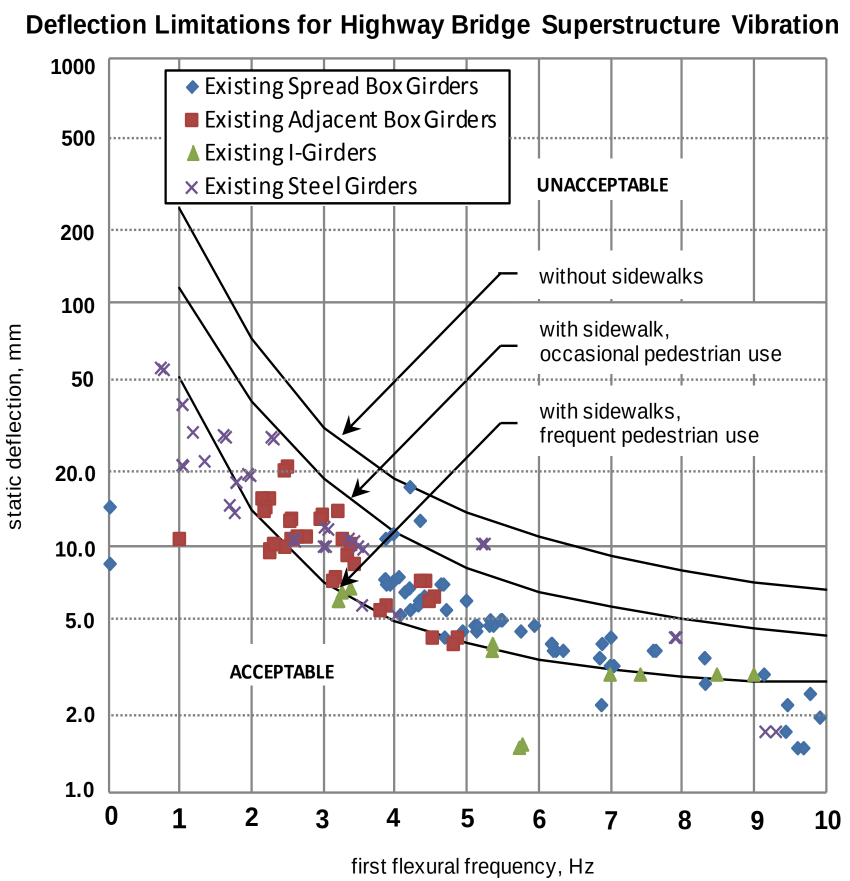
- Recommending consideration of the Canadian Code (CHBDC) criteria with load factor = 1.5

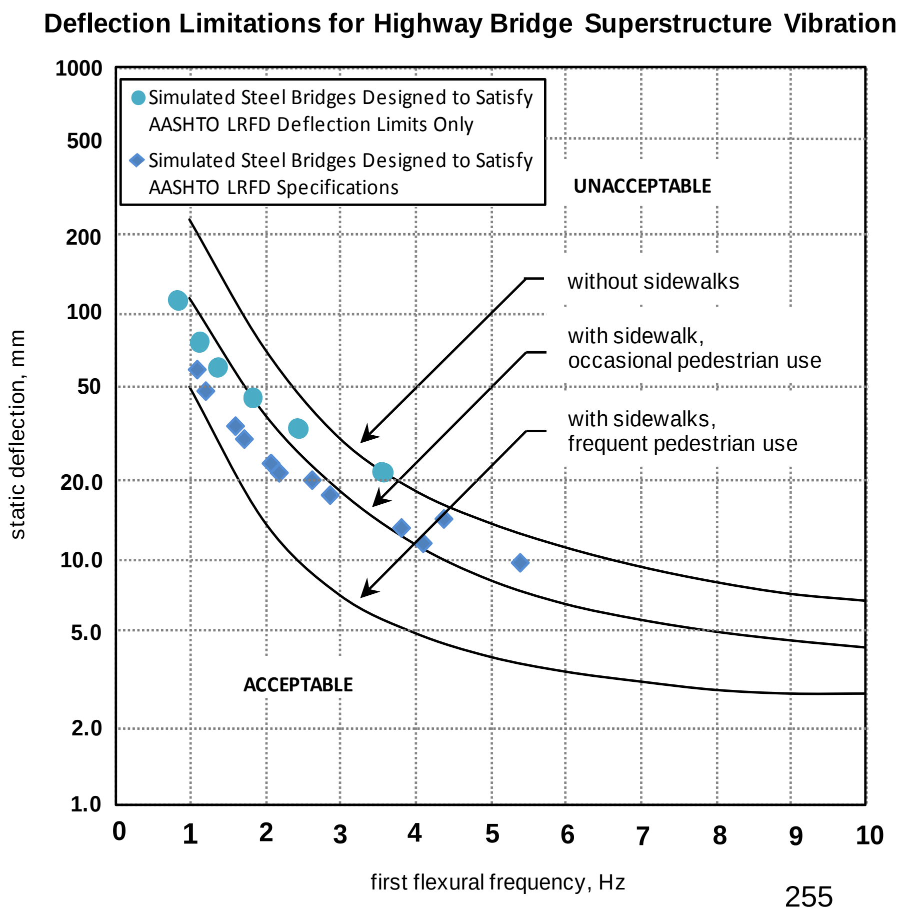


Note: frequency required

LL Deflections

- Some comparisons





LL Deflections

- Canadian (CSA) criteria more inclusive of basic factors.
- But current AASHTO provides similar trends.
- Change would require calculations not normally done for routine bridges, but software readily available and approximations available in literature.

LL Deflections

- Proposed revisions include:
  - Introduction of the deflection-frequency graph in *2.5.2.6.2—Criteria for Live Load Response.*
  - Introduction for Service V limit state that will be used to investigate LL deflections and vibrations.
  - Proposed load factors are 1.0 for permanent loads and 1.5 for live load.

Work Done With Deflection Limit State

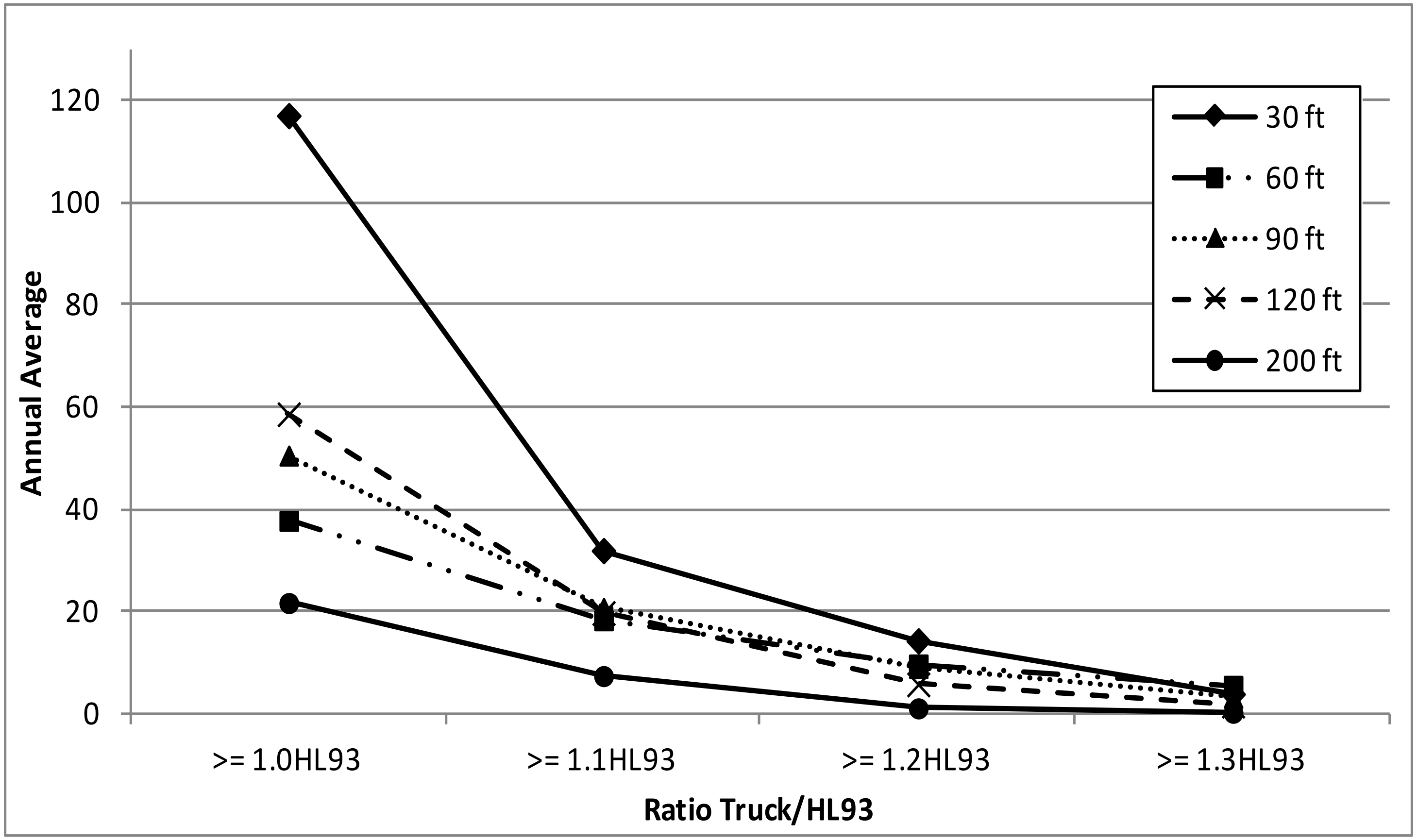
- Language prepared
- Reluctance to change current criteria
- Will need some convincing

Overload Calibration

- Performed under Service II. Load factor for live load 1.3.
- Applicable to steel bridges.
- Intended to minimize the potential of yielding under service loads.

Overload – Service II

- Annual average load occurrence from WIM data – scaled to ADTT = 2500 for all sites:



Overload – Service II

- Conclusions from WIM data alone:
  - Not enough basis to lower load factor.
  - Site-specific evaluation of unique sites warranted.
  - Design for single lane loaded is warranted.
  - Use of single lane MPF hard to justify **FOR THIS LIMIT STATE**.

Overload Calibration

- Single lane on load side, multiple load on resistance side.
- 41 steel bridges used.
- LFD β about 1.6 – 2.0, COVs about 0.32 – 0.92.
- Using R19B bias and COV on HL-93 and current load factors resulted in mean β of 1.8 but bias of only 0.09.
- If we are happy with current β, even though high for a SLS, calibrated results will yield similar behavior but with more consistency.

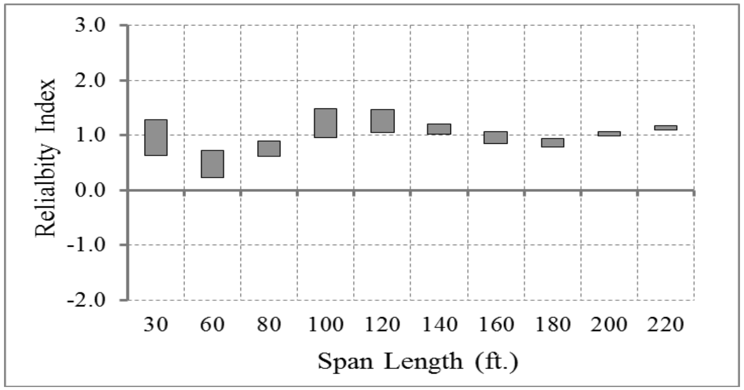
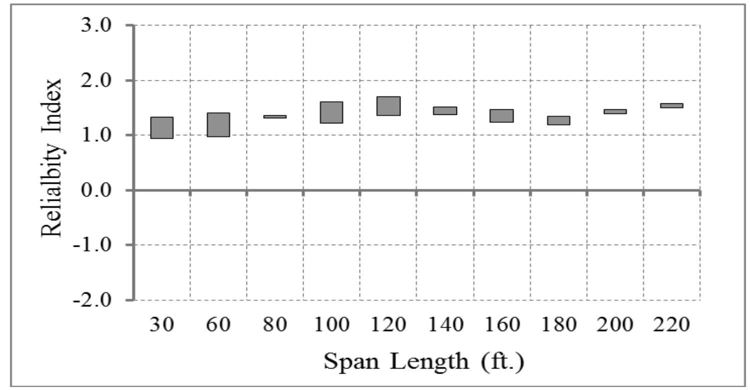
Work Done With Service II Limit State

- Commentary language prepared.
- No change in design procedure.
- Site-specific study is recommended for certain sites with high number of permit vehicles.

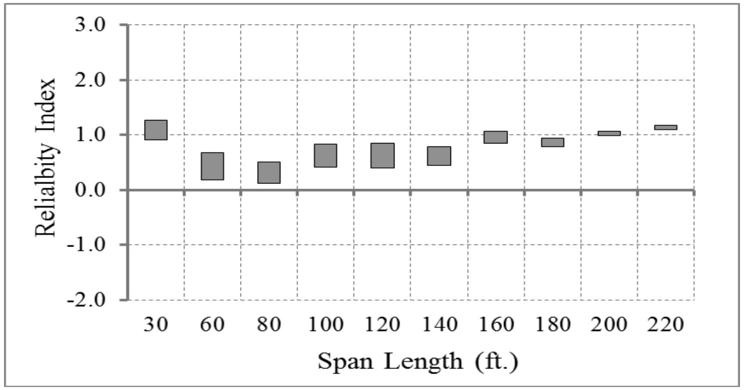
Tension in Prestressed Concrete (Service III)

- For decompression: target reliability index 1.2 for bridges in severe environmental conditions and 1.0 for normal environments.

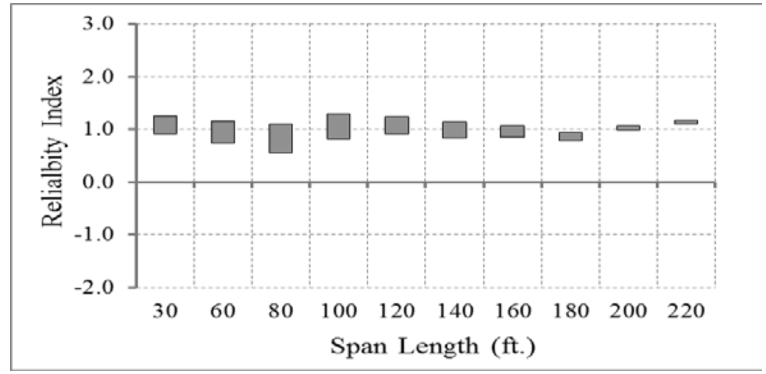
Tension in Prestressed Concrete (Service III)

	
Decompression limit state Reliability index of simulated I-girder bridges Refined time-dependent losses and elastic gain considered ADTT=5000, 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-dependent losses and elastic gain considered ADTT=5000, γ_{LL}=1.0, ($f_t = 0.0948\sqrt{f'_c}$) One-year return period

Tension in Prestressed Concrete (Service III)



Decompression limit state
Reliability index of simulated I-girder bridges
Refined time-dependent losses and elastic gain considered
ADTT=5000, 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-dependent losses and elastic gain considered
ADTT=5000, γ_{LL}=1.0, ($f_t = 0.19\sqrt{f'_c}$)
One-year return period

Tension in Prestressed Concrete (Service III), *Proposed Revisions*

Table 3.4.1-1

Load Combination	*Perm. Loads*	*LL*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*
Service III	1	~~0.80~~ γ_{LL}	1.00	—	—	1.00	1.00/ 1.20	γ_{TG}	γ_{SE}

Tension in Prestressed Concrete (Service III), *Proposed Revisions*

New Table 3.4.1-4

Component	γ_{LL}
Prestressed concrete components designed using the refined estimate of time-dependentlosses as specified in Article 5.9.5.4 in conjunction with taking advantage of the elastic gain	1.00
All other prestressed concrete components	0.80

Work Done With Service III Limit State

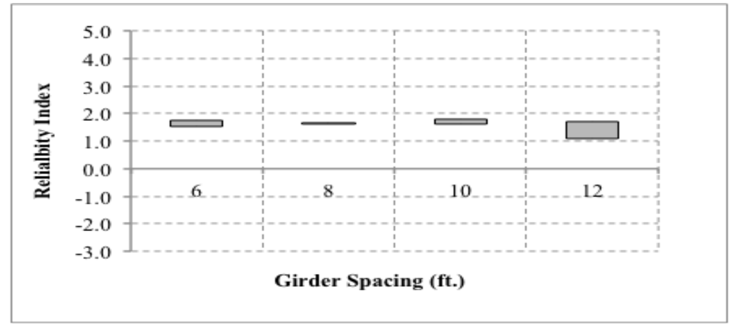
- Specifications revisions prepared and were approved.
- Revisions will appear in the next interim.
- Need to educate the users about the change from 0.8 to 1.0 load factor for some cases.

Control of Cracking in Reinforced Concrete Through Distribution of Reinforcement (Service I)

- Calibration was performed for reinforced concrete decks designed using the conventional (strip width) design method.
- Strength typically controls positive moment reinforcement, resulting in reliability index higher than shown below for positive moment.
- Results indicated that existing provisions produce uniform reliability (based on negative moment reinforcement reliability index is about 1.6 for Class 1 and 1.0 for Class 2 exposure).
- No specification revisions were deemed necessary.

Control of Cracking in Reinforced Concrete Decks (Service I)

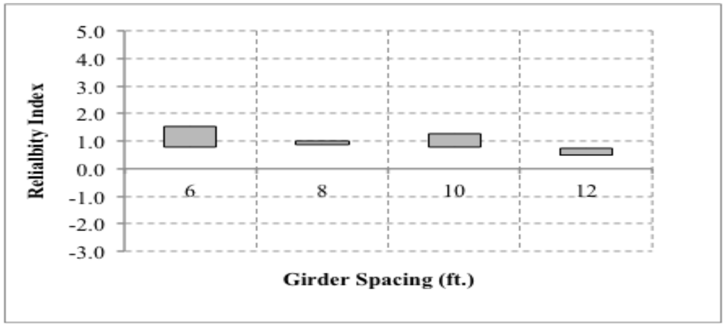
Positive Moment



Negative Moment

Class 1 Exposure
ADTT 5000
One-year return
LL load factor 1.6

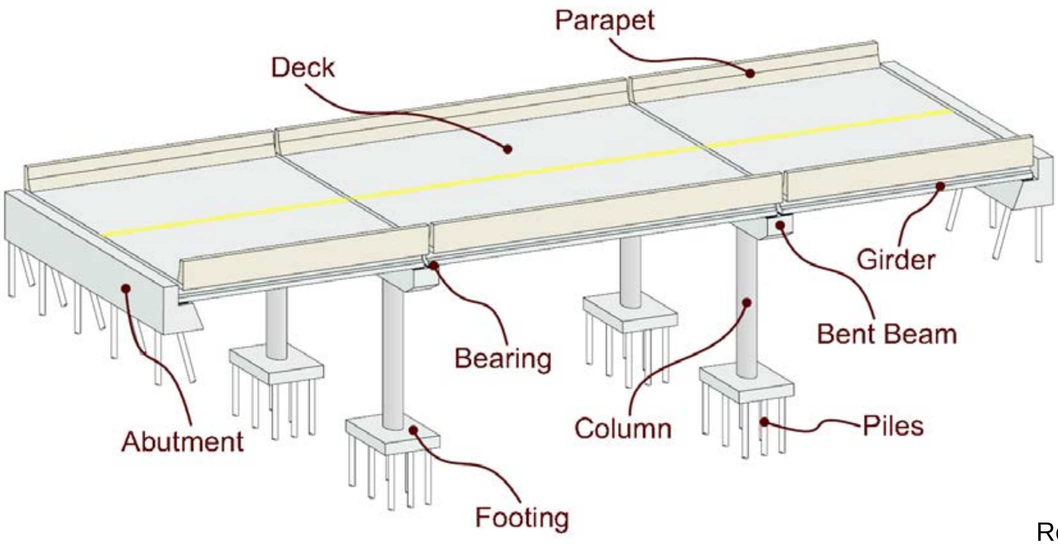
Control of Cracking in RC Decks (Service I)

 Positive Moment	 Negative Moment
	Class 2 Exposure ADTT 5000 One-year return LL load factor 1.0

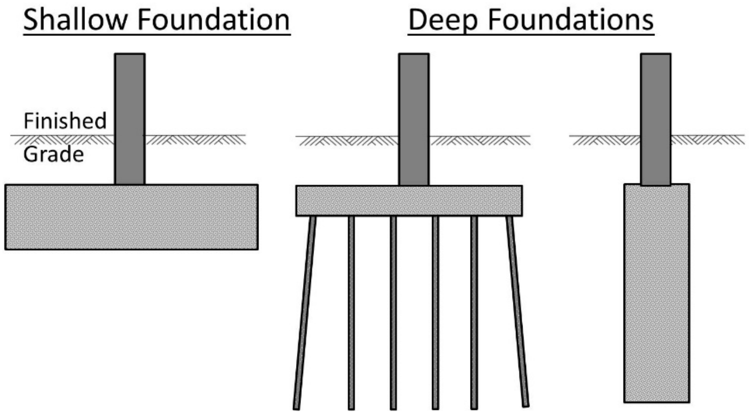
Work Done With Crack Control

- No revisions are needed.
- Need to educate the bridge community about the process.

Bridge Configuration and Foundation Types



Reference: Nielson (2005)



Foundation Deformations

- Vertical (Settlement)
- Lateral (Horizontal)
- Rotation

Consideration of Foundation Deformations in AASHTO

AASHTO Table 3.4.1-1

Superimposed Deformations

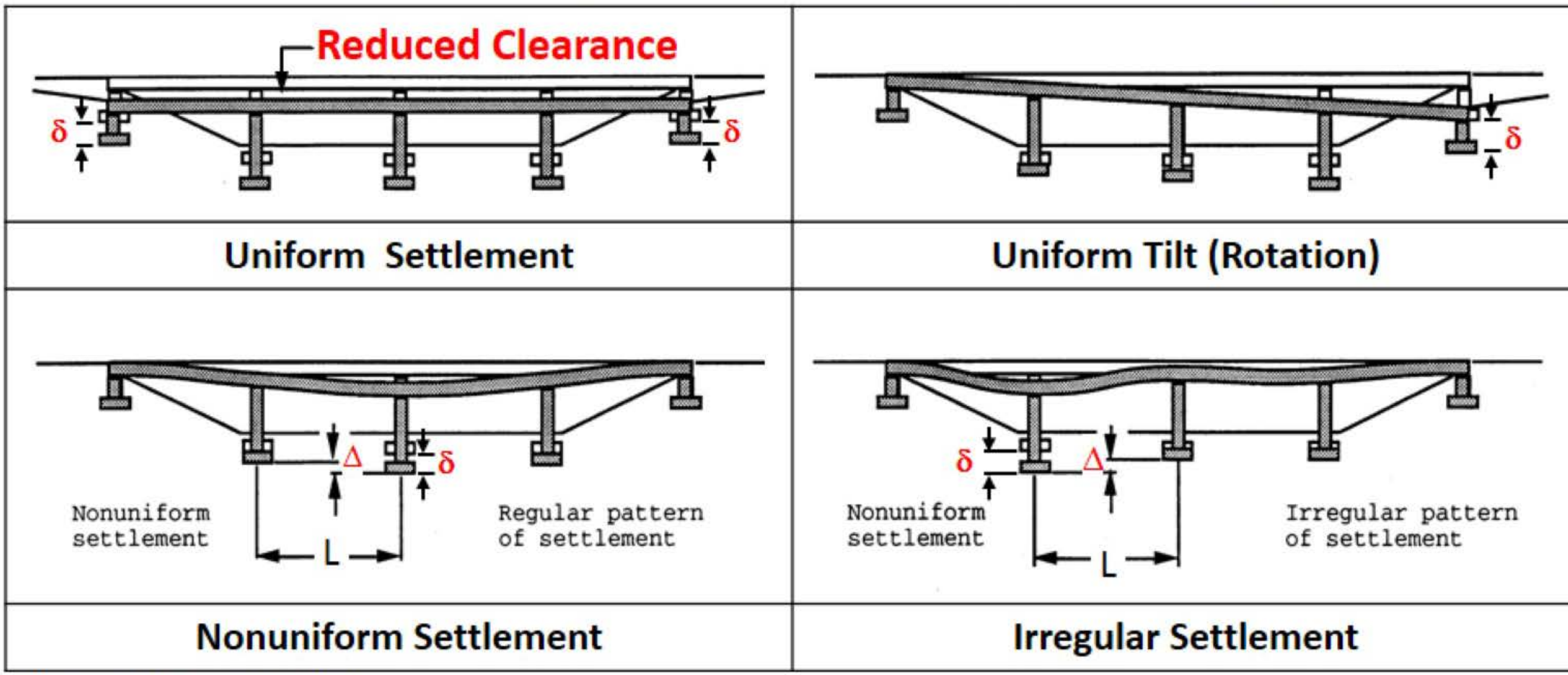
Load Combination Limit State		DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time				
											EQ	BL	IC	CT	CV
STRENGTH LIMIT	I	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
	II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
	III	γ_p	—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

Consideration of Foundation Deformations in AASHTO

Superimposed Deformations

- Article 3.12.6 – Settlement
  - *"Force effects due to extreme values of differential settlement among substructures and within individual substructure units shall be considered."*

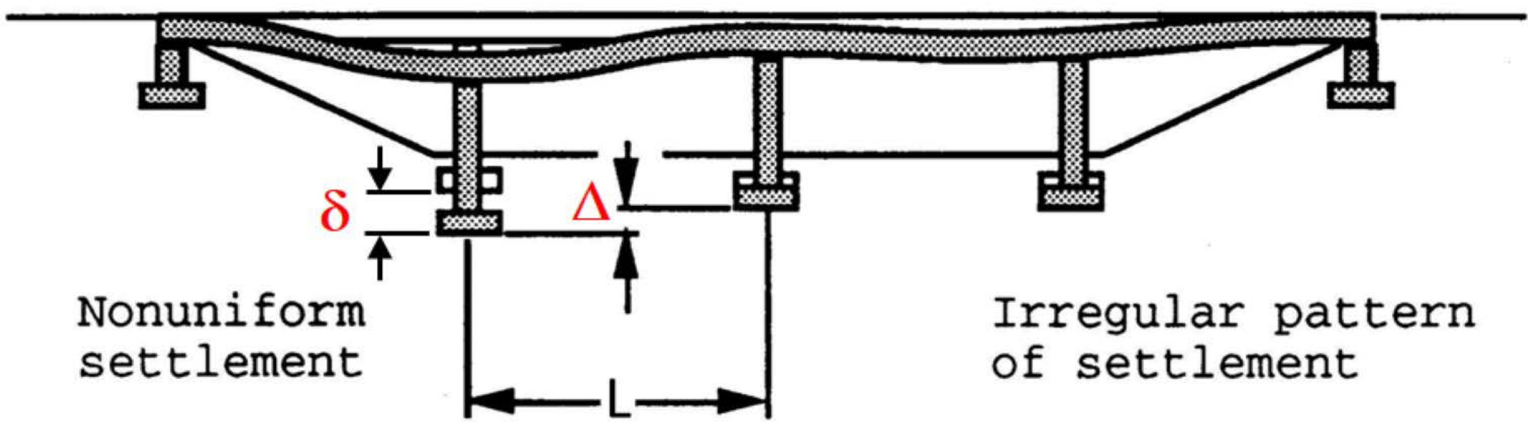
Several Foundation Deformation Patterns



Reference: After Duncan and Tan (1991)

Basic Questions

Settlement, δ, and
Angular Distortion, $A = \Delta/L$

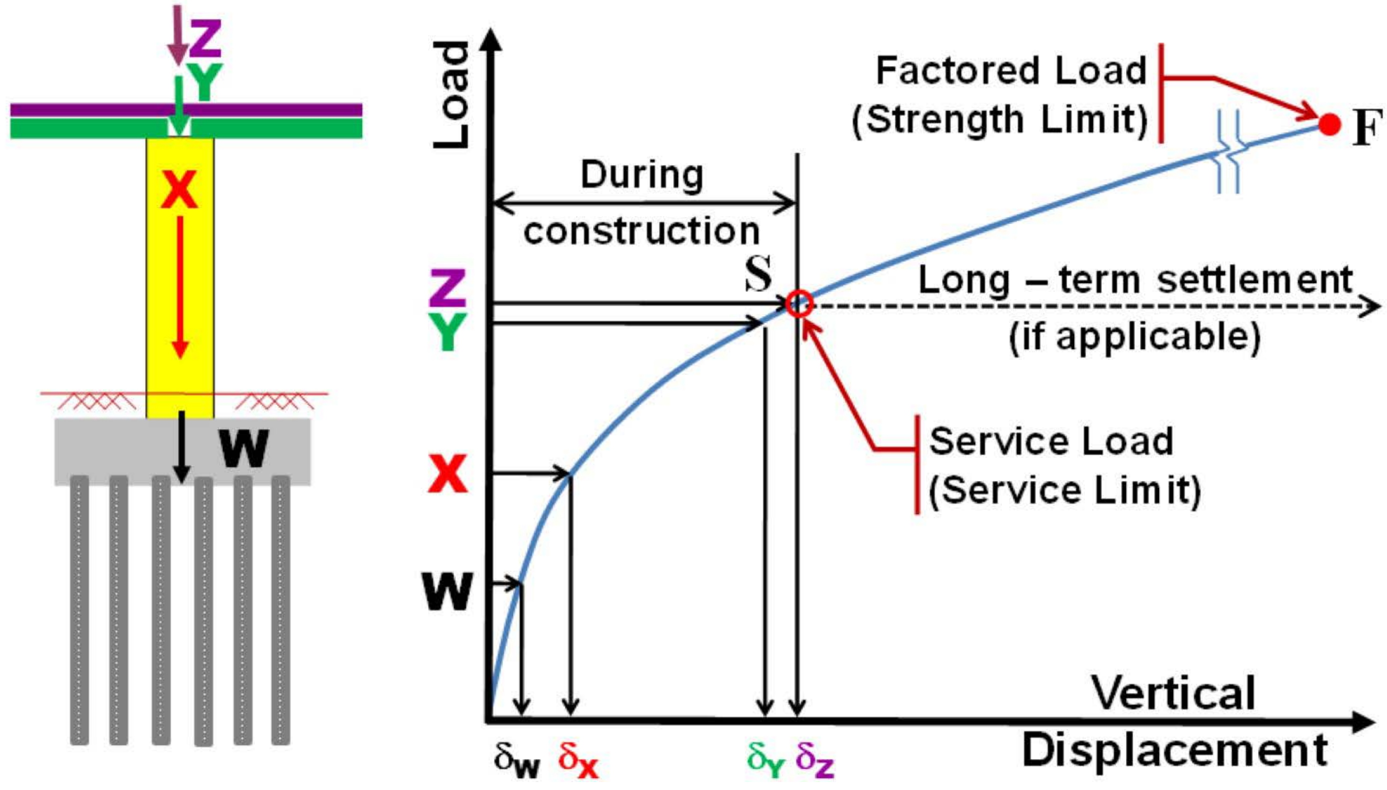


- **What is a *tolerable* value of Δ/L ?**
- **How *reliable* is the value of δ ?**

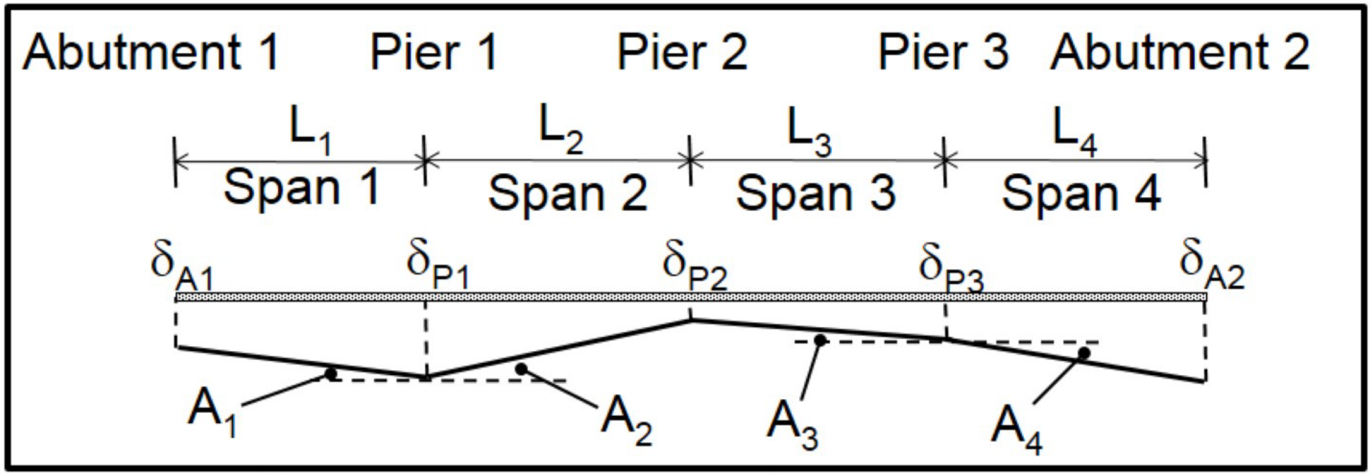
Basic Questions

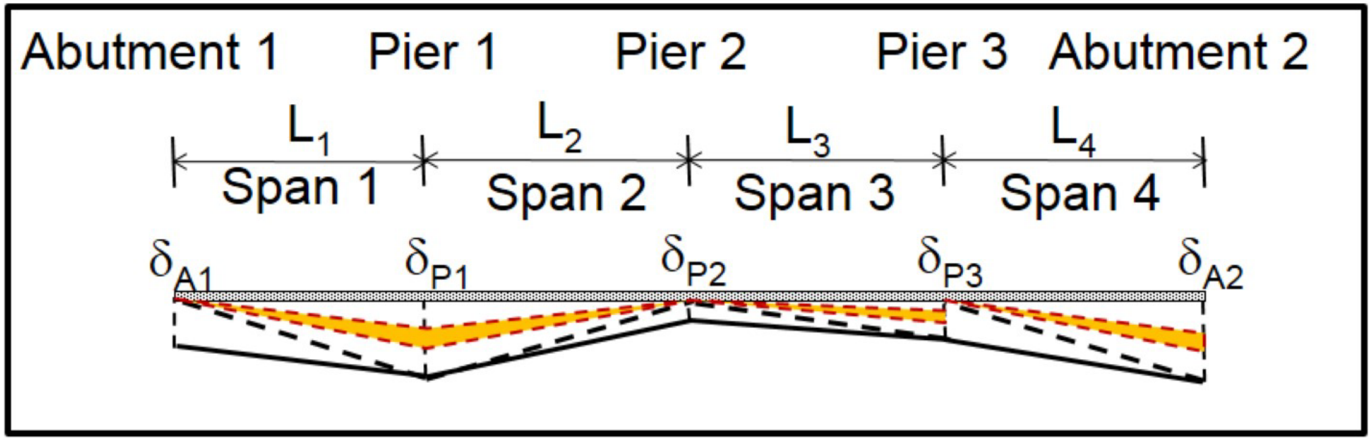
When is a Bridge Structure Affected?

Foundation could be shallow (spread footings) or deep (piles, shafts, etc.)

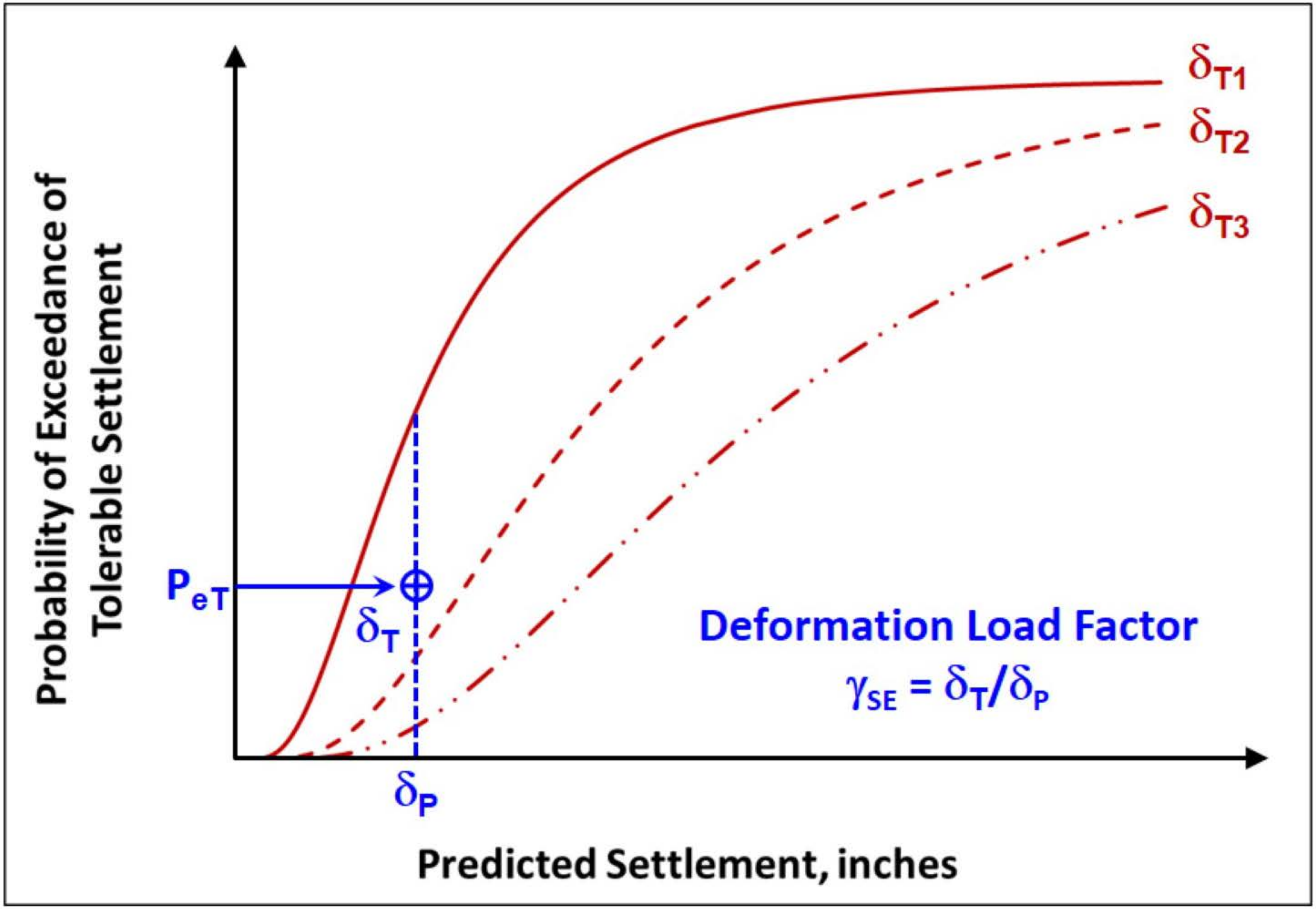


Construction Point Concept





Calibration of γ_{SE} Load Factor



Proposed Modifications to AASHTO

- Article 10.5.2 – “Service Limit States”
- Article 10.5.2 is cross-referenced in articles for various foundations 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

Proposed Modifications to AASHTO

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

Deformation	γ_{SE}
Immediate Settlement	
• Hough method	1.00
• Schmertmann method	1.25
• Local method	*
Consolidation settlement	1.00
Lateral deformation	
• P-y or SWM soil-structure interaction method	1.25
• Local method	*
*To be determined by the Owner based on local geologic conditions and calibration using a target reliability index of 0.50 for Service 1 limit state	

Work Done With Foundation Deformations

- Calibration of γ_{SE} load factor
  - General method applicable to various foundations and deformation modes
  - Example: Settlement of spread footings
  - Various methods
- Need to educate the bridge community about the process.

Questions and Contacts

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