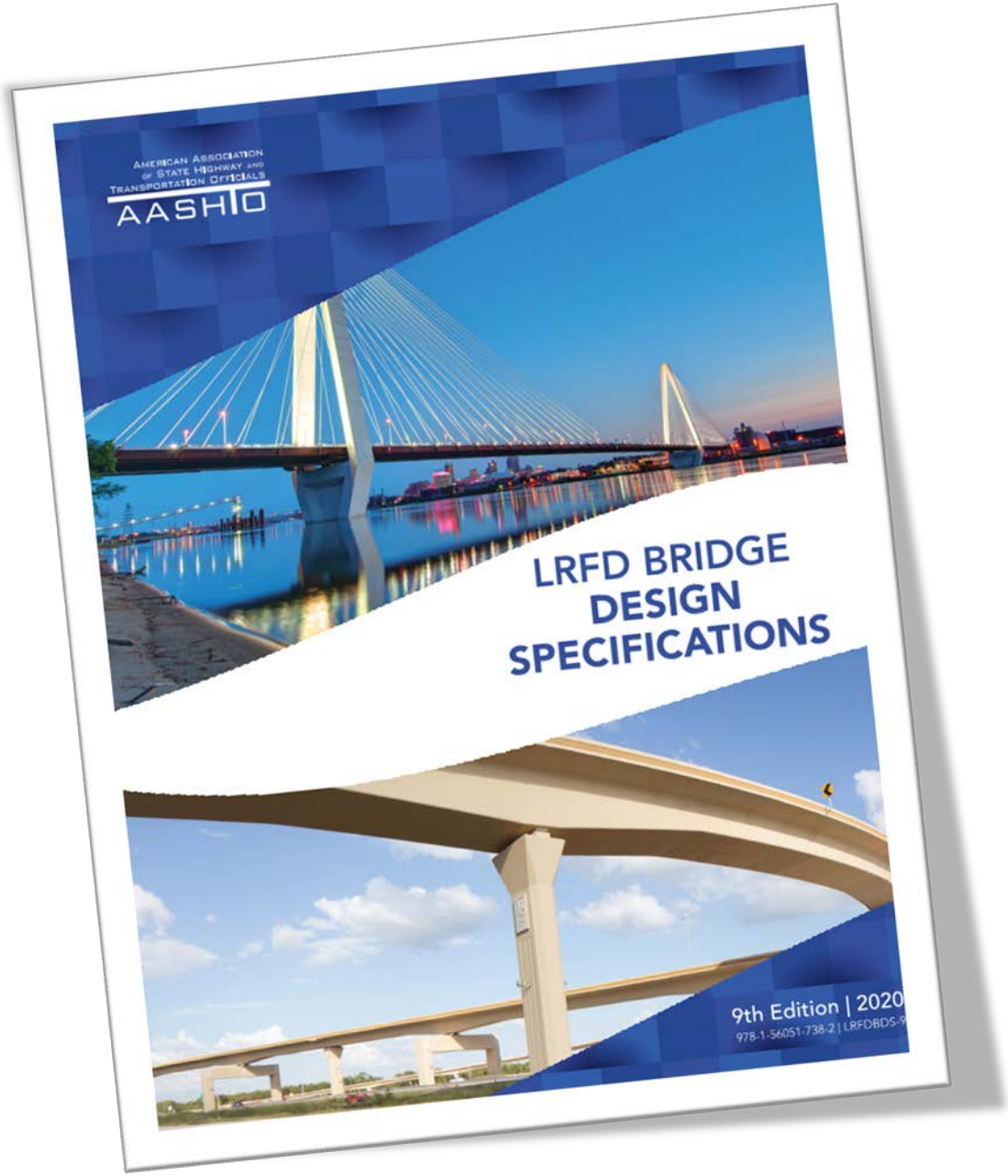


How to teach using the AASHTO LRFD Bridge Design Specifications

Michael P. Culmo, P.E.
Chief Bridge Engineer
CHA Consulting, Inc.

History of Bridge Specifications in the United States

It is important to understand and teach the history of the AASHTO Specifications to students

- It has been an evolutionary process

Pre-1900

Early Bridges were built by “Master Builders”

- No Specifications
- Covered Bridges
- Patented Designs
  - Pratt Truss
  - Howe Truss
  - Etc.

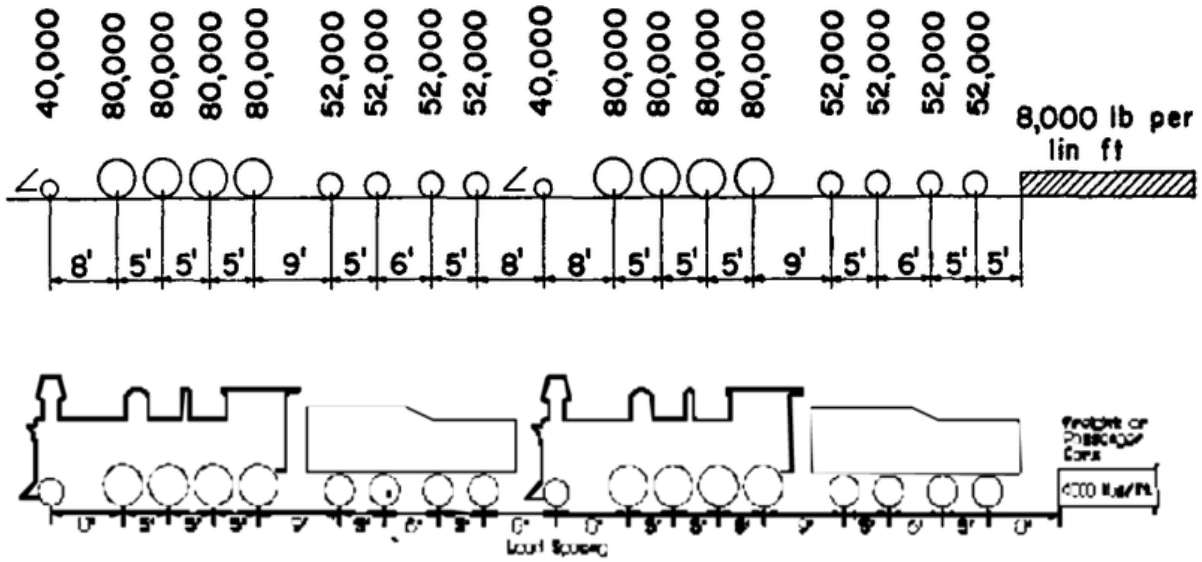
1870s

- 1 out of 4 bridges failed
- Failure rate at 40 per year
- Public was losing confidence
- Dec. 29, 1876
  - RR bridge failed with passenger train
  - 80 People died
  - Howe Truss Bridge designed by “seat of pants”
  - Tested with 6 locomotives on top
  - Essentially had a factor of safety of 1.0
  - ASCE Met: “the construction of the truss violated every cannon of our standard practice”

1894

Theodore Cooper developed a loading for trains

- Concentrated axle loads for locomotives followed by a uniform load representing the train.
- "Cooper loading"
  - E60 – 60 kip axles
  - E80 – 80 kip axles
- Became the standard in 1903
- Still in use today



December 12, 1914

Formation of AASHO

- American Association of State Highway Officials
- Committee on Bridges and Allied Structures
- Charged with development of specifications for highway bridges for
  - Design
  - Materials
  - Construction
  - Early specifications were copied and distributed to states

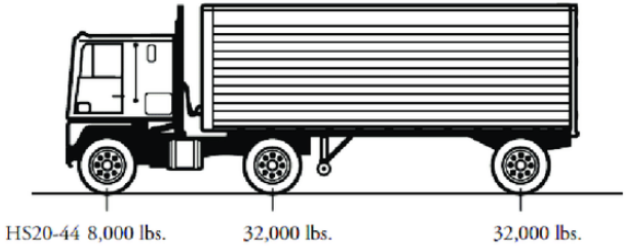
1931

AASHO published first specification

- "Standard Specifications for Highway Bridges and Incidental Structures"
  - Working stress method
  - Developed a truck loading (Similar to the Cooper Loading)
    - Axles representing the truck
    - Uniform load representing lines of vehicles
  - H loading (highway loading)
    - Notional load
    - Represents different loads on the road
    - Could adjust the overall magnitude of the load without adjusting the axle spacing

1944

AASHO Develops HS Truck: HS-20-44



Minutes from 1944 AASHO Committee Meeting

The meeting adjourned for lunch at 12:30 PM and reconvened at 2:15 PM to hear the progress report on the truck loading study being conducted at Texas A & M College. This paper disclosed the results of a tremendous amount of study which had been made on the stress producing effects of trucks weighed in 47 states at their load meter stations, and offered formulas involving span lengths, length of wheelbase of truck, and total weight of truck , which can be used to investigate these effects. Mr Kellum, Mr Wendell, and Mr Paxton led the discussions that followed, and it soon developed into a "free for all" over "them" good old fighting words "what design loading should be used".

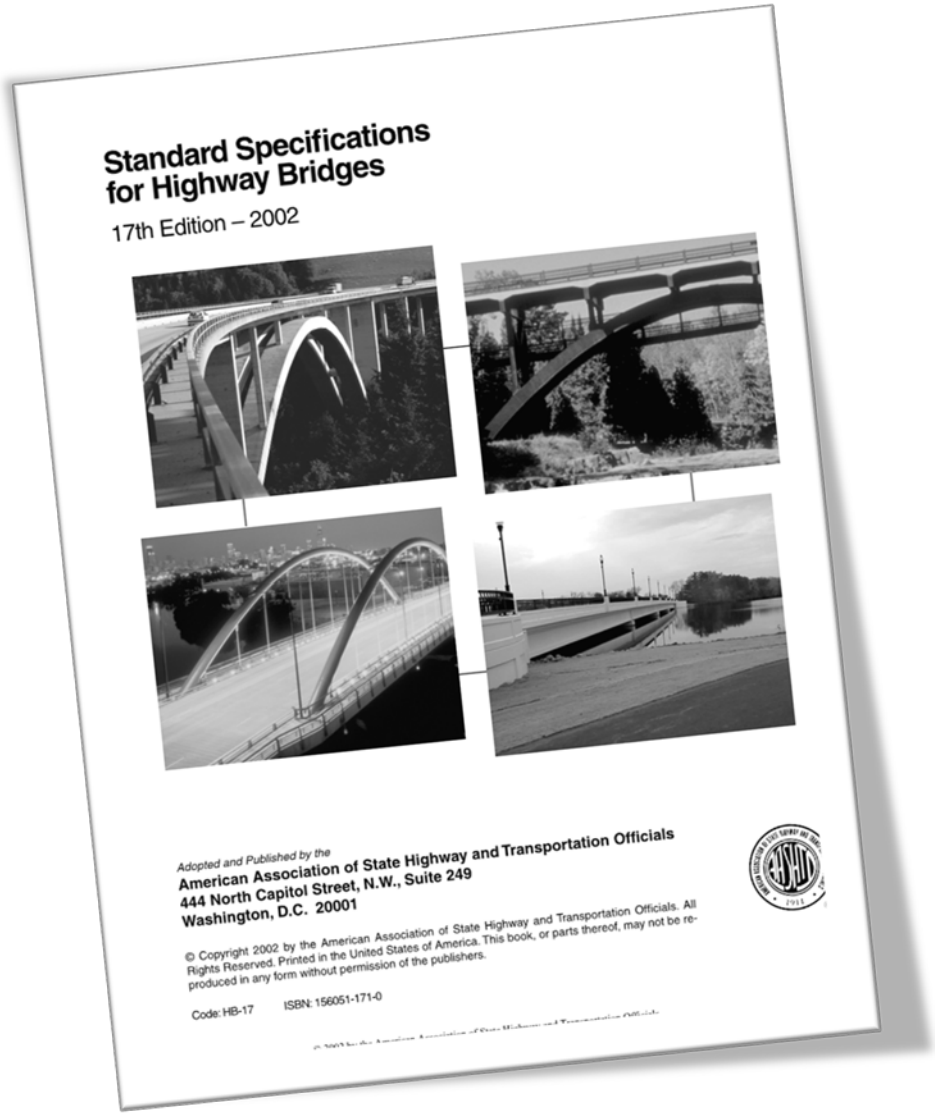
After the meeting got down to normalcy again, Mr Paxon presented the balanced design subcommittee report which started another argument.

As this was about all the refereeing that the chairman could endure in one afternoon, the meeting was adjourned at 5:00 PM

1963

AASHO becomes AASHTO

- American Association of State Highway and Transportation Officials
- Covered transportation structures
- Did not cover RR bridges
- Continued to publish the “Standard Specifications….”
- Added load factor design in the 1970’s
- 17th Edition was the “Final” Edition (2002)


Standard Specifications for Highway Bridges

17th Edition – 2002

Adopted and Published by the
American Association of State Highway and Transportation Officials
444 North Capitol Street, N.W., Suite 249
Washington, D.C. 20001

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What is AASHTO?

AASHTO is a nonprofit, nonpartisan association representing highway and transportation departments in the 50 states, the District of Columbia, and Puerto Rico. It represents all transportation modes including: air, highways, public transportation, active transportation, rail, and water. Its primary goal is to foster the development, operation, and maintenance of an integrated national transportation system.

- AASHTO Bridge & Structures Committee
  - 52 voting members
  - One from each state plus DC and PR

- Program Delivery and Operations
  - Bridges and Structures
  - Construction
  - Design
  - Environment and Sustainability
  - Materials and Pavements
  - Maintenance
  - Planning
  - Right of Way
  - Traffic Engineering
    - National Committee on Uniform Traffic Control Devices

Committee on Bridge & Structures (COBS or CBS)

Technical Committees

T-1 Security

T-2 Bearings & Exp. Devices

T-3 Seismic Design

T-4 Construction

T-5 Loads & Load Distribution

T-6 Fiber Reinf. Polymer Comp.

T-7 Guard Rail and Bridge Rail

T-8 Movable Bridges

T-9 Bridge Preservation

T-10 Concrete

T-11 Research

T-12 Sign Supports and traffic structures

T-13 Culverts

T-14 Structural Steel

T-15 Substructures & Ret. Walls

T-16 Timber structures

T-17 Welding

T-18 Bridge Mgmt, Eval. and Rehab. Insp

T-19 Computers

T-20 Tunnels

Technical Committee Work

- Meet once a year (June or July)
  - Some tech committees meet more than once
    - Concrete, Steel, Construction, etc.
- Tech committees do body of work of proposing changes to the code.

Types of AASHTO Documents

Specification: A document written in specification language format that is intended to be adopted in whole as part of a design process.

Guide Specification: A document written in specification format that MAY be adopted in whole or in part as part of a design specification.

Guideline: A document written to provide reference material that can be used to develop designs.

How do the Specification Change?

- Source of changes
  - Primarily Research
    - TRB
    - NCHRP
    - University research
    - Member states
  - Industry
  - Individual engineers?
    - Rare

Development of the LRFD Specifications

1986: AASHTO decides to initiate a study for a new specification

- Load and Resistance Factor Design (LRFD)
- Probability based code
- Written by consultants – Modjeski & Masters and others
- Completed in 1994

Development of the LRFD Specifications

LRFD Specifications were adopted by AASHTO in 1996

- Period of transition
  - Two specifications were in use
  - AASHTO did not eliminate the Standard Specifications
  - Most States continued to use the Standard Specifications in early 2000s
- Maintaining 2 specifications was too much effort
  - Standard Specifications were archived in 2002 (no further updates)
- Now: All states are designing in LRFD

Specification format

- Divided up into "Sections"
  - All inclusive
  - Like combining IBC, ACI, AISC, and more
- Most Sections have:
  - Separate Table of Contents
  - Scope
  - Definitions
  - Notation
    - Includes units for each term
    - Kips and inches for virtually all terms
    - Feet for certain terms
  - Why not Metric? It is a long story….

LRFD Page Format

Specifications on left, commentary on right

3.4—LOAD FACTORS AND COMBINATIONS

3.4.1—Load Factors and Load Combinations

The total factored force effect shall be taken as:

$$Q = \sum \eta_i \gamma_i Q_i \qquad (3.4.1\text{-}1)$$

where:

η_i = load modifier specified in Article 1.3.2
Q_i = force effects from loads specified herein
γ_i = load factors specified in Tables 3.4.1-1 to 3.4.1-5

C3.4.1

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

LRFD Truck Loading Development

- HS-20 had been around since 1944
- AASHTO looked to update the design vehicle for the LRFD Spec.
  - Not a real truck
  - “Notional” load that mimics actual traffic load
- Many states allow “Exclusion” vehicles
  - Excluded from weight laws (“Grandfathered” Trucks)
  - There was concern that the HS-20 was no longer accurate

LRFD Truck Loading Development

- Compared HS-20 to Exclusion Trucks
  - Moment results shown to right
  - Results were not favorable

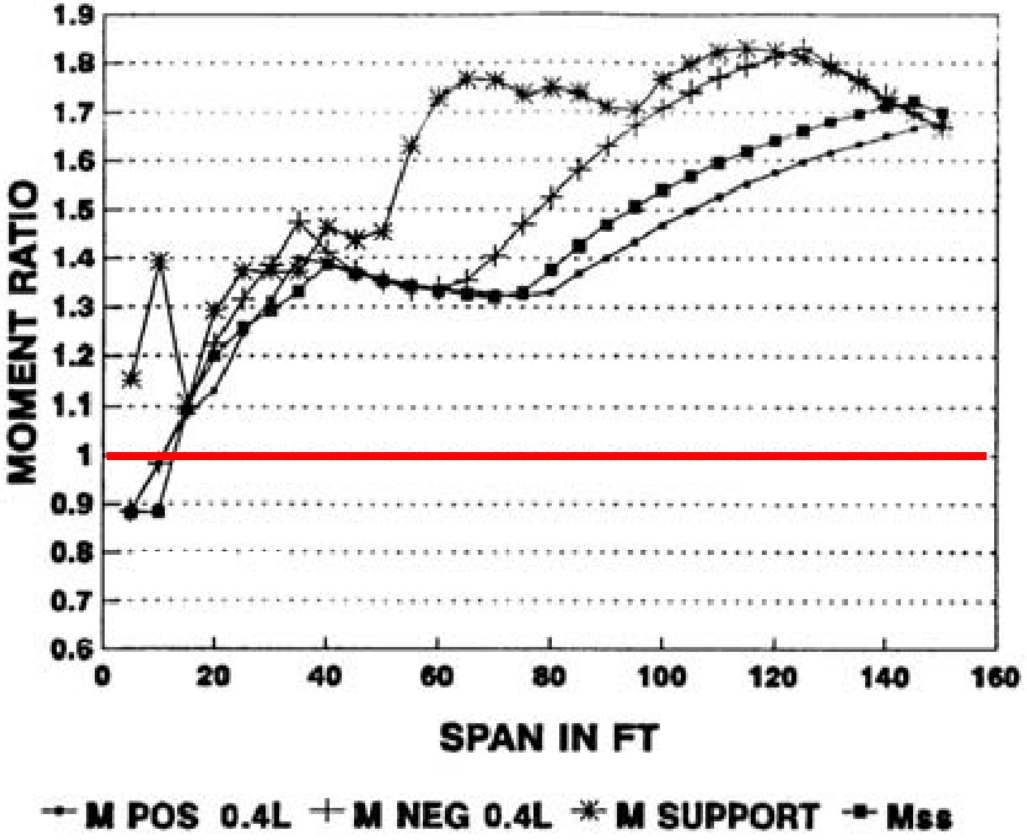


Figure C3.6.1.2.1-1—Moment Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft

LRFD Truck Loading Development

- Compared HS-20 to Exclusion Trucks
  - Shear Results shown to right
  - Results were not favorable

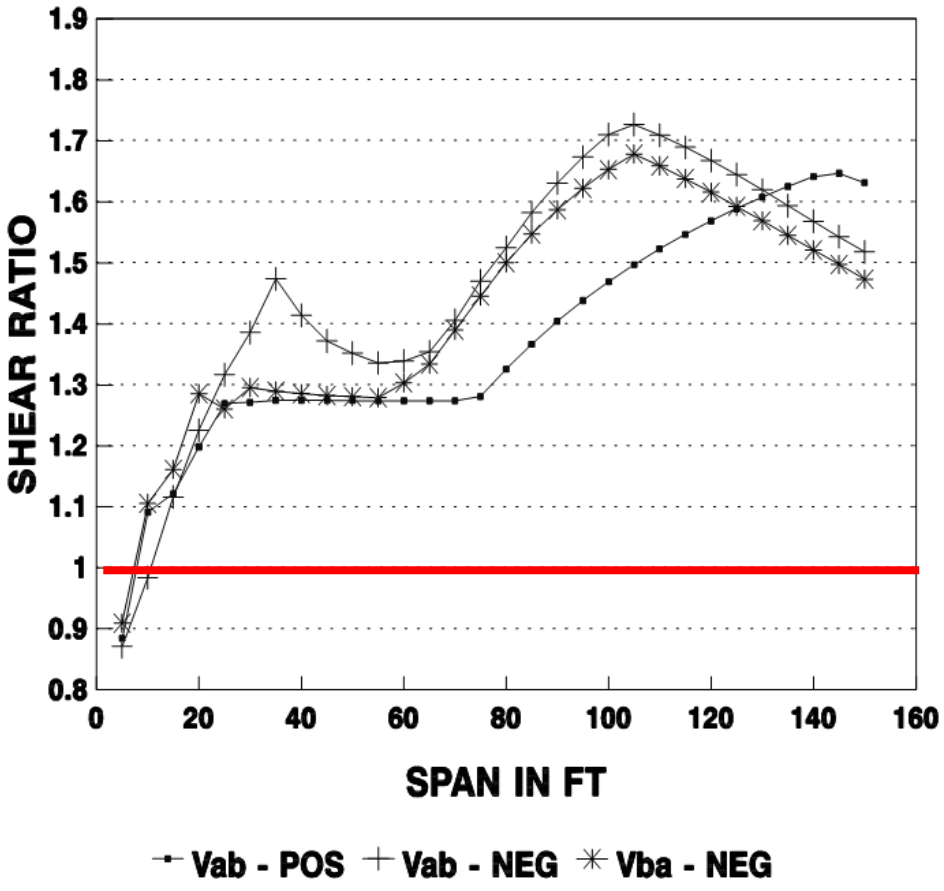


Figure C3.6.1.2.1-2—Shear Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft

LRFD Truck Loading Development

- Next steps
  - Develop a truck load that better mimics actual traffic
  - Have it work for simple spans and continuous spans
  - Mimic multiple trucks on continuous spans
- Initial ideas
  - 15 axle “notional” truck
    - Covered continuous spans well
    - Most of the struck would be off the bridge for short spans
  - Family of three trucks
- The AASHTO committee did not like the ideas
  - Overly complicated
  - Asked the researchers to try to produce something simpler
  - Try to make the HS-20 or something similar work better

LRFD Truck Loading Development

- Solution:
  - Combine the HS-20 Truck Load + Lane Load
  - Factored down to account for truck + lane (2.2 in SS, vs. 1.75 in LRFD)

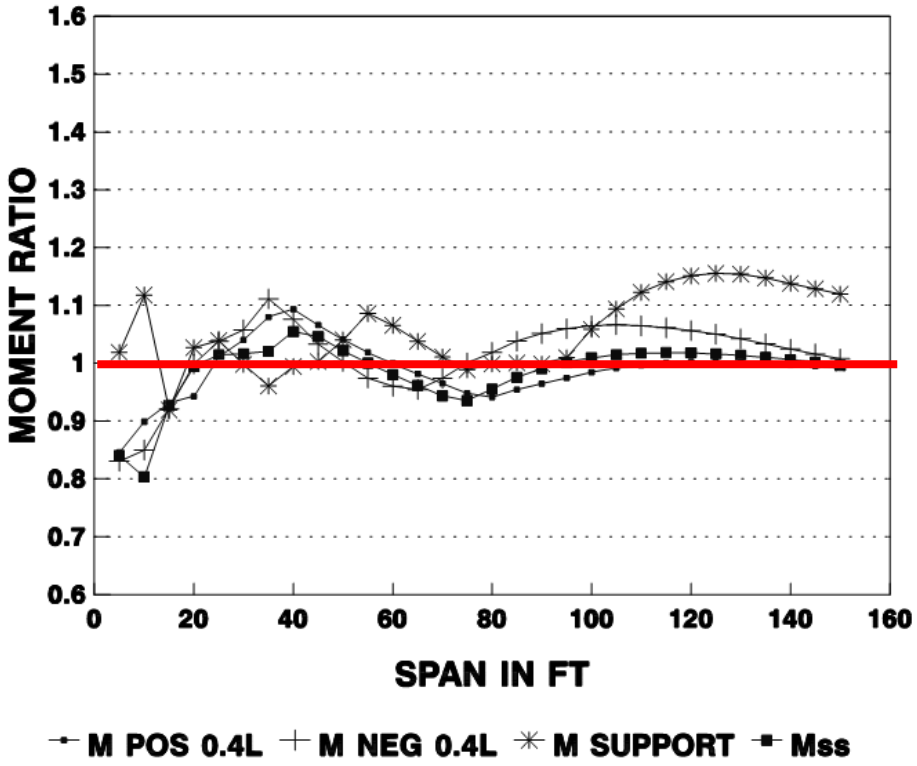


Figure C3.6.1.2.1-3—Moment Ratios: Exclusion Vehicles to Notional Model

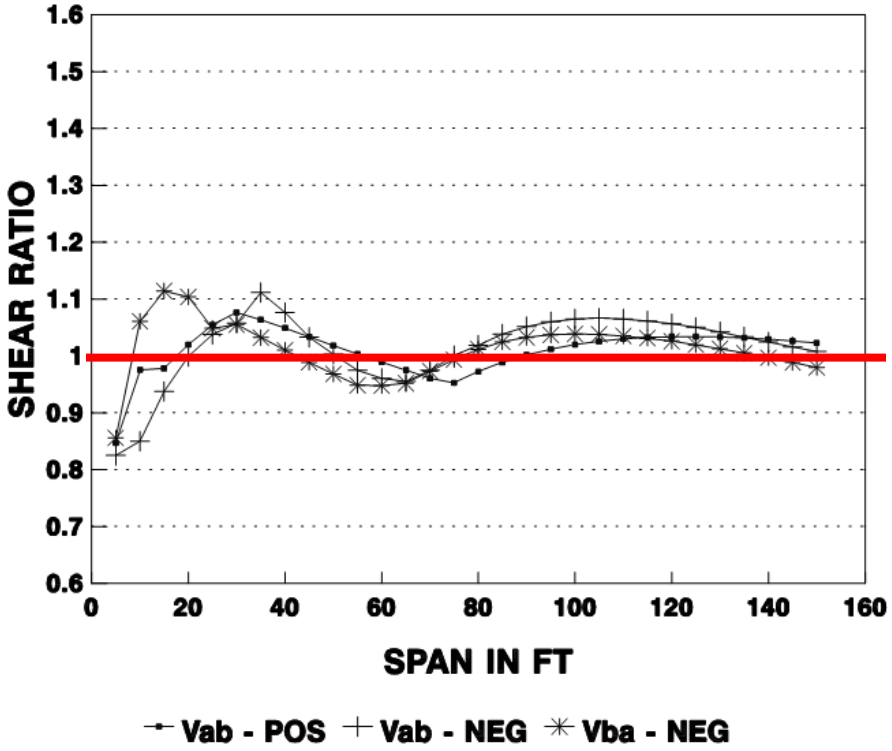
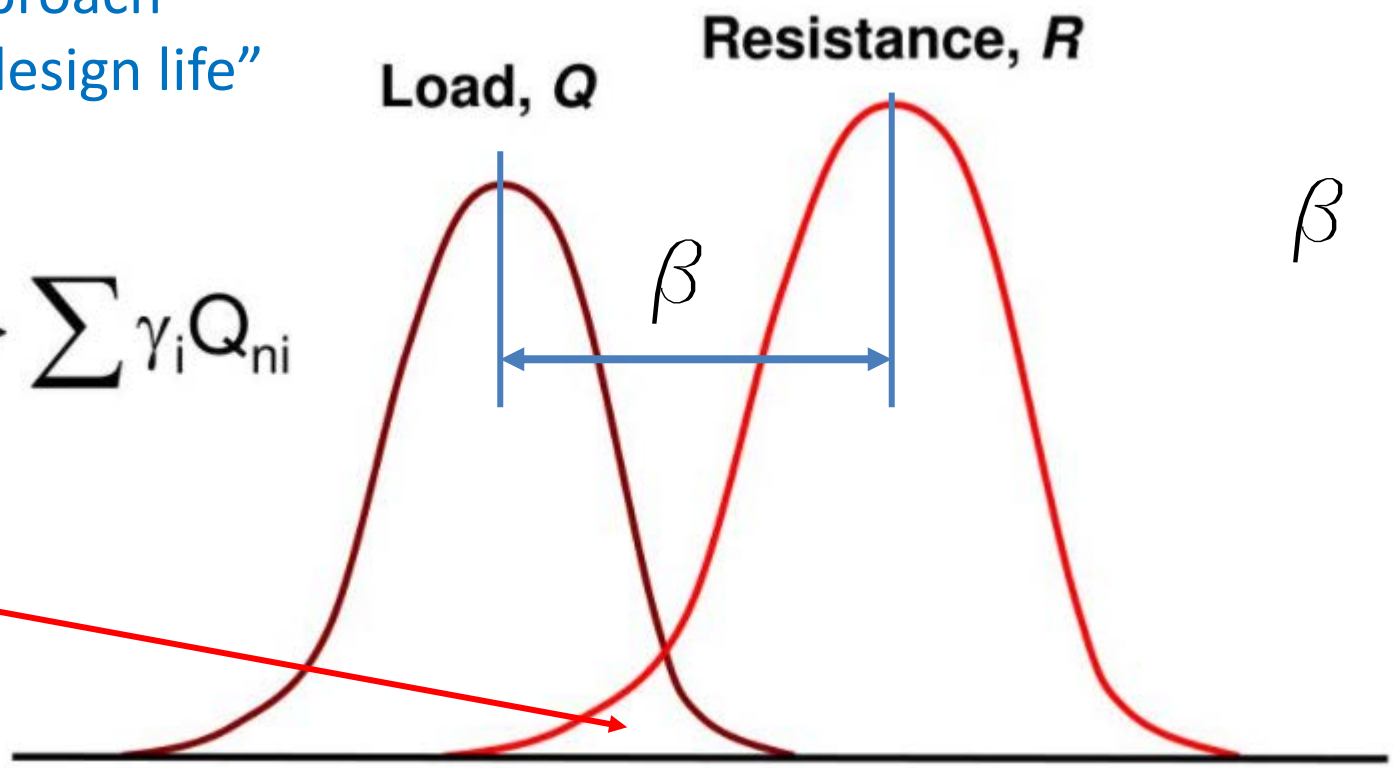


Figure C3.6.1.2.1-4—Shear Ratios: Exclusion Vehicles to Notional Model

LRFD Load Factor Development: Code Calibration 101

Probability Based Approach
based on a 75 year "design life"

$$\phi R_n > \sum \gamma_i Q_{ni}$$



AASHTO (1998) LRFD based design

β = 3.5
was chosen

What does this
area represent?

LRFD Design Equation

Basic Equation:

$$\eta\,(\Sigma\,\gamma_{DL}DL + \Sigma\,\gamma_{LL}LL) = \Phi R_n$$

Where:

γ_{DL} = load factor for dead loads

γ_{LL} = load factor for live load

η = load modifier for all loads

Φ = resistance factor

R_n = nominal strength of member

LRFD Load Combinations

Table 3.4.1-1—Load Combinations and Load Factors

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

Note: For Service I, the load factor for *EV* equals 1.2 for Stiffness Method Soil Failure as shown in Table 3.4.1-2.

Load Factors for Permanent Loads

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

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall and Compound Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• MSE wall internal stability soil reinforcement loads			
○ Stiffness Method			
▪ Reinforcement and connection rupture		1.35	N/A
▪ Soil failure – geosynthetics (Service I)		1.20	N/A
○ Coherent Gravity Method		1.35	N/A
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
• Internal and Compound Stability for Soil Failure in Soil Nail Walls		1.00	N/A
ES: Earth Surcharge		1.50	0.75

LRFD Load Combinations

- Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.
- Strength II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
- Strength III—Load combination relating to the bridge exposed to the design wind speed at the location of the bridge.

The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.5 regarding other traffic on the bridge simultaneously.

Vehicles become unstable at higher wind velocities. Therefore, high winds prevent the presence of significant live load on the bridge.

Wind load provisions in earlier editions of the specifications were based on fastest-mile wind speed measurements. The current wind load provisions are based on 3-second wind gust speed with 7 percent probability of exceedance in 50 years (mean return period of 700 years).

Load Combination Limit State	Dead Loads	Live Loads	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*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}	—	—	—	—	—

LRFD Load Combinations

- Strength IV—Load combination emphasizing dead load force effects in bridge superstructures.

The Strength IV load combination shown in these specifications was not fully statistically calibrated. It does not include live load; it controls over Strength I for components with dead load to live load ratio exceeding 7.0. These are typically long span bridges. The reliability indices tend to increase with the increase in the dead load to live load ratio, albeit at slow rate for bridges with high ratios.

- Strength V—Load combination relating to normal vehicular use of the bridge with wind of 80 mph velocity.

When applied with the load factor specified in Table 3.4.1-1 (i.e. 1.0), the 80 mph 3-second gust wind speed is approximately equivalent to the 100 mph fastest-mile wind used in earlier specifications applied with a load factor of 0.4. The latter was meant to be equivalent to a 55 mph fastest-mile wind applied with a load factor of 1.4.

Load Combination Limit State	Dead Loads	Live Loads	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ*	*BL*	*IC*	*CT*	*CV*
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—

LRFD Load Combinations

- Extreme Event I—Load combination including earthquake. The load factor for live load γ_{EQ}, shall be determined on a project-specific basis.

- Extreme Event II—Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, *CT*. The cases of check floods shall not be combined with *BL*, *CV*, *CT*, or *IC*.

Based on significant damage (LF=1.0), but survivable

Used to be 0.0, now 0.5 is suggested
Recent research indicates that they 0.0 may be justified for most bridges

Load Combination Limit State	Dead Loads	Live Loads	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ*	*BL*	*IC*	*CT*	*CV*
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00

LRFD Load Combinations

- Service I—Load combination relating to the normal operational use of the bridge with a 70 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders.
- Service III—Load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.
- Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load. For structures with unique truck loading conditions, such as access roads to ports or industrial sites which might lead to a disproportionate number of permit loads, a site-specific increase in the load factor should be considered.
- Service IV—Load combination relating only to tension in prestressed concrete columns with the objective of crack control.

Load Combination Limit State	Dead Loads	Live Loads	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ*	*BL*	*IC*	*CT*	*CV*
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—

Application of Live Load

- Most common model is “Line Girder” analysis for parallel girder bridges
- Why not 3D analysis?
  - It is common to have all girders the same size
  - Allows for future widening
  - A 3D analysis will result in different forces in each girder, which will make result in different girders
- Line girder analysis
  - Simple (less engineering costs)
  - Consistent
- 3D Analysis
  - Limited to curved bridges, heavily skewed bridges, or complex bridges

Application of Live Load

- Line Girder Analysis
  - Apply percentage of a truck to each girder
    - Live Load Distribution Factors (LLDF)
  - Converts a 3D problem to a 2D problem
  - Design using simple statics with moving loads
- Live Load Distribution Factors
  - Separate Factors for moment and shear
  - Approach
    - Analyze for one lane
    - Calculate moments and shears
    - Multiply results by the appropriate LLDF

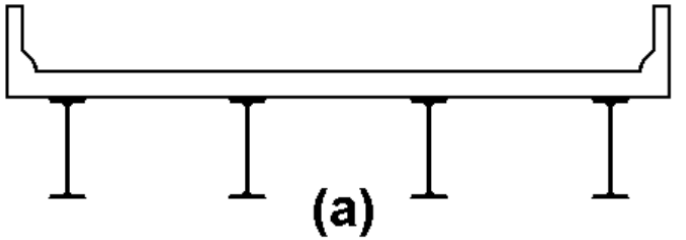
Any method of analysis that satisfies the requirements of equilibrium and compatibility and utilizes stress–strain relationships for the proposed materials may be used, including, but not limited to:

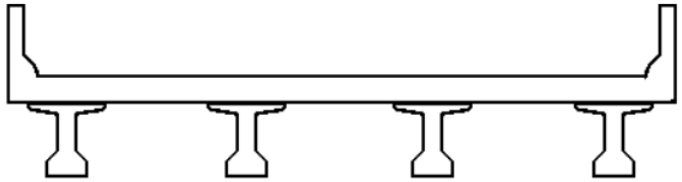
- classical force and displacement methods,
- finite difference method,
- finite element method,
- folded plate method,
- finite strip method,
- grid analogy method,
- series or other harmonic methods,
- methods based on the formation of plastic hinges, and
- yield line method.

Application of Live Load

This is just a sampling of common bridges: There are different equations for different bridge types

	Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Moment	Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
			use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$
Shear	Concrete Deck or Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$
			Lever Rule	Lever Rule	$N_b = 3$





Be sure to check range of applicability

How to read the Specifications

- Words matter
  - Certain words are very important

The term "notional" is often used in these Specifications to indicate an idealization of a physical phenomenon, as in "notional load" or "notional resistance." Use of this term strengthens the separation of an engineer's "notion" or perception of the physical world in the context of design from the physical reality itself.

The term "shall" denotes a requirement for compliance with these Specifications.

The term "should" indicates a strong preference for a given criterion.

The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criteria may also be used in a manner consistent with the LRFD approach to bridge design.

How to read the Specifications

- Words matter
  - Most text written by committee; therefore, the text may not be perfect and difficult to understand

5.9.4.4—Pretensioned Anchorage Zones

5.9.4.4.1—Splitting Resistance

The factored splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:

$$P_r = f_s A_s \qquad (5.9.4.4.1\text{-}1)$$

where:

f_s = stress in steel not to exceed 20.0 ksi
A_s = total area of reinforcement located within the distance $h/4$ from the end of the beam (in.^{2})
h = overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)

For pretensioned I-girders or bulb tees, A_s shall be taken as the total area of the vertical reinforcement located within a distance of $h/4$ from the end of the member, where h is the overall height of the member (in.).

For pretensioned solid or voided slabs, A_s shall be taken as the total area of the horizontal reinforcement located within a distance of $h/4$ from the end of the member, where h is the overall width of the member (in).

For pretensioned box or tub girders, A_s shall be taken as the total area of vertical reinforcement or horizontal reinforcement located within a distance $h/4$ from the end of the member, where h is the lesser of the overall width or height of the member (in.).

For pretensioned members with multiple stems A_s shall be taken as the total area of vertical reinforcement, divided evenly among the webs, and located within a distance $h/4$ from the end of each web.

The resistance shall not be less than 4 percent of the total prestressing force at transfer.

The reinforcement shall be as close to the end of the beam as practicable.

Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.

$$0.04\ Pt \leq fs\ As$$

How to read the Specifications

- Words matter

6.10.1.9—Web Bend-Buckling Resistance

6.10.1.9.1—Webs without Longitudinal Stiffeners

The nominal bend-buckling resistance shall be taken as:

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad (6.10.1.9.1\text{-}1)$$

but not to exceed the smaller of $R_h F_{yc}$ and $F_{yw}/0.7$

in which:

k = bend-buckling coefficient

$$= \frac{9}{(D_c/D)^2} \qquad (6.10.1.9.1\text{-}2)$$

where:

D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1.

D = web depth (in.)

R_h = hybrid factor specified in Article 6.10.1.10.1

How to read the Specifications

- Punctuation matters

Where required, lateral bracing should be placed either in or near the plane of a flange or chord being braced. Investigation of the requirement for lateral bracing shall include, but not be limited to:

- transfer of lateral wind loads to the bearings as specified in Article 4.6.2.7,
- transfer of lateral loads as specified in Article 4.6.2.8, and
- control of deformations and cross-section geometry during fabrication, erection, and placement of the deck.

And = all 3 apply
If it was or, then only one need apply

6.10.1.10.2—Web Load-Shedding Factor, R_b

When checking constructibility according to the provisions of Article 6.10.3.2, or:

- the section is composite and is in positive flexure and the web satisfies the requirement of Article 6.10.2.1.1 or 6.11.2.1.2, as applicable, or:
- the web satisfies:

$$\frac{2D_c}{t_w} \le \lambda_{rw} \qquad (6.10.1.10.2\text{-}1)$$

then, R_b shall be taken equal to 1.0.

Otherwise:

- in lieu of a strain–compatibility analysis considering the web effective widths, for longitudinally-stiffened sections in which one or more continuous longitudinal stiffeners are provided that satisfy $d_s/D_c < 0.76$:

$$R_b = 1.07 - 0.12\frac{D_c}{D} - \frac{a_{wc}}{1200 + 300a_{wc}}\left[\frac{D}{t_w} - \lambda_{rwD}\right] \le 1.0 \qquad (6.10.1.10.2\text{-}2)$$

- for all other cases:

$$R_b = 1.0 - \frac{a_{wc}}{1200 + 300a_{wc}}\left(\frac{2D_c}{t_w} - \lambda_{rw}\right) \le 1.0 \qquad (6.10.1.10.2\text{-}3)$$

in which:

λ_{rw} = limiting slenderness ratio for a noncompact web, expressed in terms of $2D_c/t_w$, calculated as follows:

- for longitudinally-stiffened sections:

$$= \left(\frac{2D_c}{D}\right)\lambda_{rwD} \qquad (6.10.1.10.2\text{-}4)$$

- for all other cases:

$$4.6\sqrt{\frac{E}{F_{yc}}} \le \lambda_{rw} = \left(3.1 + \frac{5.0}{a_{wc}}\right)\sqrt{\frac{E}{F_{yc}}} \le 5.7\sqrt{\frac{E}{F_{yc}}} \qquad (6.10.1.10.2\text{-}5)$$

λ_{rwD} = limiting slenderness ratio for a noncompact web, expressed in terms of D/t_w, calculated as follows:

- for homogeneous longitudinally-stiffened sections:

$$= 0.95\sqrt{\frac{Ek}{F_{yc}}} \qquad (6.10.1.10.2\text{-}6)$$

- for hybrid longitudinally-stiffened sections:

$$= \left(\frac{1}{2D_c/D}\right)5.7\sqrt{\frac{E}{F_{yc}}} \qquad (6.10.1.10.2\text{-}7)$$

a_{wc} = for all sections except as noted below, ratio of two times the web area in compression to the area of the compression flange

$$= \frac{2D_c t_w}{b_{fc} t_{fc}} \qquad (6.10.1.10.2\text{-}8)$$

for composite longitudinally-stiffened sections in positive flexure:

$$= \frac{2D_c t_w}{b_{fc} t_{fc} + b_s t_s (1 - f_{DC1}/F_{yc})/3n} \qquad (6.10.1.10.2\text{-}9)$$

where:

b_{fc} = full width of the compression flange
b_s = effective width of the concrete deck (in.)
d_s = distance from the centerline of the closest plate longitudinal stiffener or from the gauge line of the closest angle longitudinal stiffener to the inner surface or leg of the compression flange element (in.)
D = web depth (in.)
D_c = depth of the web in compression in the elastic range (in.). For composite sections, D_c shall be determined as specified in Article D6.3.1. For longitudinally-stiffened sections, all

HELP!!!!

How to read the Specifications

- Words matter

5.9.3.4—Refined Estimates of Time-Dependent Losses

5.9.3.4.1—General

For nonsegmental prestressed members, more accurate values of creep-, shrinkage-, and relaxation-related losses than those specified in Article 5.9.3.3 may be determined in accordance with the provisions of this article. For precast pretensioned girders without a composite topping and for precast or cast-in-place nonsegmental post-tensioned girders, the provisions of Articles 5.9.3.4.4 and 5.9.3.4.5, respectively, shall be considered before applying the provisions of this article.

How to read the Specifications

- Common “may” provision

2.5.2.6.3—Optional Criteria for Span-to-Depth Ratios

Unless otherwise specified herein, if an Owner chooses to invoke controls on span-to-depth ratios, the limits in Table 2.5.2.6.3-1, in which S is the slab span length and L is the span length, both in feet, may be considered in the absence of other criteria. Where used, the limits in Table 2.5.2.6.3-1 shall be taken to apply to overall depth unless noted.

Table 2.5.2.6.3-1—Traditional Minimum Depths for Constant Depth Superstructures

Superstructure		Minimum Depth (Including Deck) When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced Concrete	Slabs with Main Reinforcement Parallel to Traffic	$\frac{1.2(S+10)}{30}$	$\frac{S+10}{30} \geq 0.54$ ft
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 6.5$ in.	$0.027L \geq 6.5$ in.
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$
Steel	Overall Depth of Composite I-Beam	$0.040L$	$0.032L$
	Depth of I-Beam Portion of Composite I-Beam	$0.033L$	$0.027L$
	Trusses	$0.100L$	$0.100L$

Conclusions

- The AASHTO LRFD Bridge Design Specification are an all-inclusive design “code”
- It was developed and is maintained by the AASHTO Committee on Bridges and Structures (COBS)
- It is a probability-based design code
- The most common form of design for beams is a simplified line-girder approach
- You need to exercise great care in reading provisions

Questions?