Steel Bridge Design Basics

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HDR

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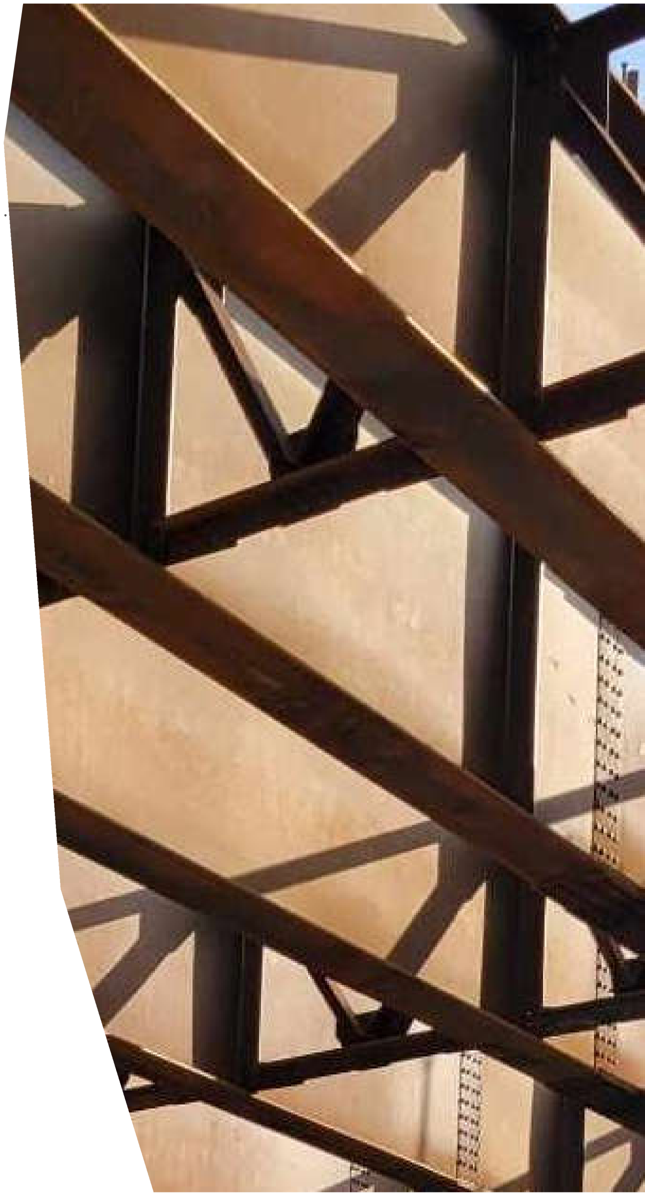
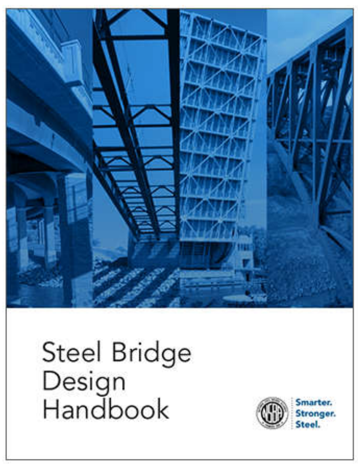
Steel Bridge Design Basics

1. Introduction
2. Requirements/Rules of Thumb/Good Practices
3. Material Considerations
4. Analysis Considerations
5. Design Considerations
6. Design Resources
7. Conclusions



But first….credit, where it's due…

- Brandon Chavel, PhD, PE - Vice President - Bridge for AISC/NSBA
- Frank Russo, PhD, PE – Russo Structural Services
- AISC/NSBA Resources



Fundamentals of Steel Bridge Engineering

This set of Powerpoint modules can be used to cover a comprehensive bridge course. Each lesson dives into the respective topic in full detail, and all modules include extensive speaker notes. The slides can be used in parts, and they can be modified or used as-is to fit the needs and contents for a particular course.

The content is closely tied to the Steel Bridge Design Handbook maintained by the National Steel Bridge Alliance (NSBA). All modules reflect the *AASHTO LRFD Bridge Design Specifications, 9th Edition*. Any differences in the *10th Edition* are clearly indicated.

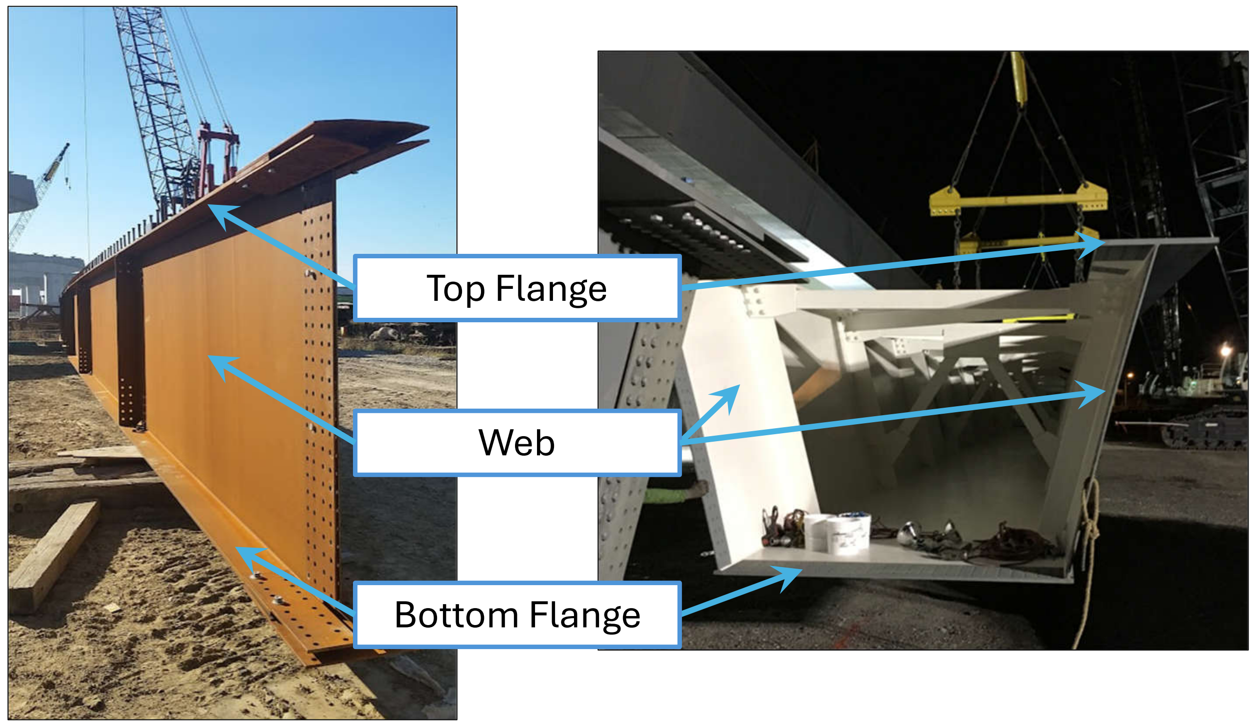
- Lesson 1: Intro to Bridges and Bridge Steels
- Lesson 2: Bridge Planning and Layout
- Lesson 3: Loads
- Lesson 4: Methods of Analysis
- Lesson 5: Shear in Girders
- Lesson 6: Flexure Part 1 - Fundamental Calculations
- Lesson 7: Flexure Part 2 - Constructability, Service Limit State, and Fatigue & Fracture Limit States
- Lesson 8: Flexure Part 3 - Strength Limit State: Noncomposite Sections and Composite Sections in Negative Bending
- Lesson 9: Flexure Part 4 - Strength Limit State: Composite Sections in Positive Bending and Shear Connection
- Lesson 10: Flexure Part 5 - Bracing for Flexure
- Lesson 11A: Splices and Connections - General Concepts and Welded Connections
- Lesson 11B: Splices and Connections - Bolted Connections and Girder Field Splices
- Lesson 12: Tension and Compression Members
- Lesson 13: Bearings and Joints
- Lesson 14: Bridge Decks

https://www.aisc.org/nsba/design-and-estimation-resources/standard-bridge-plans/
https://www.aisc.org/nsba/design-and-estimation-resources/steel-bridge-design-handbook/
https://www.aisc.org/education/university-programs/ta-fundamentals-of-steel-bridge-engineering/

1. Introduction

Introduction

- Steel Girder Components / Terminology



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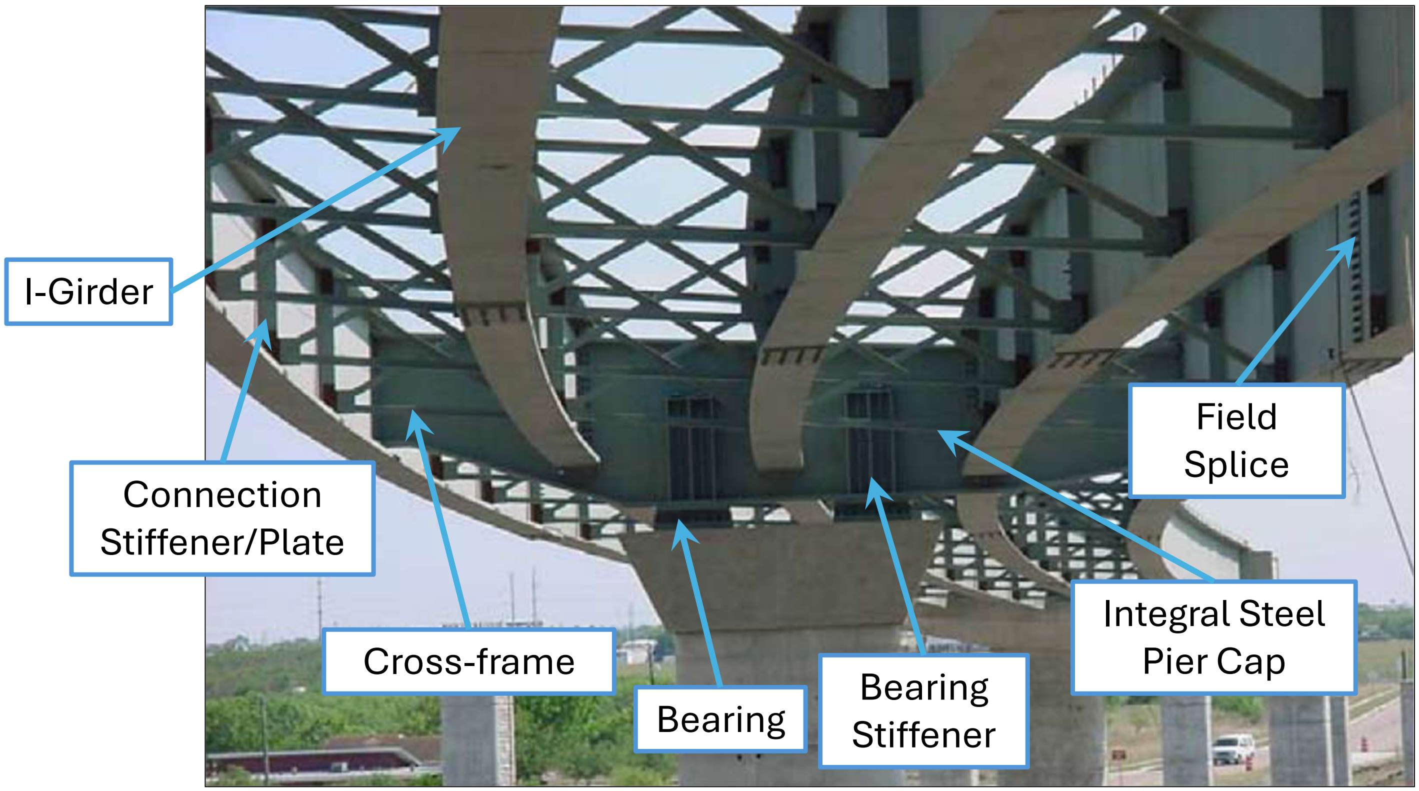

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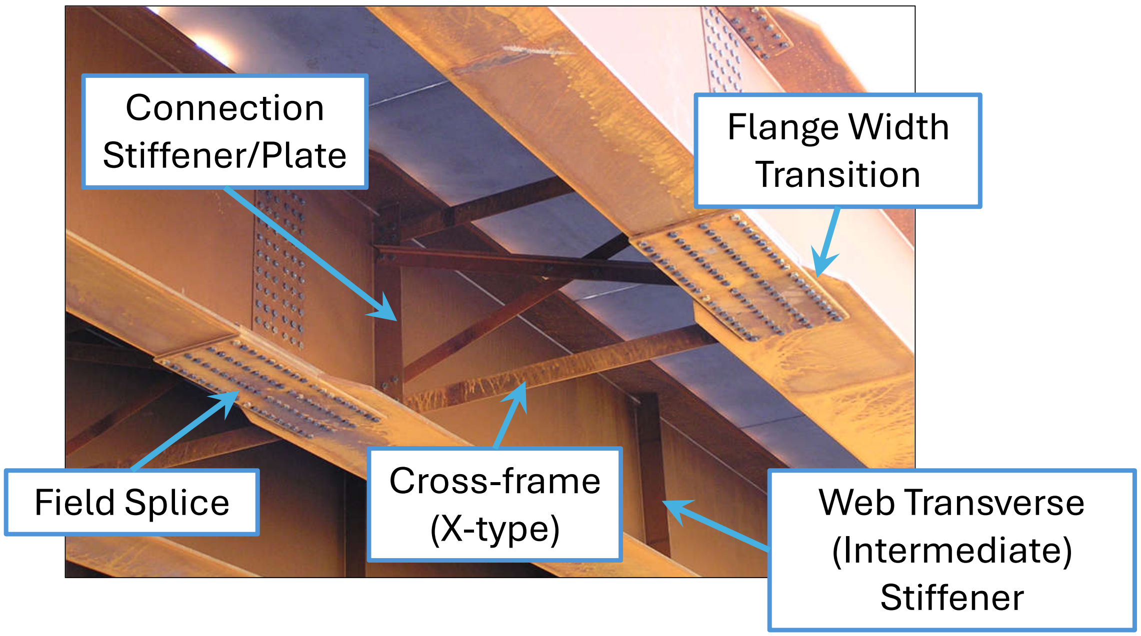

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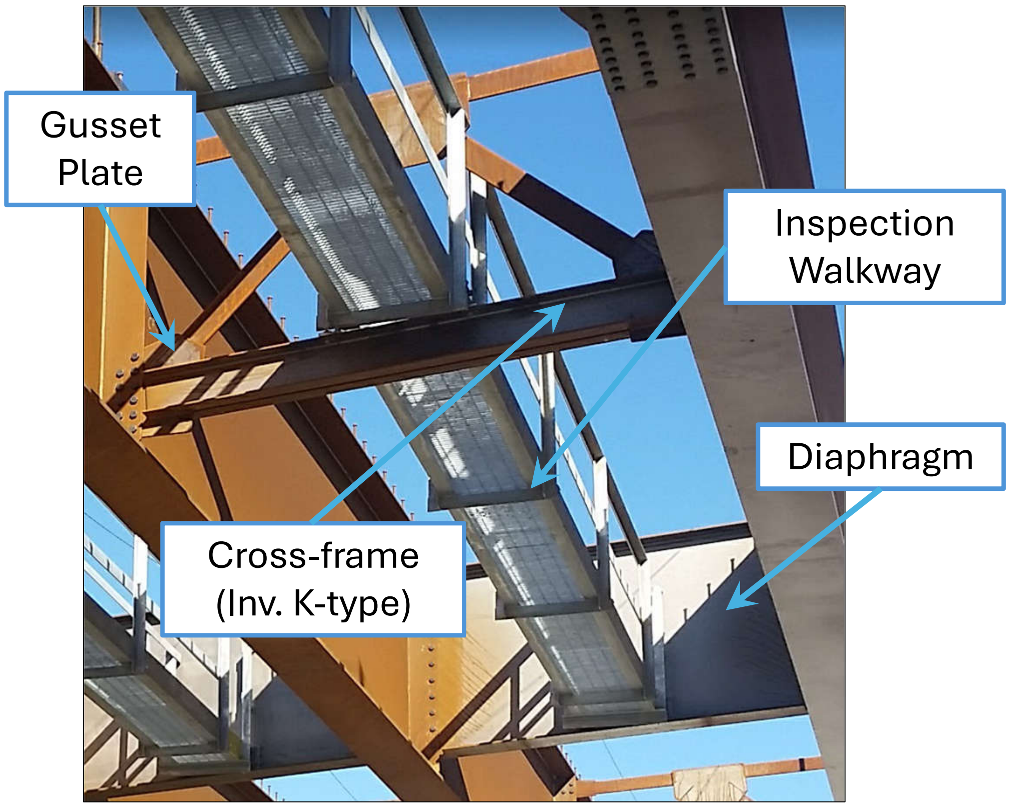

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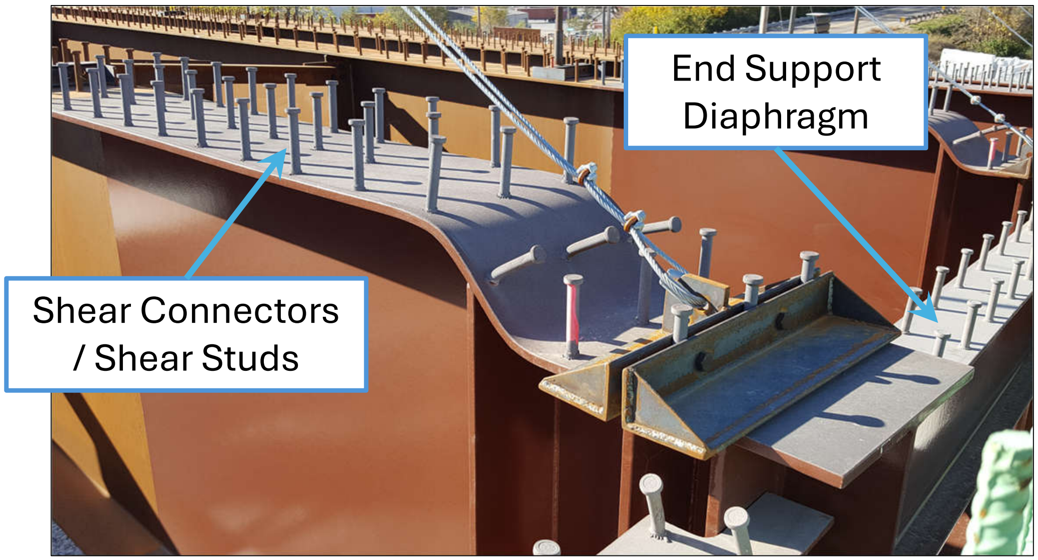

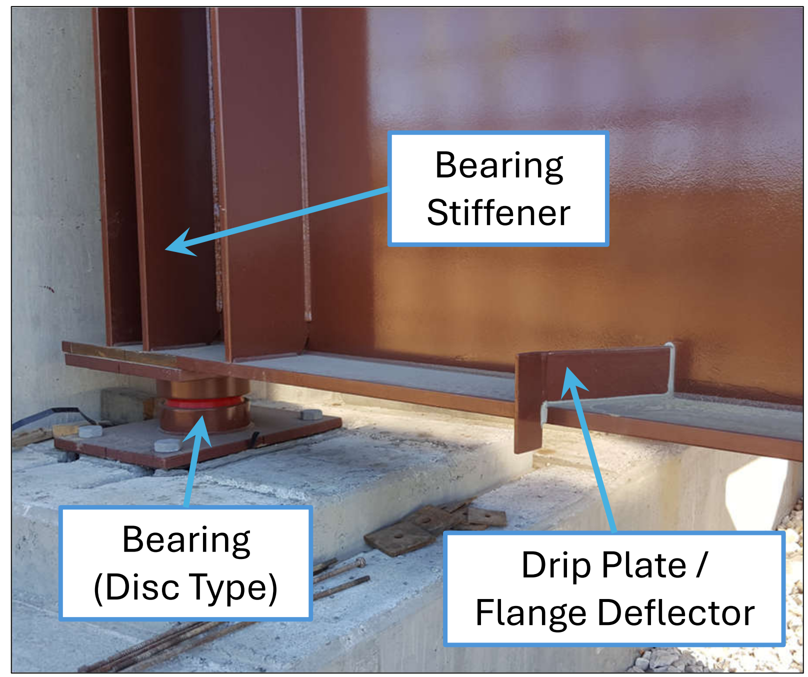

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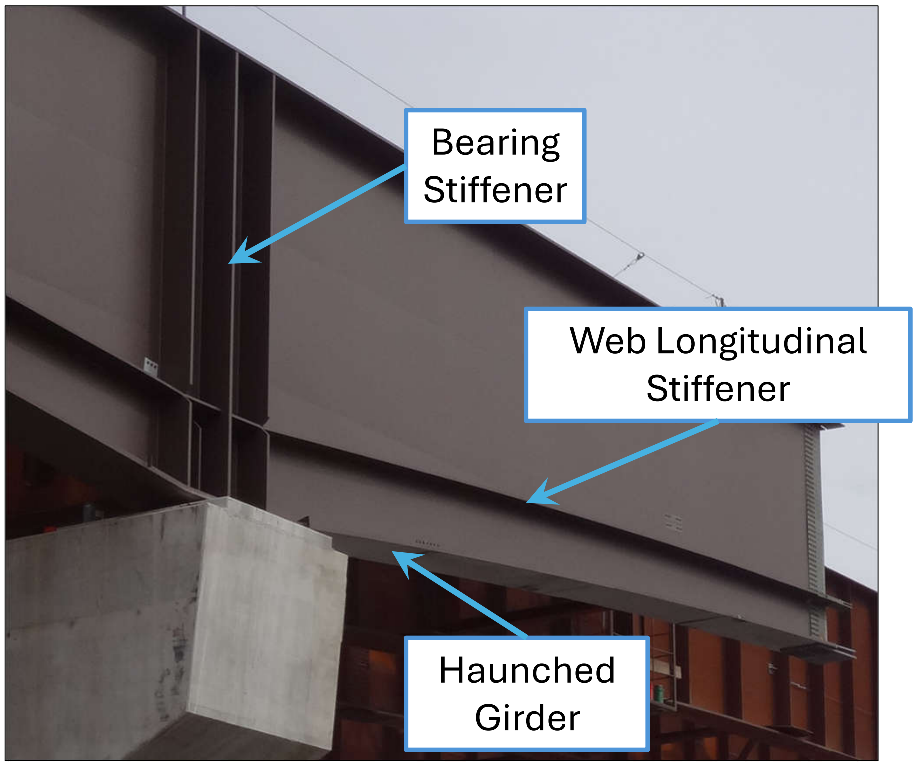

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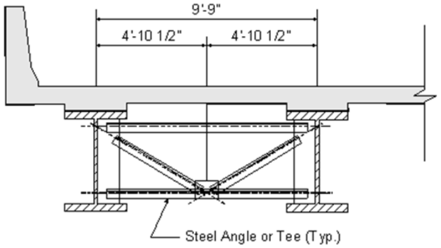
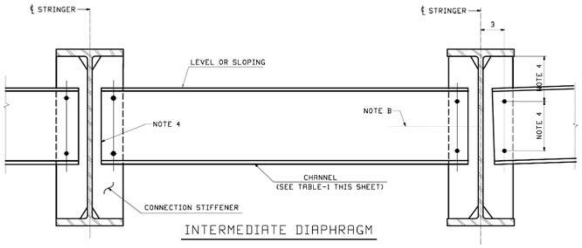


Cross-frames



Diaphragms

Introduction

- Steel Girder Components / Terminology
  - I-beam bottom flange bracing

  - Tub girder internal bracing

2. Requirements / Rules of Thumb / Good Practices

Requirements/Rules of Thumb/Good Practices

- As Frank Russo likes to say…

Bridge design is a unique combination of

- ***Shall / Must***
  - (AASHTO)
- ***Should***
  - (AASHTO Commentary)
- ***It would be good if …***
  - (NSBA Collaboration Documents)
- ***I wish you would …***
  - (other guidance, fabricator and erector preferences)
- ***Don't you dare …***
  - (avoid this at all costs)

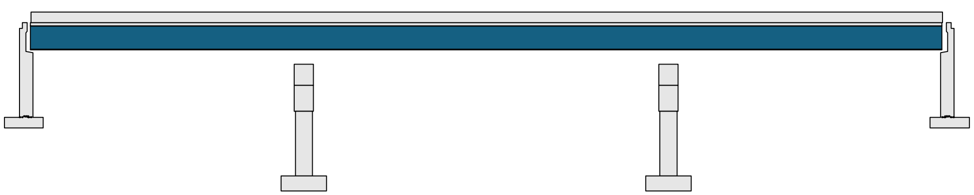
There are many good answers. The goal of this presentation is help you avoid the bad ones

Requirements/Rules of Thumb/Good Practices

It would be good if …

Span layout

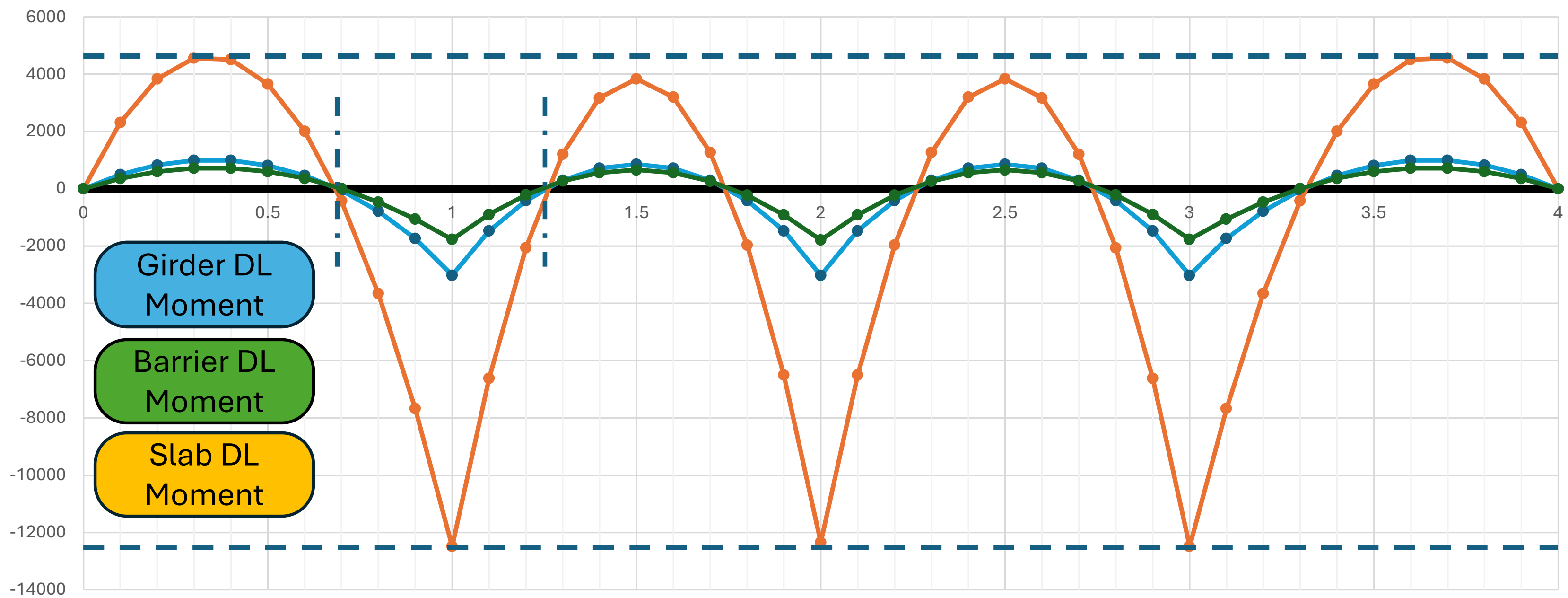
- For multi-span bridges, continuous layout generally preferred
- Balanced span arrangement – how?
- End spans 75% - 82% of center span



Balanced Span Arrangement

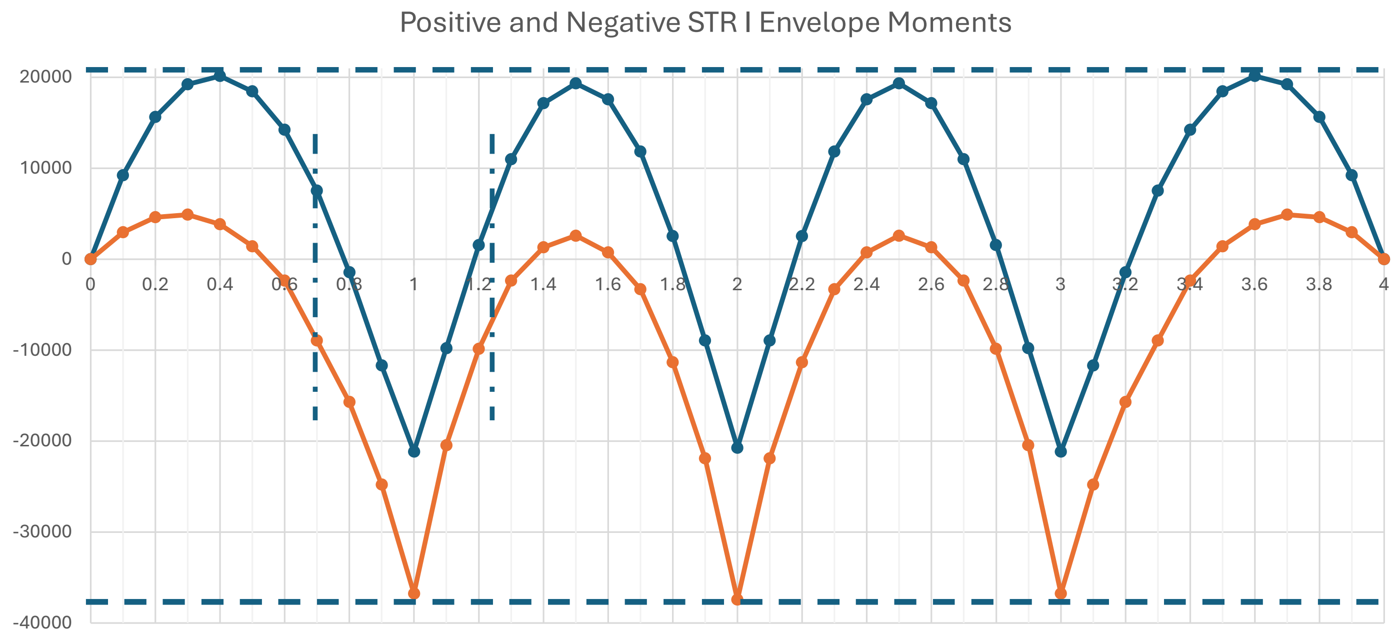
Requirements/Rules of Thumb/Good Practices

- Balanced spans
    - Example 4-Span – Strength I Envelope Moments
      199’-255’-255’-199’ (0.78L : 1.0L : 1.0L : 0.78L)



Requirements/Rules of Thumb/Good Practices

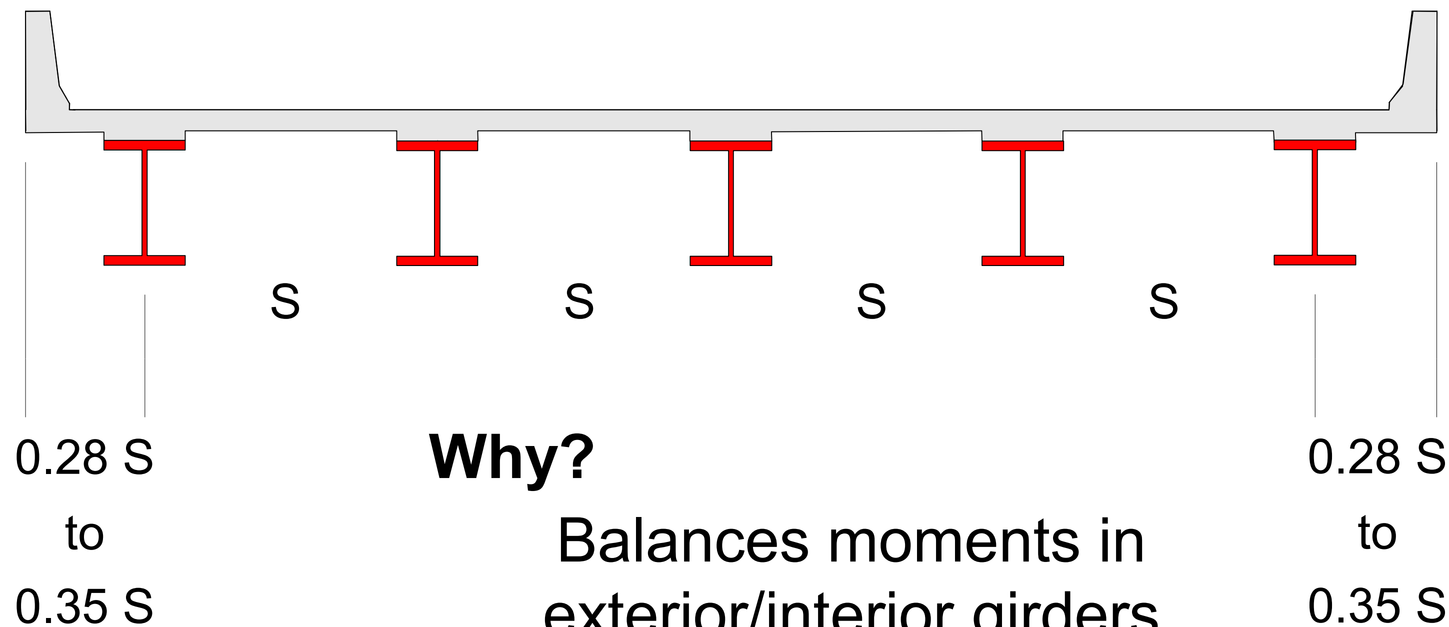
- Balanced spans
  - Example 4-Span – DL Moments
    199’-255’-255’-199’ (0.78L : 1.0L : 1.0L : 0.78L)



Requirements/Rules of Thumb/Good Practices

It would be good if …

- Spacings and overhangs



Why?

Balances moments in exterior/interior girders

Requirements/Rules of Thumb/Good Practices

It would be good if …

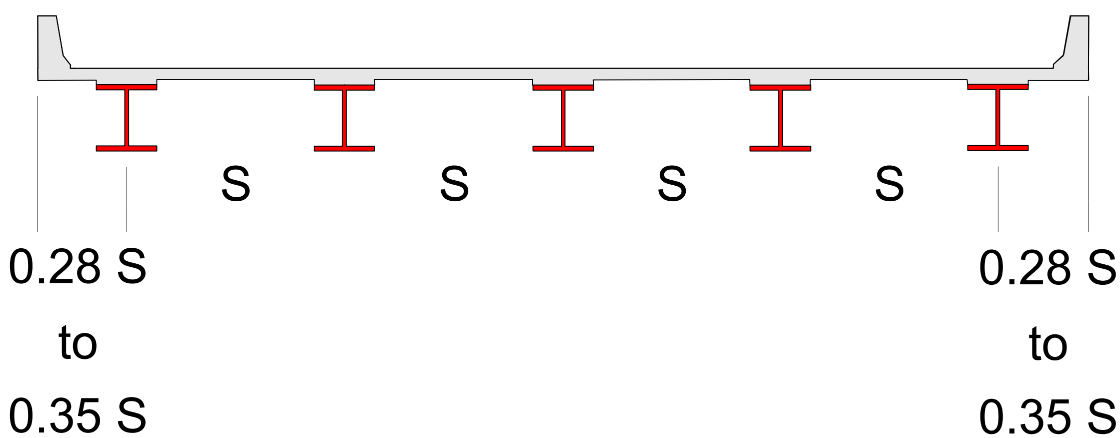
- Spacings and overhangs

2.5.2.7.1—Exterior Beams on Girder System Bridges

Unless future widening is virtually inconceivable, the load-carrying capacity of exterior beams shall not be less than the load-carrying capacity of an interior beam.

C2.5.2.7.1

This provision applies to any longitudinal flexural members traditionally considered to be stringers, beams, or girders.



Requirements/Rules of Thumb/Good Practices

Practical beam spacings

- 8 – 10 ft …. Everybody does this and has these in the inventory
- 12 ft …. Common, not as common
- 14 ft … getting to be a more unique design
- >14 ft, rare, limited by deck designs
- Some other considerations
  - Do you anticipate half width deck reconstruction in the future?
    - If so, you need an odd number of beams to have one down the middle.
  - How are your decks formed (SIP or lumber ?)

Requirements/Rules of Thumb/Good Practices

Practical beam spacings

- Minimize girder lines
  - When depth limitations aren't a factor
- Fabricator's perspective
  - Less welding per pound of steel
  - Fewer cross-frames
  - Fewer shop and field splices
  - Less material to inspect, coat, ship

- Erector's perspective
  - Fewer girders/field splices/cross-frames/bearings to erect/install
  - Stiffer structure – smaller differential deflections

Requirements/Rules of Thumb/Good Practices

Must or should depending on owner

Span to depth ratios – suggested

- AASHTO LRFD 2.5.2.6.3
- Beam portion (web)
  - 0.033L = L/30 (single span)
  - 0.027L (Continuous)
- Composite I-Beam
  - 0.040L = L/25 (single span)
  - 0.032L (Continouous)

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
Steel	Overall Depth of Composite I-Beam	0.040L	0.032L
	Depth of I-Beam Portion of Composite I-Beam	0.033L	0.027L

Note – the continuous span criteria assumes double-end continuity. For example, it can be rationalized to use 0.030L for end spans for the beam portion, and thus all spans

Requirements/Rules of Thumb/Good Practices

Flange proportioning limits

- AASHTO LRFD 6.10.2.2

Compression and tension flanges shall be proportioned such that:

$$\frac{b_f}{2t_f} \le 12.0, \tag{6.10.2.2-1}$$

$$b_f \ge D/6, \tag{6.10.2.2-2}$$

$$t_f \ge 1.1\, t_w, \tag{6.10.2.2-3}$$

$$0.1 \le \frac{I_{yc}}{I_{yt}} \le 10 \tag{6.10.2.2-4}$$

Shall/must

- AASHTO Commentary

  (NSBA recommendation)

$$b_{tfs} \ge \frac{L_{fs}}{85} \tag{C6.10.2.2-1}$$

It would be good if …
Don't dare not do this

Requirements/Rules of Thumb/Good Practices

Flange proportioning – negative moment regions

- Bottom Flange
  - Typically controlled by buckling at strength limit state
- Top Flange
  - Typically controlled by tension flange yielding at strength limit state
- Welded shop splice
  - Preferably off interior pier near first cross-frame
- Flanges in this region are often wider than positive moment region

Requirements/Rules of Thumb/Good Practices

Flange proportioning – positive moment regions

- Bottom Flange
  - Typically controlled by fatigue or tension flange yielding
- Top Flange
  - Typically controlled by constructability
  - Combined major axis and lateral bending due to deck casting
- Bottom flange will typically be larger than top flange.
- In large spans, there could be additional welded shop splices.

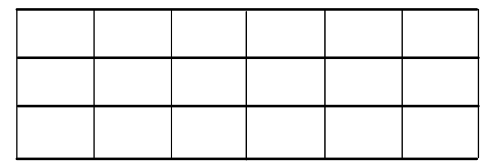
Requirements/Rules of Thumb/Good Practices

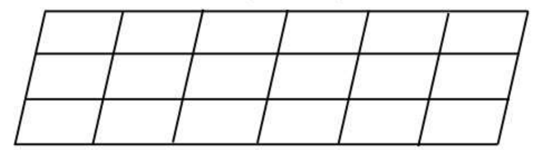
Cross-frame & diaphragm layout

- Cross-frames / diaphragms in straight or slightly skewed bridges generally do not have "as-designed forces"
  - What is "slightly skewed"?
  - Skew index = $I_S = \dfrac{w_g \tan\theta}{L_s}$ (4.6.3.3.2-2)
- Spacing?

C6.7.4.1

The arbitrary requirement for diaphragms or cross-frames spaced at not more than 25.0 ft in the AASHTO *Standard Specifications for Highway Bridges* (2002) has been replaced by a requirement for rational analysis to establish the necessary spacing according to the requirements specified herein.





- What governs their size if there are no computed loads?
  - You will soon find out

Requirements/Rules of Thumb/Good Practices

Cross-frames & diaphragms

- Geometry
  - Depth
    - **Make them at least 0.8 of girder depth**
  - Type – V, X, Inv V, Diaphragm
    - Aspect ratio (Girder Spa/Girder Depth)

6.7.4.2.1—General

Diaphragms or cross-frames for rolled beams and plate girders should be as deep as practicable, but at a minimum should be at least 0.5 of the beam depth for rolled beams and 0.75 of the girder depth for plate girders. Cross-frames in horizontally curved bridges should contain diagonals and top and bottom chords.

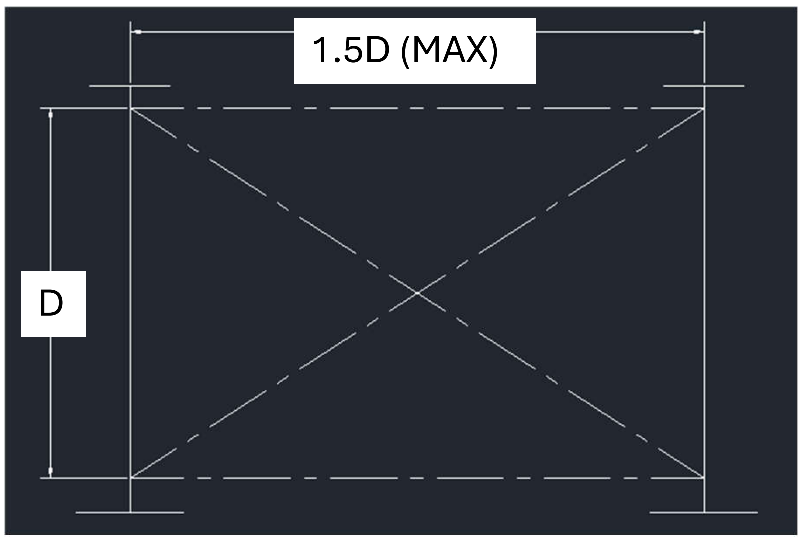
Diaphragms or cross-frames for rolled-beam and plate-girder bridges shall satisfy the stability bracing stiffness and strength requirements specified in Article 6.7.4.2.2, as applicable.

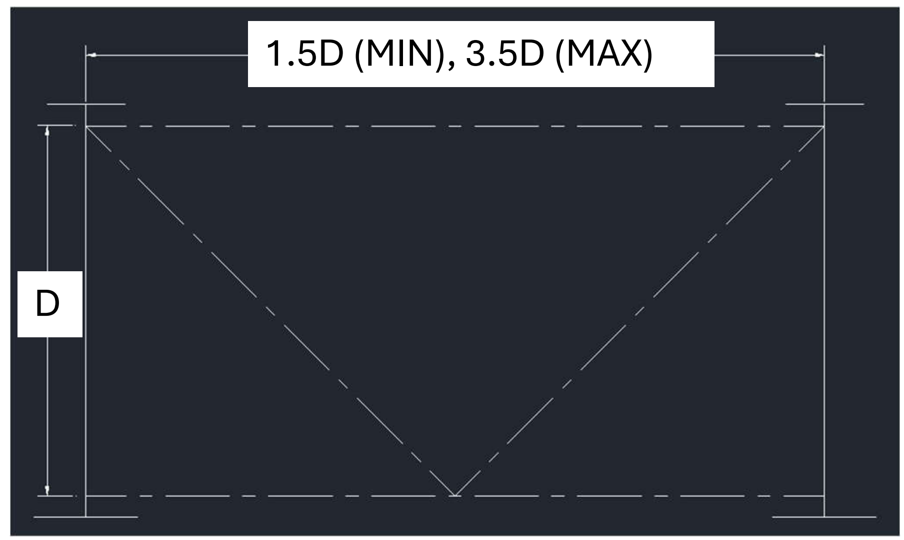



Requirements/Rules of Thumb/Good Practices

Cross-frames & diaphragms

- Aspect ratio ranges of applicability for cross-frame types



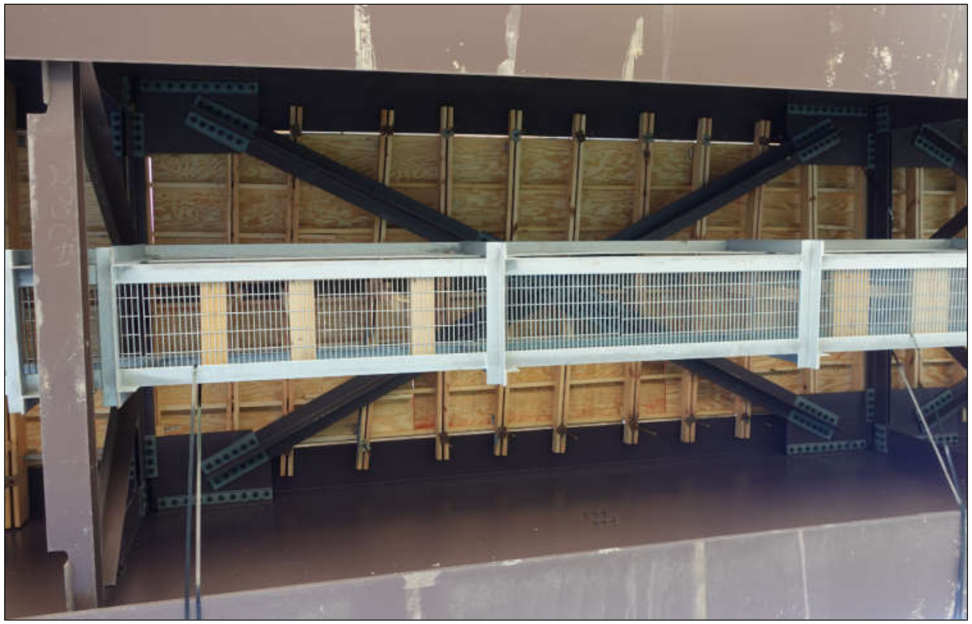


Requirements/Rules of Thumb/Good Practices

Lateral bracing

- Should be investigated for all stages of construction
- Stability requirements
- Wind during construction

- When we get up around 200 ft spans, take a closer look



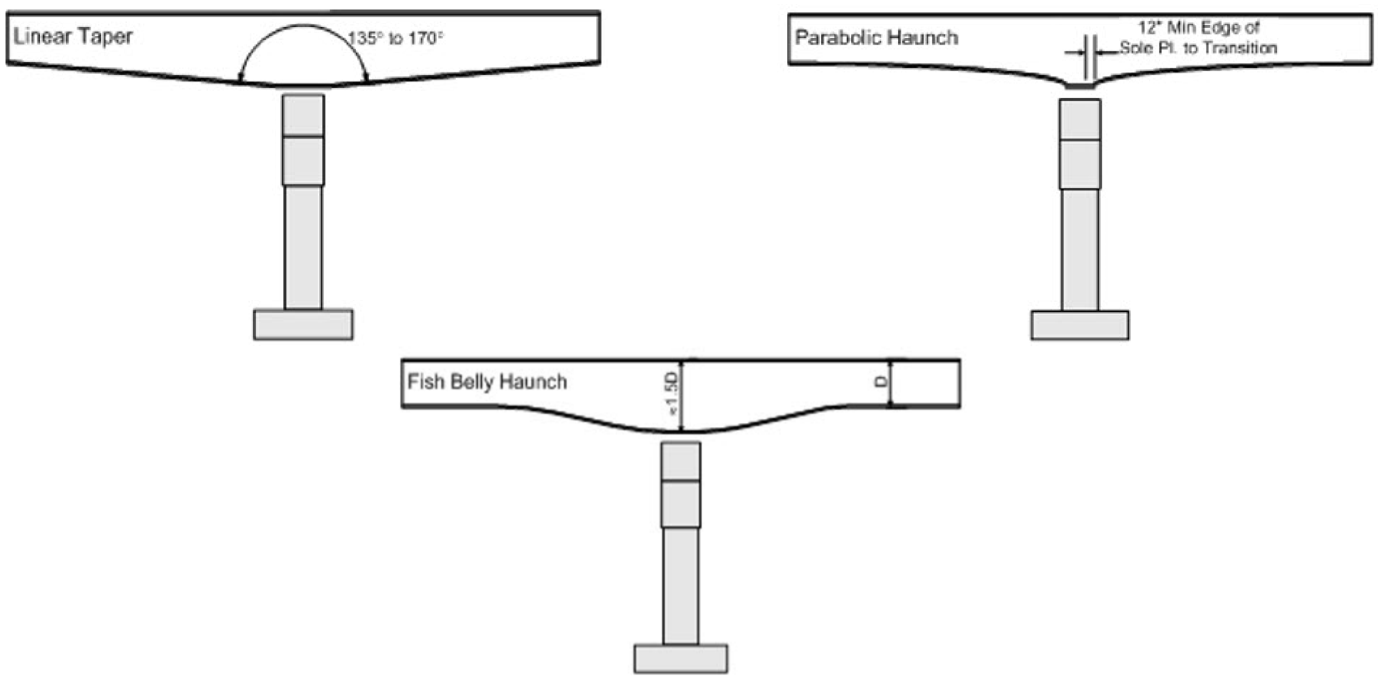
Requirements/Rules of Thumb/Good Practices

Variable web depth members

- Clearances, aesthetics, economics
- Typical linear or parabolic variation
- Minimum depths previously discussed – suggest applying to 10 percent of span away from bearing.
- AASHTO 6.10.1.4 for guidance

Requirements/Rules of Thumb/Good Practices

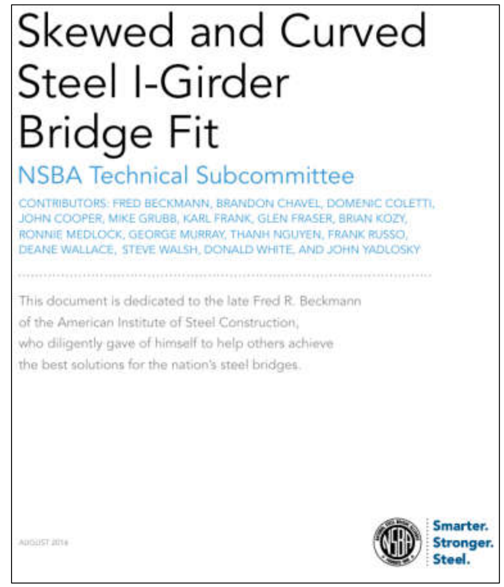
Variable web depth members



Requirements/Rules of Thumb/Good Practices

Fit condition

- When is it desired for the girders to be "plumb"?
  - The "fit" or "fit condition" of an I-girder bridge refers to the deflected girder geometry associated with a specific load condition in which the cross-frames or diaphragms are detailed to connect to the girders.



Skewed and Curved Steel I-Girder Bridge Fit

NSBA Technical Subcommittee

CONTRIBUTORS: FRED BECKMANN, BRANDON CHAVEL, DOMENIC COLETTI, JOHN COOPER, MIKE GRUBB, KARL FRANK, GLEN FRASER, BRIAN KOZY, RONNIE MEDLOCK, GEORGE MURRAY, THANH NGUYEN, FRANK RUSSO, DEANE WALLACE, STEVE WALSH, DONALD WHITE, AND JOHN YADLOSKY

This document is dedicated to the late Fred R. Beckmann of the American Institute of Steel Construction, who diligently gave of himself to help others achieve the best solutions for the nation's steel bridges.

AUGUST 2014

Smarter. Stronger. Steel.

Table 1 **Common Fit Conditions**

Loading Condition Fit	Construction Stage Fit	Description	Practice
No-Load Fit (NLF)	Fully-Cambered Fit	The cross-frames are detailed to fit to the girders in their fabricated, plumb, fully-cambered position under zero dead load.	The fabricator (detailer) sets the drops using the no-load elevations of the girders (i.e., the fully cambered girder profiles).
Steel Dead Load Fit (SDLF)	Erected Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge steel dead load at the completion of the erection.	The fabricator (detailer) sets the drops using the girder vertical elevations at steel dead load, calculated as the fully cambered girder profiles minus the steel dead load deflections.
Total Dead Load Fit (TDLF)	Final Fit	The cross-frames are detailed to fit to the girders in their ideally plumb as-deflected positions under the bridge total dead load.	The fabricator (detailer) sets the drops using the girder vertical elevations at total dead load, which are equal to the fully cambered girder profiles minus the total dead load deflections.

https://www.aisc.org/globalassets/nsba/technical-documents/skewed-and-curved-i-girder-bridge-fit-full-2016-revision.pdf

Requirements/Rules of Thumb/Good Practices

Fit condition

- Recommendations
- Article 6.7.2

The contract documents should state the fit condition for which the cross-frames or diaphragms are to be detailed for the following I-girder bridges:

- straight bridges where one or more support lines are skewed more than 20 degrees from normal;
- horizontally curved bridges where one or more support lines are skewed more than 20 degrees from normal and with an L/R in all spans less than or equal to 0.03; and
- horizontally curved bridges with or without skewed supports and with a maximum L/R greater than 0.03.

Table 3 **Recommended Fit Conditions for Straight I-Girder Bridges (including Curved I-Girder Bridges with *L/R* in all spans ≤ 0.03)[1]**

Square Bridges and Skewed Bridges up to 20 deg Skew			
	Recommended	Acceptable	Avoid
Any span length	Any		None
Skewed Bridges with Skew > 20 deg and $I_s \le 0.30$ +/-			
	Recommended	Acceptable	Avoid
Any span length	TDLF or SDLF		NLF
Skewed Bridges with Skew > 20 deg and $I_s > 0.30$ +/-			
	Recommended	Acceptable	Avoid
Span lengths up to 200 ft +/-	SDLF	TDLF	NLF
Span lengths greater than 200 ft +/-	SDLF		TDLF & NLF

Table 4 **Recommended Fit Conditions for Horizontally Curved I-Girder Bridges ($(L/R)_{MAX} > 0.03$)[1]**

Radial or Skewed Supports			
	Recommended	Acceptable	Avoid
$(L/R)_{MAX} \ge 0.2$	NLF [2,3]	SDLF [4]	TDLF
All other cases	SDLF	NLF	TDLF

3. Material Considerations

Material Considerations

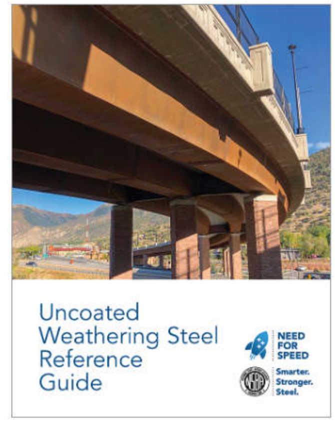
- AASHTO standards vs ASTM standards
- ASTM A709 -> AASHTO M270
  - Bridge steel
  - Additional toughness requirements
- Some ASTMs don't have an AASHTO counterpart

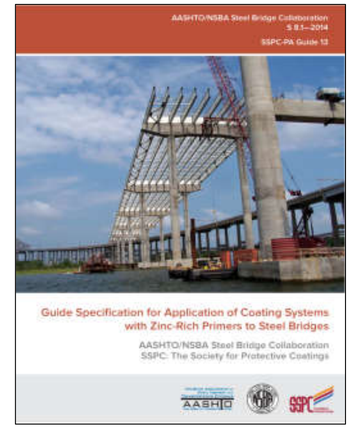
Table 6.4.1-1—Mechanical Properties of Structural Steel by Shape, Strength, and Thickness

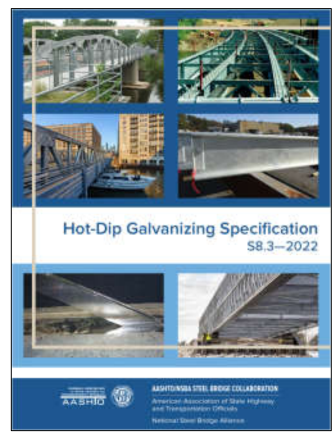
AASHTO Designation	M 270M/ M 270 Grade 36	M 270M/ M 270 Grade 50	M 270M/ M 270 Grade 50S	M 270M/ M 270 Grade 50W	M 270M/ M 270 Grade HPS 50W	M 270M/ M 270 Grade HPS 70W	M 270M/ M 270 Grade HPS 100W	
Equivalent ASTM Designation	A709/ A709M Grade 36	A709/ A709M Grade 50	A709/ A709M Grade 50S	A709/ A709M Grade 50W	A709/ A709M Grade HPS 50W	A709/ A709M Grade HPS 70W	A709/ A709M Grade HPS 100W	
Thickness of Plates, in.	Up to 4.0 incl.	Up to 4.0 incl.	Not Applicable	Up to 4.0 incl.	Up to 4.0 incl.	Up to 4.0 incl.	Up to 2.5 incl.	Over 2.5 to 4.0 incl.
Product Forms	All Groups	All Groups	All Groups	All Groups	N/A	N/A	N/A	N/A
Minimum Tensile Strength, F_u, ksi	58	65	65	70	70	85	110	100
Specified Minimum Yield Point or Specified Minimum Yield Strength, F_y, ksi	36	50	50	50	50	70	100	90

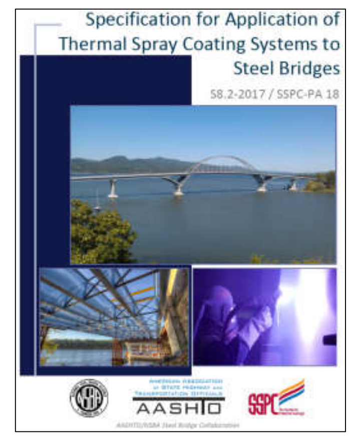
Material Considerations

- Corrosion Resistance
  - Weathering Steel
    - Usage guidance in Reference Guide
  - Paint coatings
  - Galvanizing
  - Metallizing (thermal sprayed coatings)





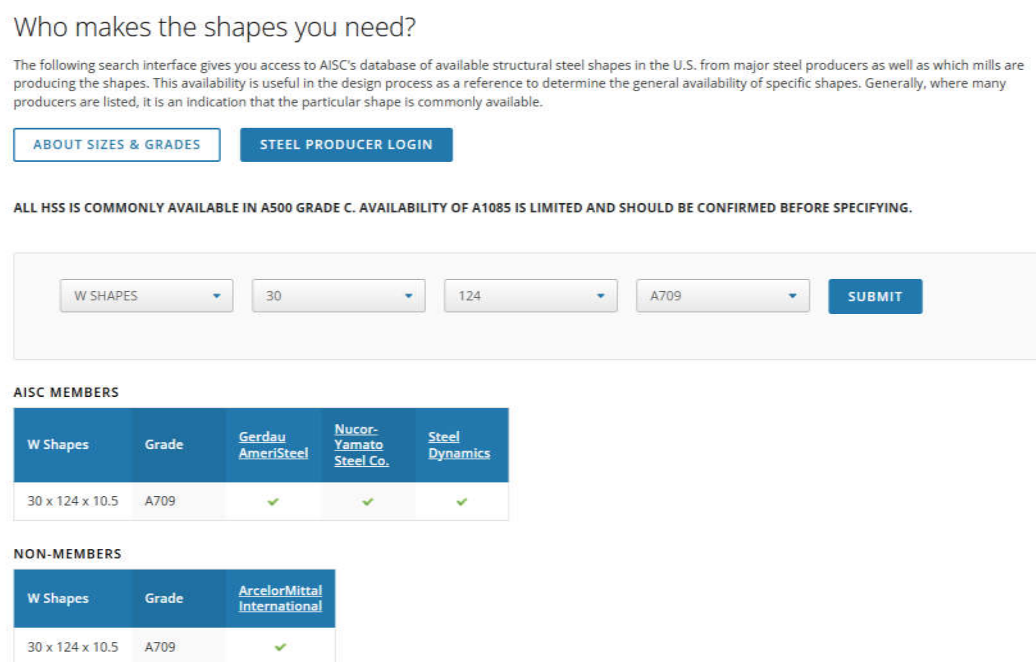




https://www.aisc.org/nsba/design-resources/uncoated-weathering-steel-reference-guide/
https://www.aisc.org/nsba/design-and-estimation-resources/aashto-nsba-collaboration/aashto-nsba-collaboration2/

Material Considerations

- Procurement
  - Lead times (grade, thickness, markets, shapes, plate)
  - Shape availability
  - Plate availability



Who makes the shapes you need?

The following search interface gives you access to AISC's database of available structural steel shapes in the U.S. from major steel producers as well as which mills are producing the shapes. This availability is useful in the design process as a reference to determine the general availability of specific shapes. Generally, where many producers are listed, it is an indication that the particular shape is commonly available.

ABOUT SIZES & GRADES

STEEL PRODUCER LOGIN

ALL HSS IS COMMONLY AVAILABLE IN A500 GRADE C. AVAILABILITY OF A1085 IS LIMITED AND SHOULD BE CONFIRMED BEFORE SPECIFYING.

W SHAPES | 30 | 124 | A709 | SUBMIT

AISC MEMBERS

W Shapes	Grade	Gerdau AmeriSteel	Nucor-Yamato Steel Co.	Steel Dynamics
30 x 124 x 10.5	A709	✓	✓	✓

NON-MEMBERS

W Shapes	Grade	ArcelorMittal International
30 x 124 x 10.5	A709	✓

Material Considerations

Plate Availability

The length availability for the various plate widths and thicknesses is a very common question engineers have when designing highway structures. Understanding the availability of plate material while performing design iterations will ensure that the material used can be sourced from at least one of the domestic steel mills and result in a better economy for the overall bridge superstructure.

View the tables below, and check out our *Modern Steel Construction* article **Steel Plate Availability for Highway Bridges** (published October 2023).

MAXIMUM PLATE LENGTH AVAILABILITY (INCHES) -- ASTM A709 GRADE 50 & 50W

All lengths are readily available from three domestic mills.

Plate Thickness	Plate Width--Grade 50 & 50W								
	72	**78**	**84**	**90**	**96**	**102**	**108**	**114**	**120**
⅜	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034
½	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034
9/16	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034
⅝	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034
¾	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034	1,034
⅞	1,034	1,034	1,034	1,034	1,034	1,034	1,026	972	923
1	1,034	1,034	1,034	1,034	1,034	1,030	980	680	680

https://www.aisc.org/nsba/design-and-estimation-resources/plate-availability/

Steel Plate Availability for Highway Bridges

Christopher Garrell, PE, And Travis Hopper, PE

7 Minutes

An overview of plate sizes commonly produced by domestic mills.

A QUESTION MANY ENGINEERS encounter when designing highway bridge structures is the availability of various plate lengths, widths, and thicknesses. The possibilities and options can seem infinite and overwhelming. However, understanding the availability of plate material while performing design iterations will ensure that the material specified can be readily sourced from domestic steel mills, which usually yields improved fabrication speed and better economy for the overall bridge superstructure.

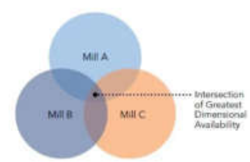


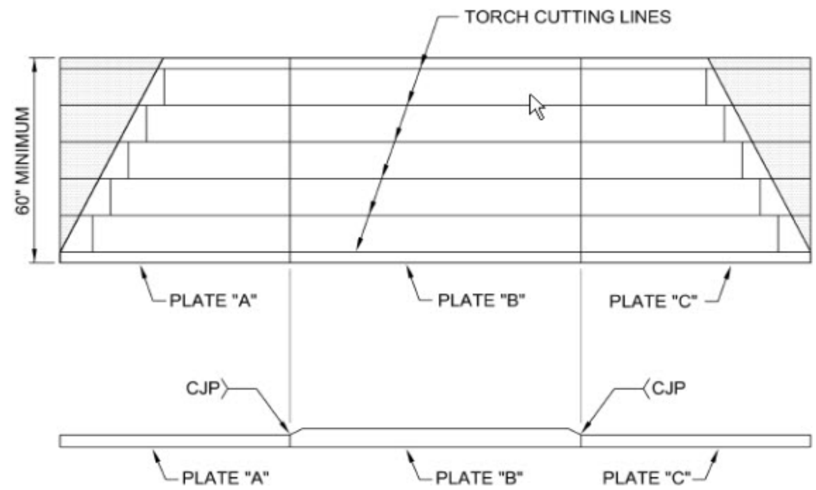
Fig. 1. Rationalization of plate thickness, width, and...

The information listed in this article is not intended to be an all-encompassing summary of available plates that a mill may be able to produce. It is intended to provide an overview of where the thicknesses, widths, and lengths produced by each mill intersect with one another resulting in the greatest dimensional availability (Figure 1). Other widths, thicknesses, and lengths may be available from one or more of these producers. In cases where a dimension is not shown, one should consult the steel mill or a local steel bridge fabricator. More information can be found on the AISC Certification page under "Find a Certified Company" page (aisc.org/certification). Alternatively, feel free to contact an NSBA Regional Bridge Steel Specialist (see sidebar on page 19). The AISC Steel Solutions Center can also assist you by phone at 866.ASK.AISC and online at

Material Considerations

Fabrication

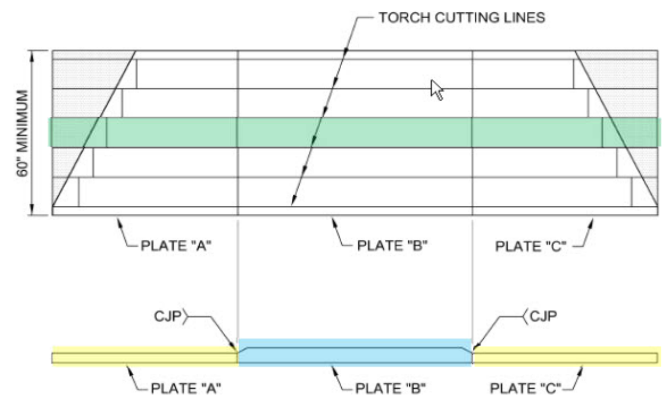
- Flanges
  - Flanges of desired thickness are cut from the ordered plate into the desired segments.
  - Preferably, if the design facilitates it, slabs are welded together first, and then the flanges are cut (“stripped”) from the welded slabs (a process known as “slab welding”)
    - If flange segments are unique, flanges are stripped first and then welded.

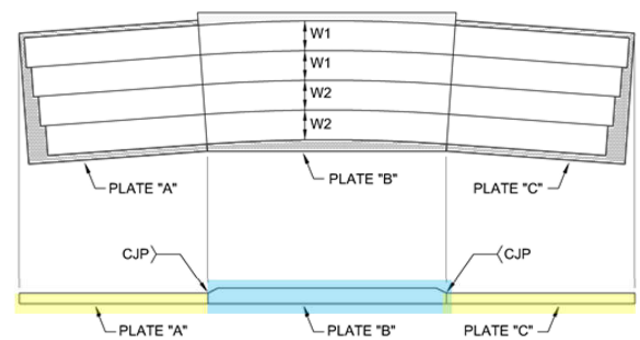


Material Considerations

Fabrication

- Flanges
  - Rules of thumb…
    - $t_f \geq 0.75$ in.; $b_f \geq 12.0$ in.
    - 1/8 in. increments up to 2 ½ in., then ¼ in.
    - Practical thickness limit of 3 in. (Spec. up to 4 in.)
  - Limit number of plate thicknesses.
  - Keep same flange width in field sections.
    - Only change thickness at welded shop splices
    - Thinner plate ≥ ½ thickness of thicker plate
  - No more than two butt splices in field section
  - Think about plate availability lengths

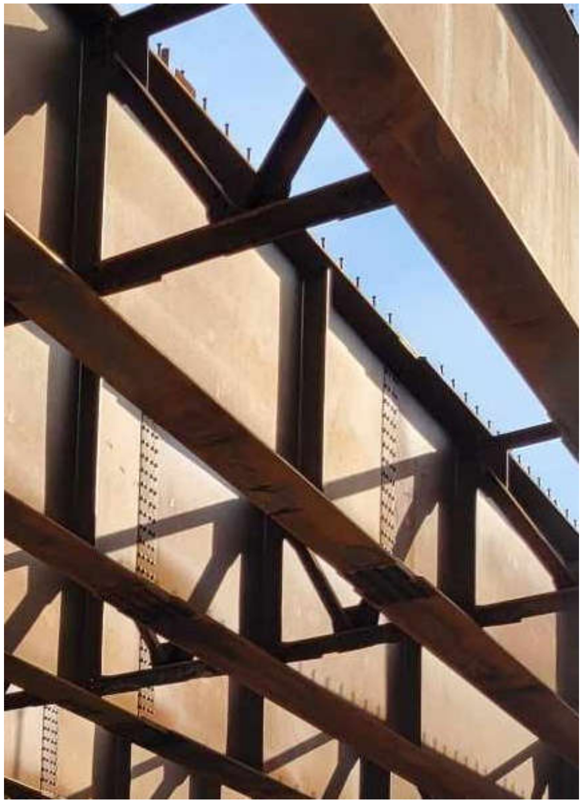




Material Considerations

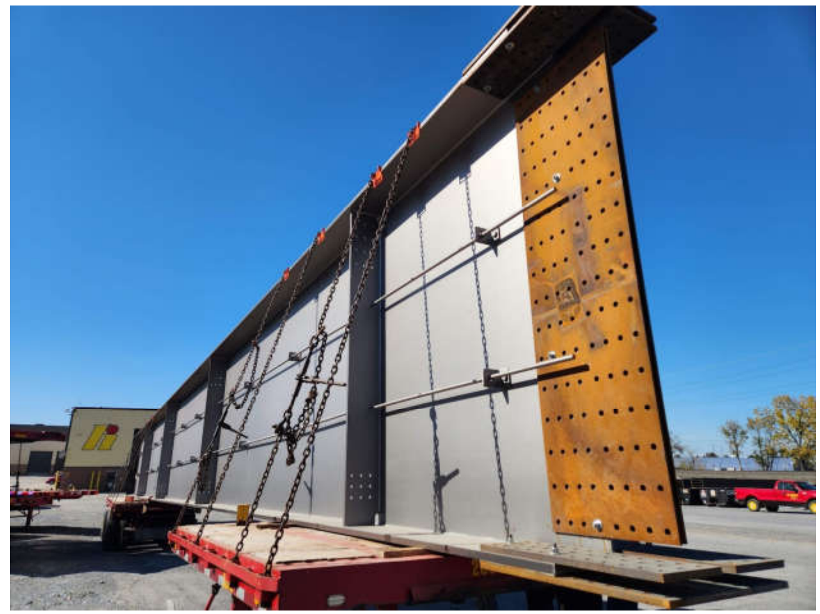
Fabrication

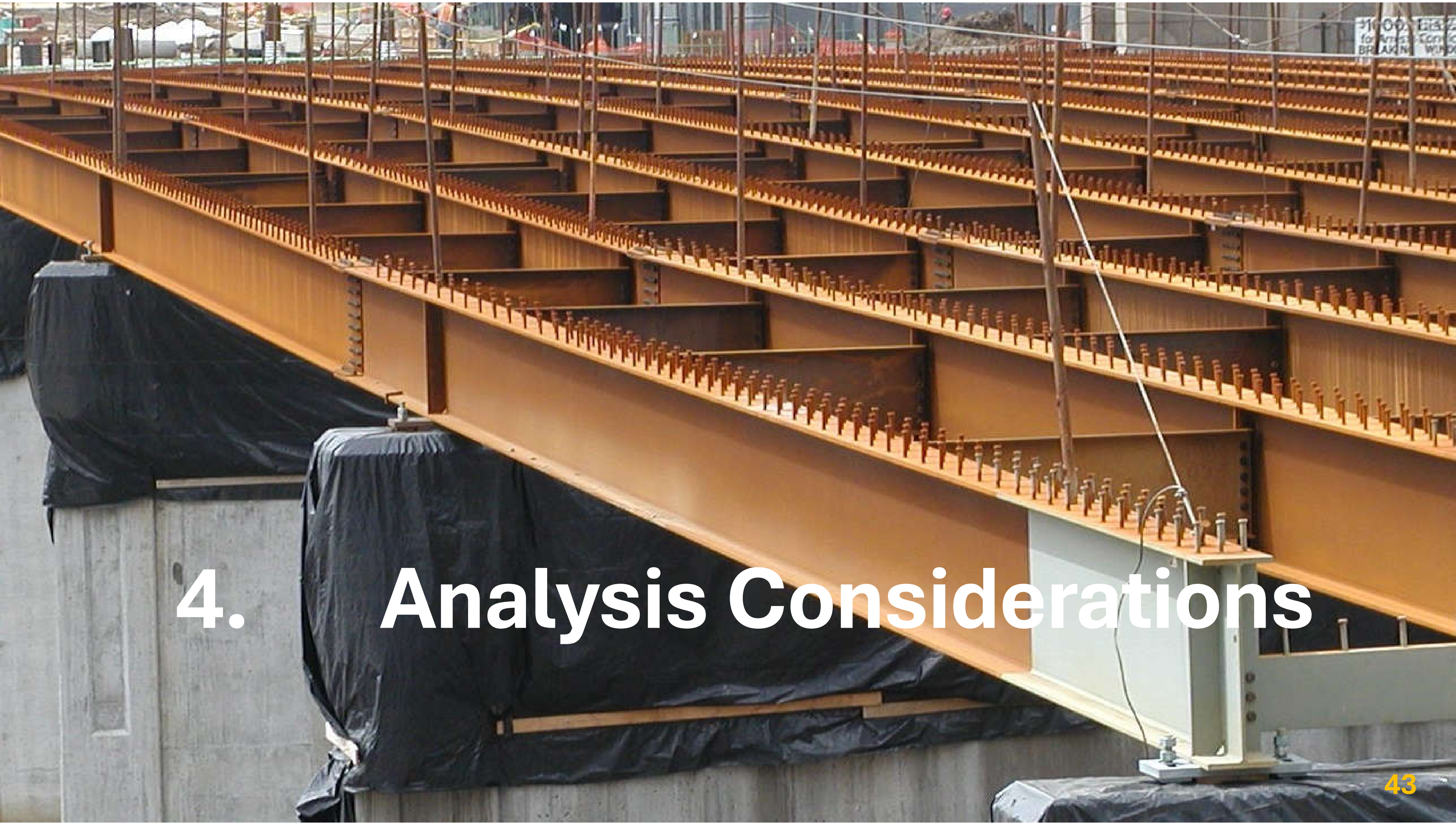
- Webs
  - Depth considerations
  - Avoid longitudinal stiffeners on routine bridges
  - Unstiffened or "Partially stiffened"
  - Consider fabrication and transportation



Material Considerations

- Girder field section lengths
  - Common field sections are up to 140 ft long and 50 tons.
  - Many owners have guidance on max field section lengths.
  - Consult fabricators early in the design process for constraints.

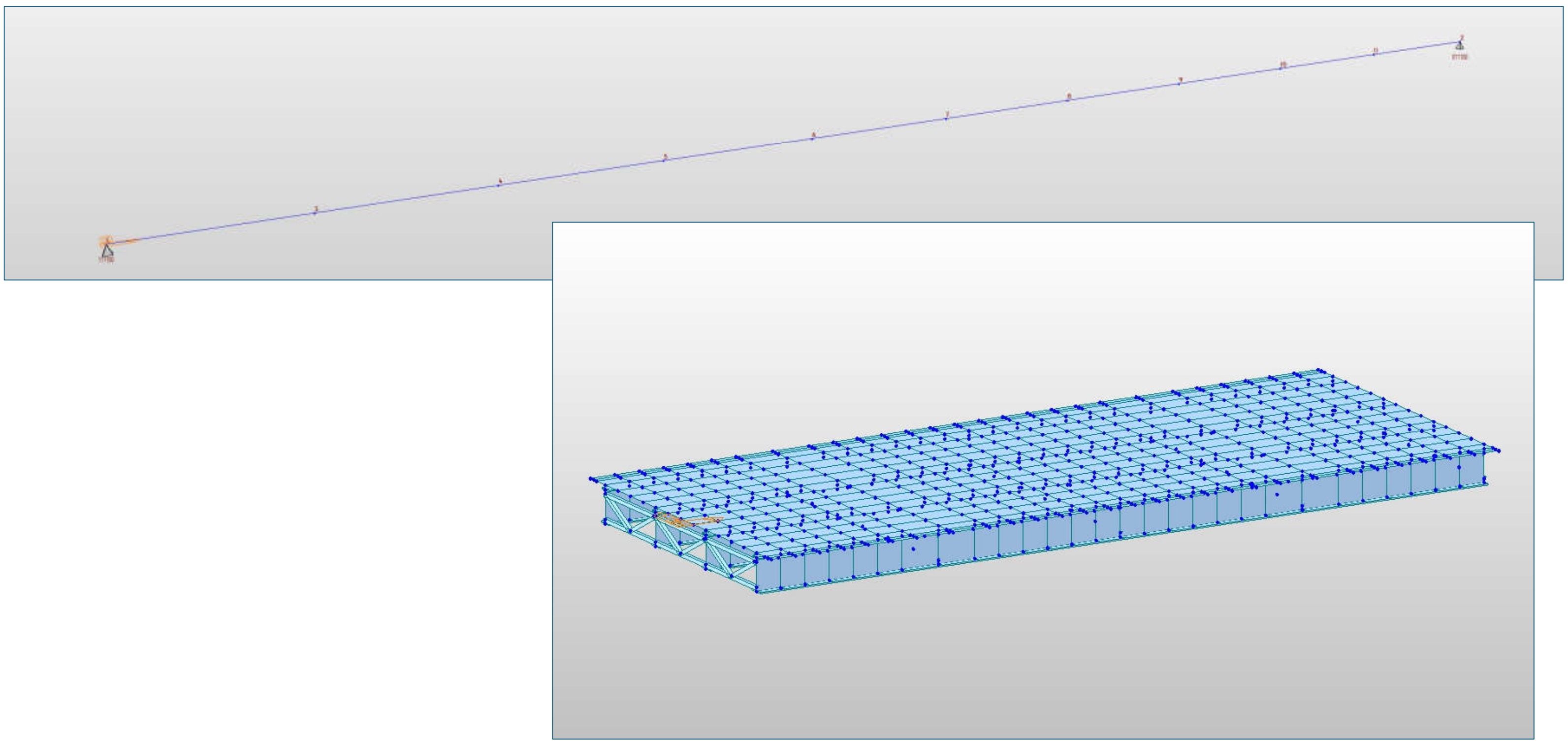




4. Analysis Considerations

Analysis Considerations

- Appropriate level of analysis?



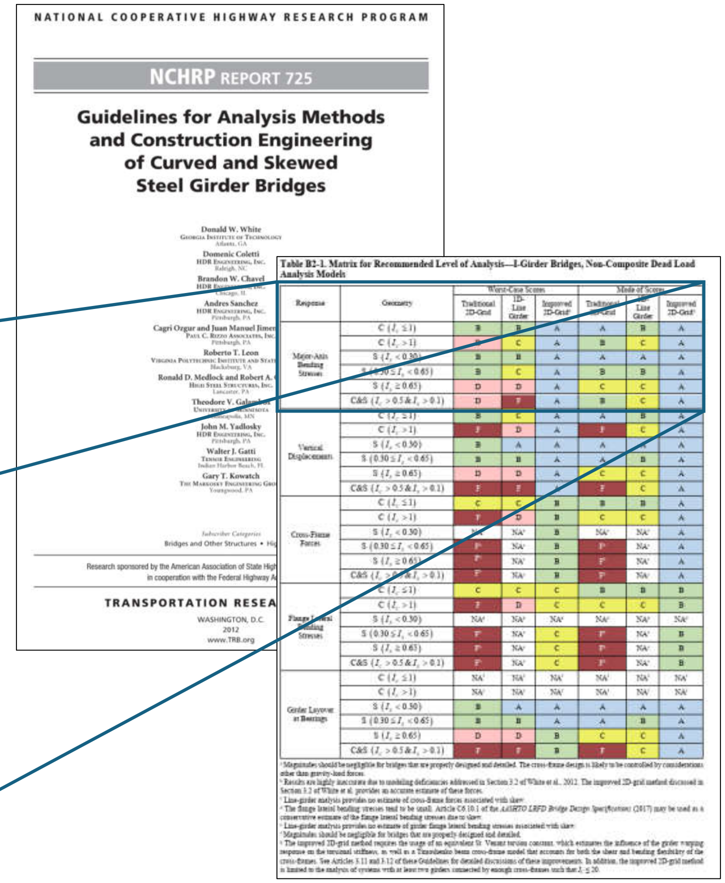
Analysis Considerations

- Appropriate level of analysis?
  - Hand calcs?
  - Simple computer models?
  - Detailed computer models?
  - What do we want out of the model?
    - Forces? Displacements? Reactions?
- Understand implications/limitations

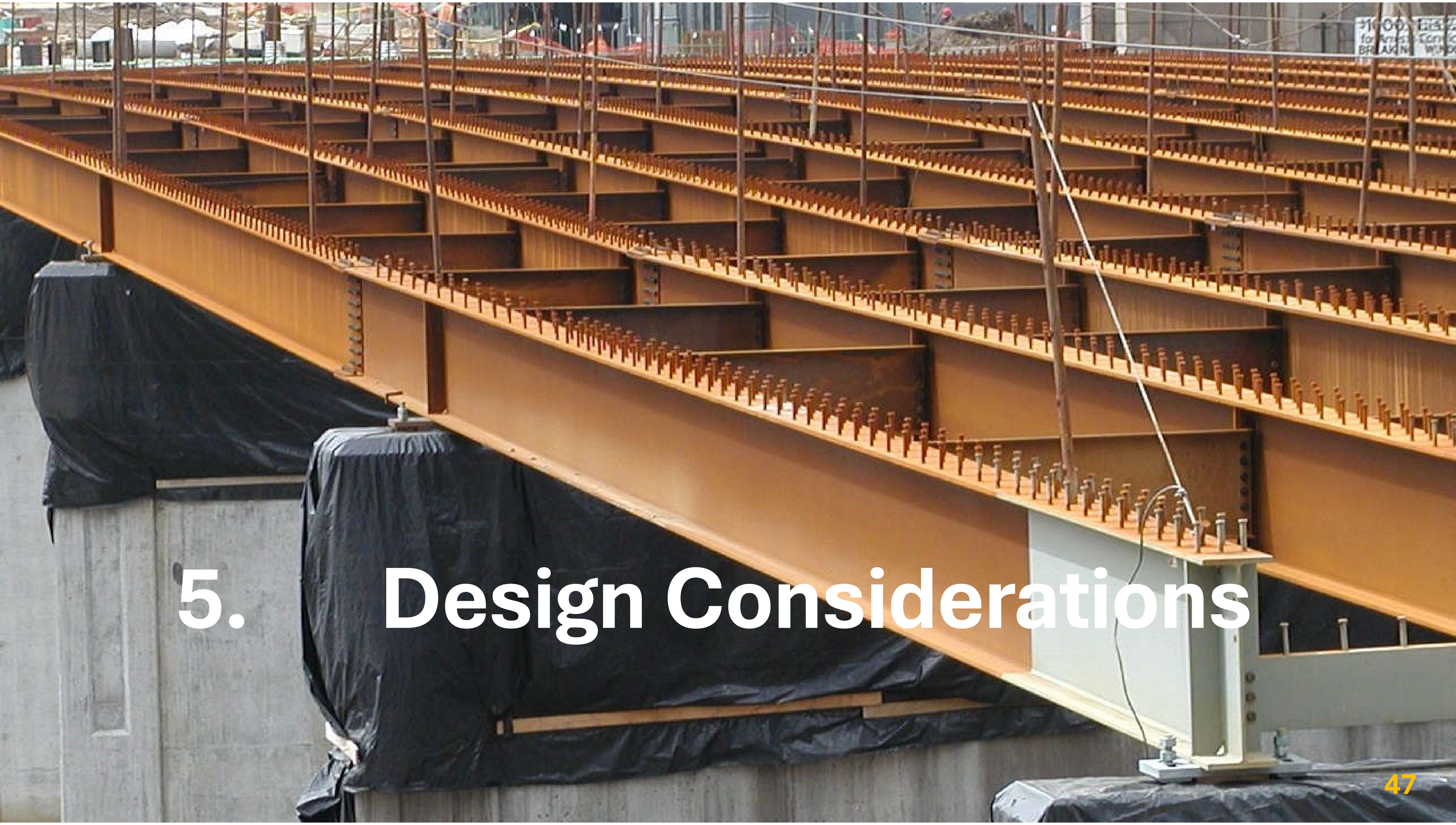
In the end, create the simplest model to get the desired results

Analysis Considerations

- Appropriate level of analysis?
  - Resources
    - NCHRP Report 725
    - AASHTO/NSBA Collaboration G13.1
  - Recommendations on methods of analysis
    - Ability of 1D and 2D methods compared to 3D



Response	Geometry	Worst-Case Scores			Mode of Scores		
		Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid[g]	Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid[g]
Major-Axis Bending Stresses	C $(I_c \le 1)$	B	B	A	A	B	A
	C $(I_c > 1)$	D	C	A	B	C	A
	S $(I_s < 0.30)$	B	B	A	A	A	A
	S $(0.30 \le I_s < 0.65)$	B	C	A	B	B	A
	S $(I_s \ge 0.65)$	D	D	A	C	C	A
	C&S $(I_c > 0.5 \,\&\, I_s > 0.1)$	D	F	A	B	C	A



5. Design Considerations

Design Considerations

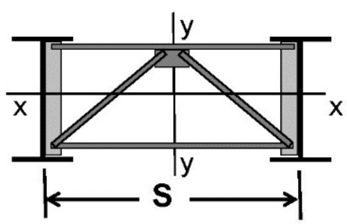
Let’s talk about a few topics related to design

- Stability
  - Global stability
  - Stability bracing
- Wind load during construction

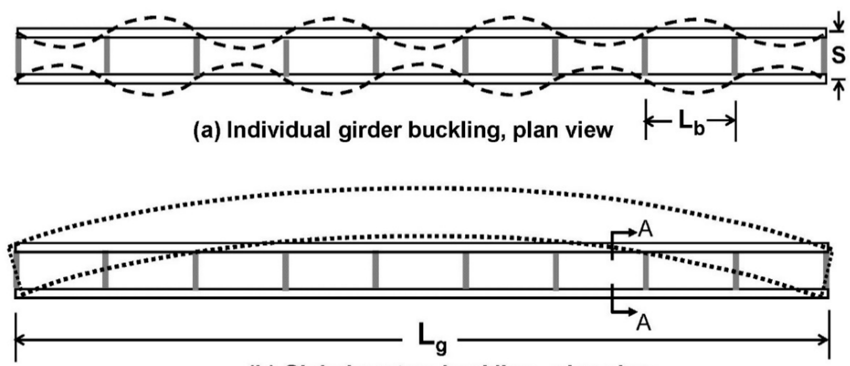
Design Considerations

Global buckling of narrow steel units

- For a twin-girder system: L_b vs L_g
  - Bracing spacing controls individual girder lateral-torsional buckling
  - Bracing size and spacing doesn't control system buckling



(c) System cross section A-A

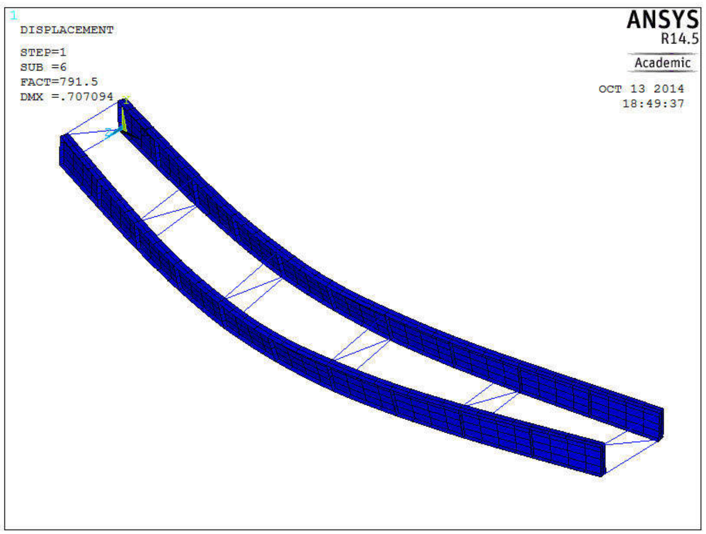


(a) Individual girder buckling, plan view

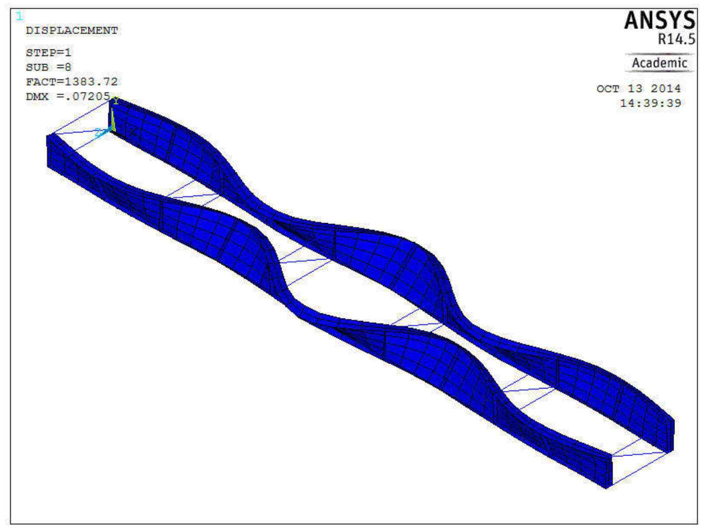
(b) Global system buckling , plan view

Design Considerations

Global buckling of narrow steel units



Global Buckling
(M_{cr} = 792 k-ft)



Buckling Between Cross-Frames
(M_{cr} = 1384 k-ft)

Design Considerations

System buckling of narrow steel units (three or fewer girders)

$$M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

Where:

- Mgs = nominal buckling resistance of the **girder system** (k-in)
- w_g= spacing of twin girders or for 3 girder system use spacing between the two exterior girders (in)
- E = modulus of elasticity of steel girder (ksi)
- L = length of span under consideration (in)

Design Considerations

System buckling of narrow steel units (three or fewer girders)

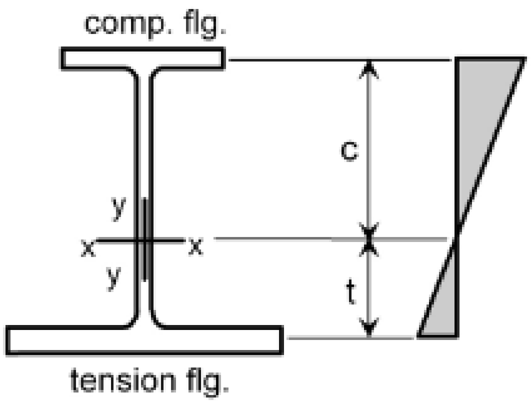
$$M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

- C_{bs} = system moment gradient modifier
  = 1.1 for simply-supported units
  = 2.0 for continuous-span units
- Ix = Non-composite single girder strong-axis moment of inertia
- For non-prismatic girder properties – AASHTO recommends a length-weighted average for Ix, and $Ieff$.

Design Considerations

System buckling of narrow steel units (three or fewer girders)

$$M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$



$$\mathrm{I}_{eff} = I_{yc} + (t/c) I_{yt}$$

Design Considerations

System buckling of narrow steel units (three or fewer girders)

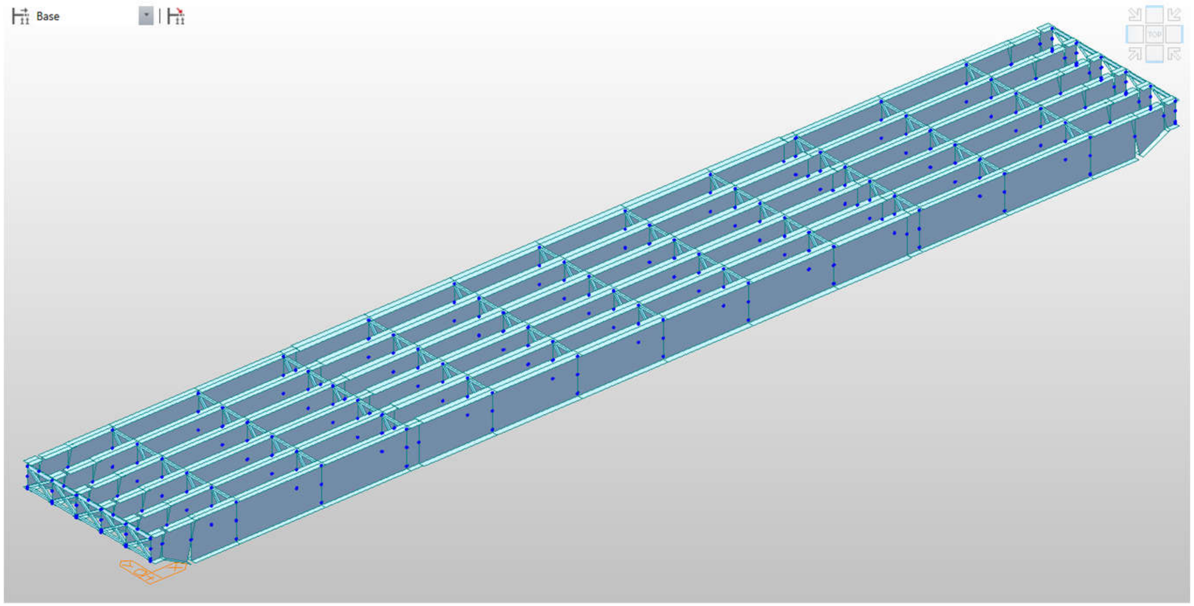
$$M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

Considering all of the girders across the width of the unit within the span, the sum of the largest total factored moments during deck placement should not exceed 70% of Mgs.

Design Considerations

System buckling of narrow steel units (three or fewer girders)

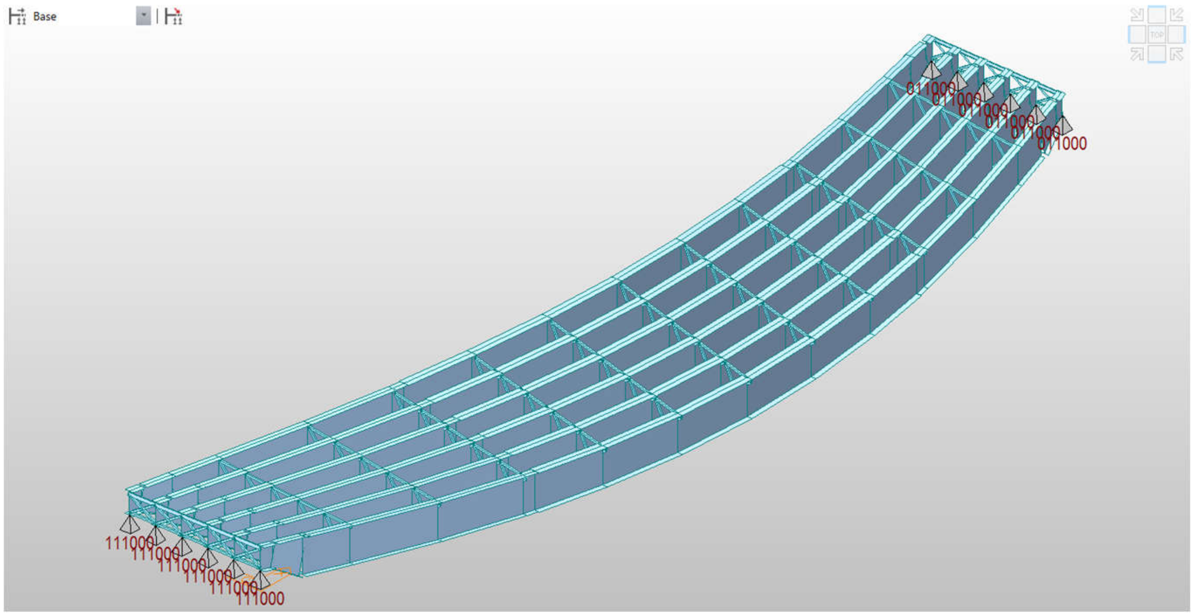
- Example: 283 ft single span I-girder bridge



Design Considerations

System buckling of narrow steel units (three or fewer girders)

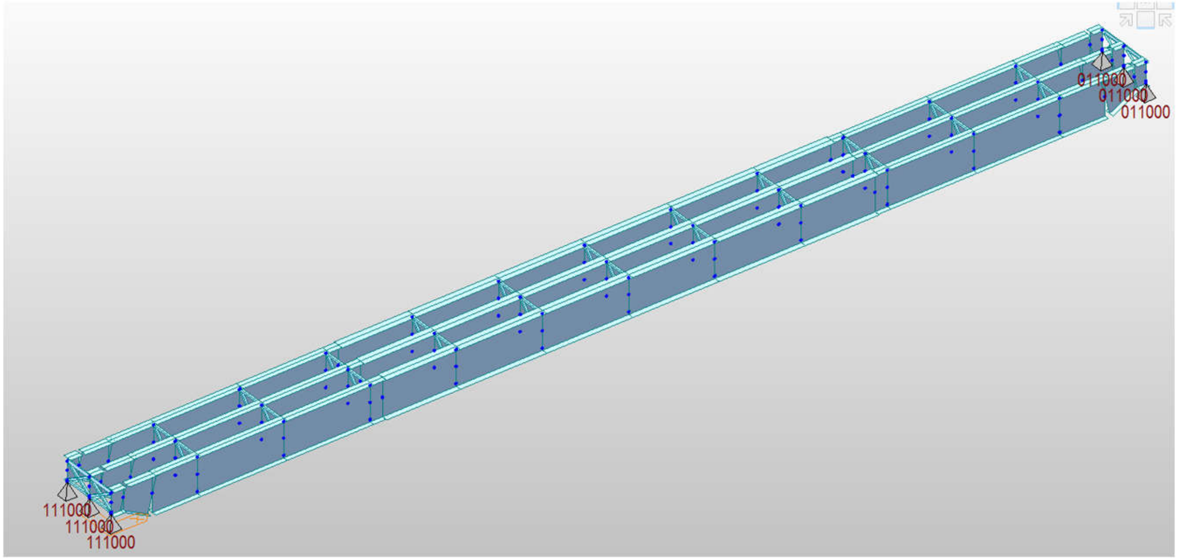
- Deck cast eigenvalue = 1.68 ☺



Design Considerations

System buckling of narrow steel units (three or fewer girders)

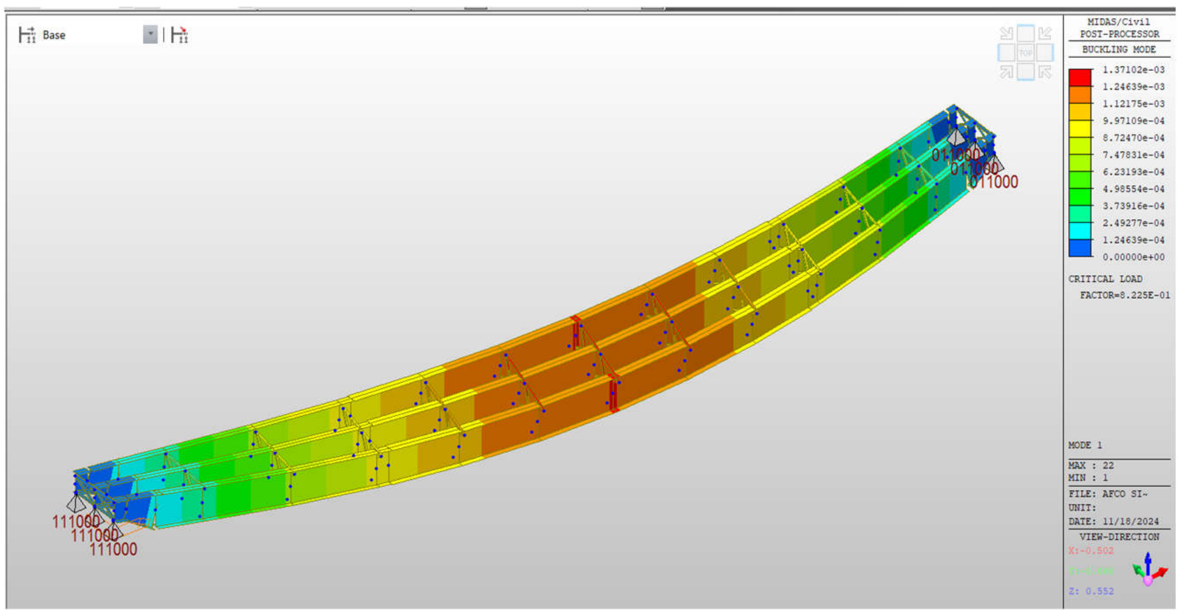
- First stage of construction



Design Considerations

System buckling of narrow steel units (three or fewer girders)

- Deck cast eigenvalue = 0.82 ☹



Design Considerations

System buckling of narrow steel units (three or fewer girders)

$$M_{gs} = C_{bs}\frac{\pi^2 w_g E}{L^2}\sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

$$M_{gs} = 1.1\frac{\pi^2(16\ ft)E}{(283\ ft)^2}\sqrt{2.5x10^5\ in^4\ * 2.0x10^4\ in^4}$$

$$M_{gs} \cong 56{,}000\ ft - kips$$

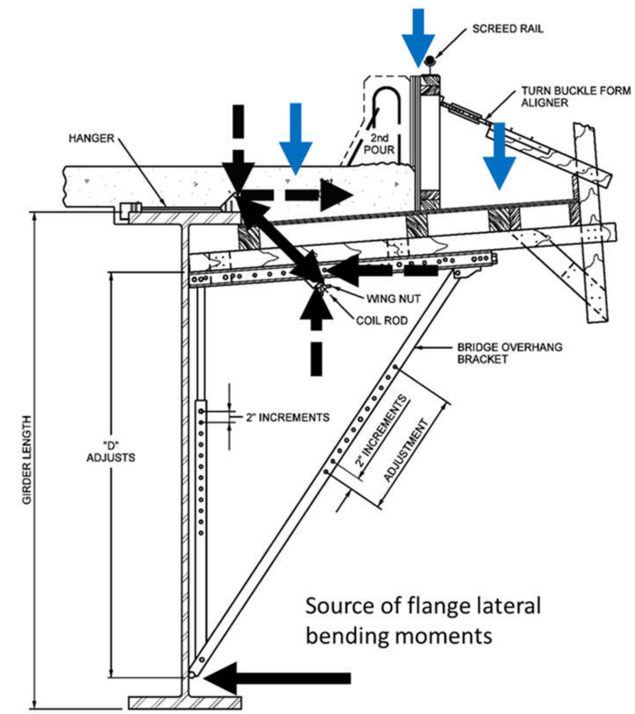
- M_u in EACH girder during deck casting = 27,000 ft-kips * 3
  = 81,000 >>> 0.7*56,000. **<u>“Overstressed” by 2</u>**
- Recall the eigenvalue of 0.82 is in the three-girder system, a desired value is 1.5 or better, off by a factor of 1.83, like the 2 from above.

Design Considerations

Stability bracing requirements

- Bracing forces?
  - Recall from earlier discussion – sometimes we don't have bracing design forces
  - Reactions to bracket loads
  - Wind load on steel
    - During erection
    - Final conditions

We might ask…Anything else?



Design Considerations

Stability bracing requirements

- What does AASHTO LRFD 6.7.4.2 say?
- Depth requirements
  - Rolled beams – 0.5 x member depth
  - Plate girder – 0.75 x member depth
- What about strength or stiffness bracing provisions?
  - Except slenderness, we haven't had any guidance…

…until now. The 10^{th} Edition now does.

Design Considerations

Stability bracing requirements

- 6.7.4.2.1 … Diaphragms or cross-frames for rolled-beam and plate-girder bridges **shall satisfy the stability bracing stiffness and strength requirements specified in Article 6.7.4.2.2**, as applicable.

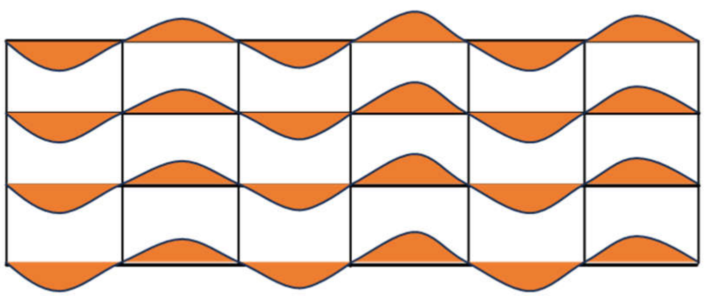
- 6.7.4.2.2 Stability Bracing Requirements (**new article**)

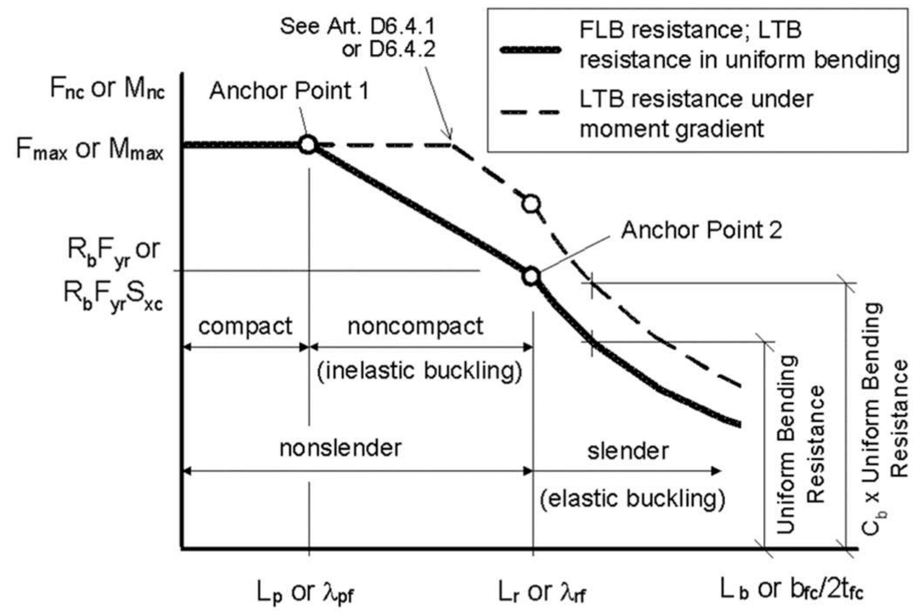
6.7.4.2.2—Stability Bracing Requirements

In addition to the minimum design requirements specified in Article 6.7.4.1, diaphragms or cross-frames for all rolled-beam and plate-girder bridges shall satisfy the following stability bracing stiffness requirement for the applicable noncomposite *DC* loads and any construction loads applied to the fully erected steelwork:

Design Considerations

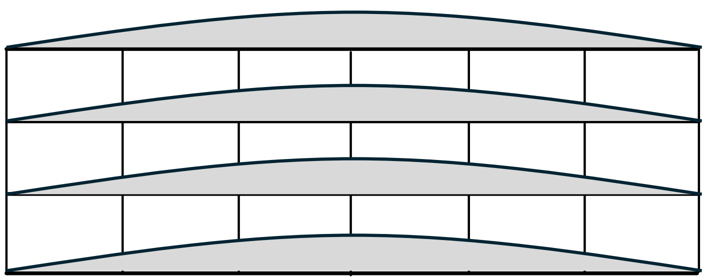
Stability bracing requirements

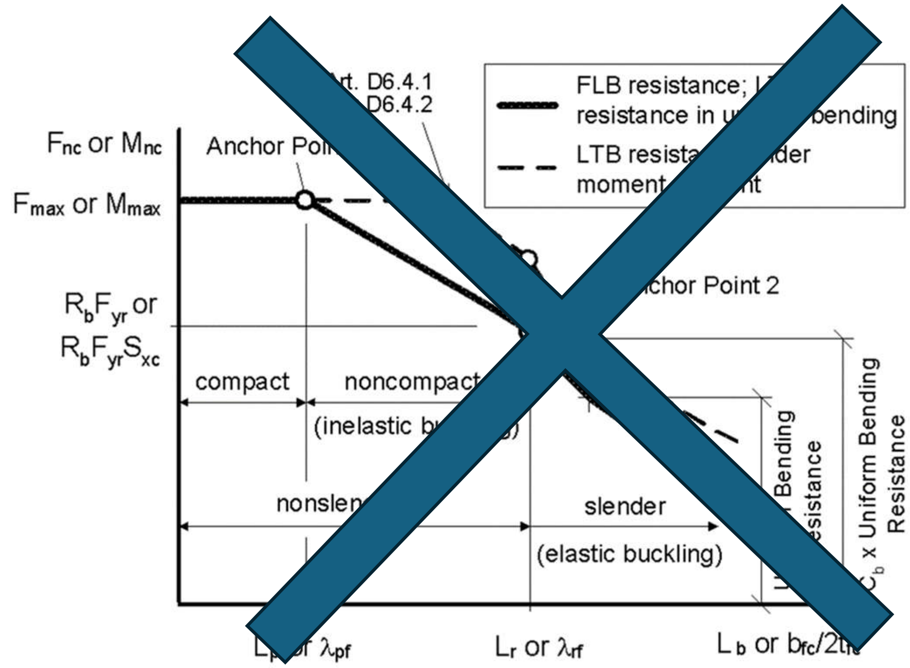




Design Considerations

Stability bracing requirements





Design Considerations

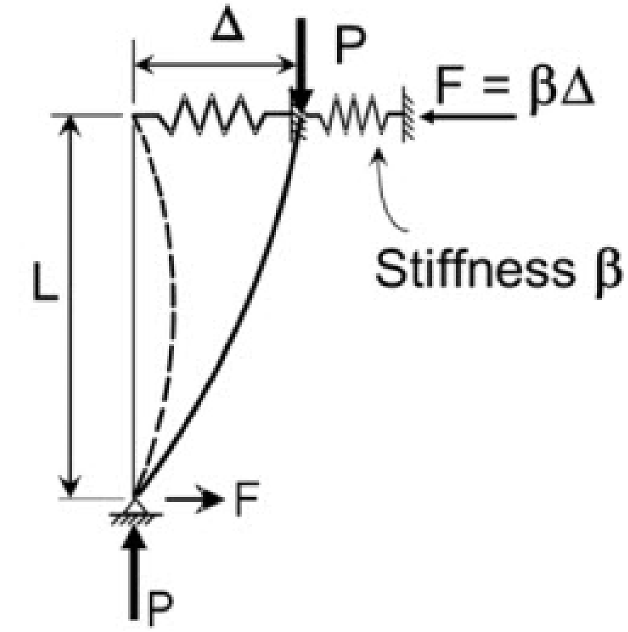
Stability bracing requirements

- What are the requirements for bracing systems?
  - Bracing plays a major role in the stability of the structural system.
  - Effective bracing must satisfy both <u>**strength**</u> and <u>**stiffness**</u> to have a safe system.
  - Provisions outlined in the following slides allow engineers to verify the adequacy of the bracing.

Design Considerations

Stability bracing requirements

- Ideal brace stiffness, β
- Buckling occurs between brace points
- Initial imperfection / out-of-straightness

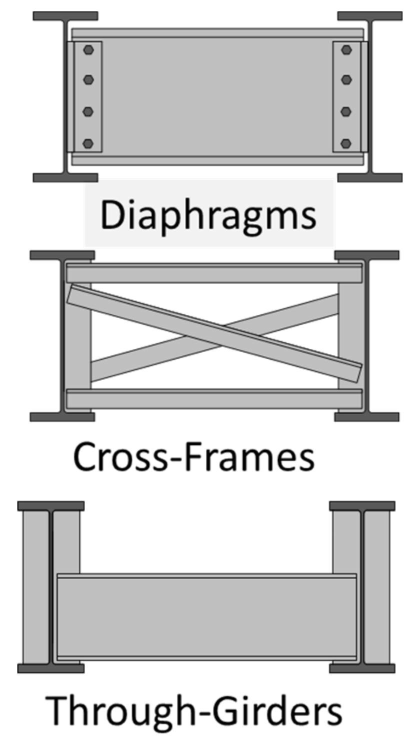


Design Considerations

Stability bracing requirements

- Fundamental concept with torsional bracing:
  - Grider is fully braced at a location if twist is prevented.
  - Stiffness requirement:

$$(\beta_T)_{act} \geq (\beta_T)_{req} \qquad (6.7.4.2.2\text{-}1)$$



Design Considerations

Stability bracing requirements

$(\beta_T)_{req}$= required stiffness of the torsional brace system (kip-in./rad) calculated as follows:

- For diaphragms and cross-frames whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:

$$= \frac{2.4L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b} \right)^2 \qquad (6.7.4.2.2\text{-}2)$$

- Otherwise:

$$= \frac{3.6L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b} \right)^2 \qquad (6.7.4.2.2\text{-}3)$$

Design Considerations

Stability bracing requirements

- Actual bracing stiffness

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_{br}} + \frac{1}{\beta_{\text{sec}}} + \frac{1}{\beta_g}\right)} \qquad (6.7.4.2.2\text{-}6)$$

β_{br} = brace stiffness of the diaphragm or cross-frame that restrains twist of a beam or girder (kip-in./rad.) determined as follows:

β_{sec} = cross-sectional distortion stiffness for stability bracing (kip-in./rad.) determined as follows:

β_g = effective in-plane girder stiffness for stability bracing (kip-in./rad.)

Springs in series

$$k_{total} = \frac{1}{\frac{1}{k1} + \frac{1}{k2} + \frac{1}{k3}}$$

Design Considerations

Stability bracing requirements

- β_{br}, bracing stiffness of cross-frame/diaphragm

For an X-type cross-frame, compression-diagonal system:

$$= \frac{A_d E S^2 h_b^2}{L_d^3} \qquad (6.7.4.2.2\text{-}8)$$

For a K-type cross-frame:

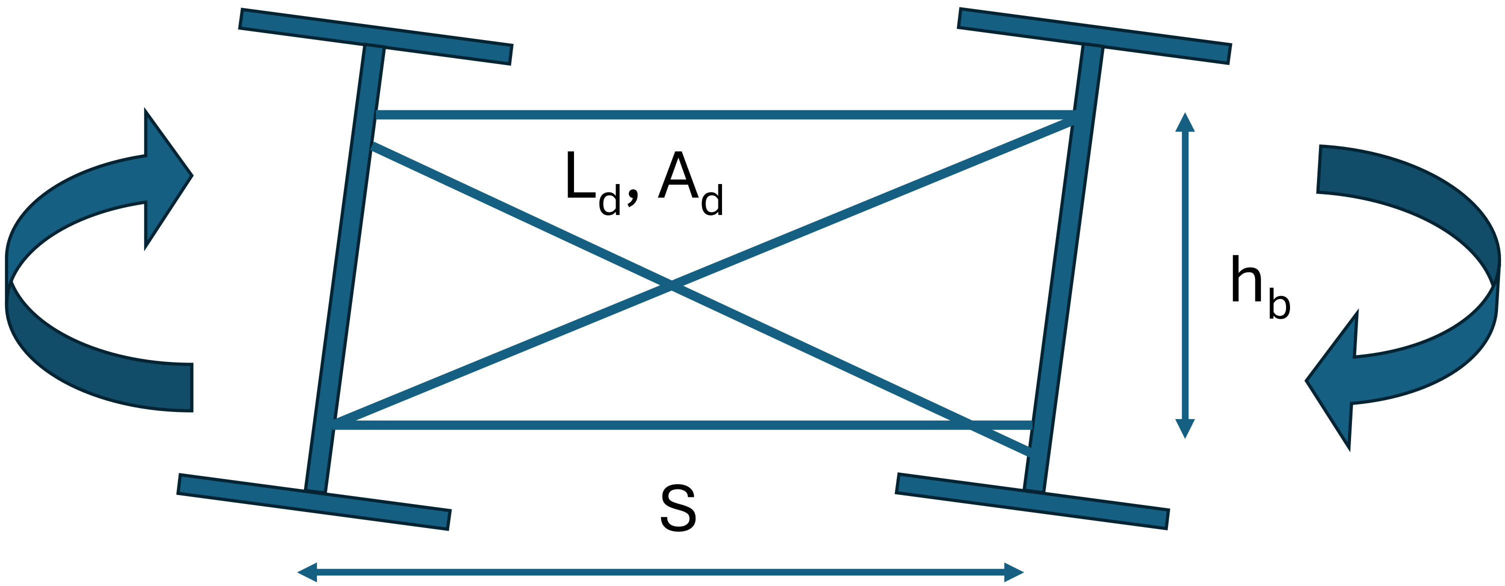
$$= \frac{2ES^2 h_b^2}{\dfrac{8L_d^3}{A_d} + \dfrac{S^3}{A_s}} \qquad (6.7.4.2.2\text{-}9)$$

For a diaphragm attached at or above mid-height of the beam or girder:

$$= \frac{6EI_b}{S} \qquad (6.7.4.2.2\text{-}10)$$

Design Considerations

Stability bracing requirements



$$\beta_{br} = \frac{A_d E S^2 h_b^2}{L_d^3} \qquad (6.7.4.2.2\text{-}8)$$

Design Considerations

Stability bracing requirements

- β_{sec}, cross-sectional distortion stiffness

β_{sec} = cross-sectional distortion stiffness for stability bracing (kip-in./rad.) determined as follows:

- For diaphragms and cross-frames, whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:
  $= \infty$ (infinity)

- Otherwise:

$$\beta_{\sec i} = \frac{3.3E}{h_i}\left(\frac{D}{h_i}\right)^2\left(\frac{1.5h_i t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \qquad (6.7.4.2.2\text{-}12)$$

Design Considerations

Stability bracing requirements

- β_g, in-plane girder stiffness is a function of:
  - Individual girder stiffness at point under consideration, I_x
  - Number of girders in span under consideration, n_g

β_g = effective in-plane girder stiffness for stability bracing (kip-in./rad.)

$$= \frac{24\left(n_g - 1\right)^2 S^2 E I_x}{n_g L^3} \qquad (6.7.4.2.2\text{-}13)$$

Design Considerations

Stability bracing requirements

- Summary of new stiffness requirements

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_{br}} + \frac{1}{\beta_{\sec}} + \frac{1}{\beta_g}\right)} \geq (\beta_T)_{req} = \frac{2.4L}{\phi_{sb} nEI_{yeff}} \left(\frac{M_u}{C_b}\right)^2$$

- Flange proportions (b/t) directly influences I_{yeff}
- Number of braces, n, influences the required stiffness of each brace
- β_{br} is related to girder web height, spacing, and stiffness of bracing elements
- β_{sec} can be commonly ignored
- β_g is related to I_{xx} of the girder

Design Considerations

Stability bracing requirements

- Summary of new stiffness requirements

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_{br}} + \frac{1}{\beta_{\sec}} + \frac{1}{\beta_g}\right)} \geq (\beta_T)_{req} = \frac{2.4L}{\phi_{sb} nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2$$

- What if this isn't satisfied?
  - One option: if flange level lateral bracing is used over 0.2L adjacent to the support, required stiffness shall be multiplied by 0.70.

Design Considerations

Stability bracing requirements

- What about strength?

- For diaphragms and cross-frames, whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:

$$M_{br} = \frac{2.4L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \tag{6.7.4.2.2-14}$$

- Otherwise:

$$M_{br} = \frac{3.6L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) \tag{6.7.4.2.2-15}$$

Design Considerations

Stability bracing requirements

- Strength requirement

$$M_{br} = \boxed{\frac{2.4L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2}\boxed{\left(\frac{L_b}{500h_o}\right)}$$

- Resolved into member forces.
- Required stability bracing forces in each member are combined with other force effects acting during construction.

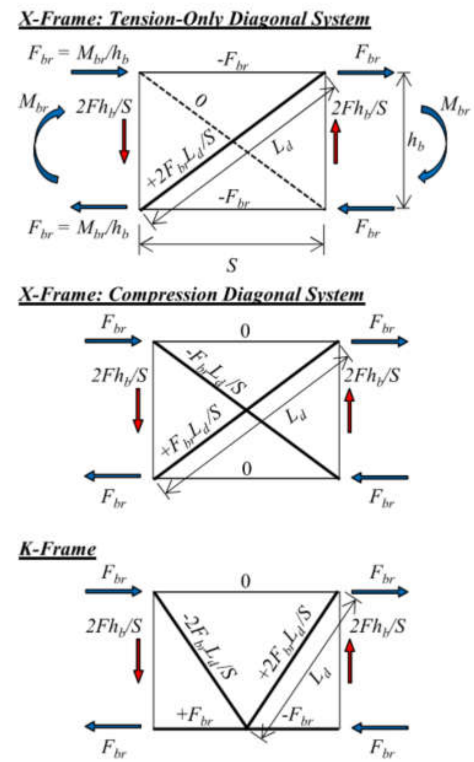


Figure C6.7.4.2.2-1—Distribution of the Required Bracing Moment, M_{br}, to Cross-Frame Members in Various Configurations

Design Considerations

Stability bracing requirements

- **DO NOT** finalize the design of your girder until you start checking these interactions
- The cross-frame itself is unlikely to be “the problem”
- If these checks fail it is much more likely associated with your girder properties

Design Considerations

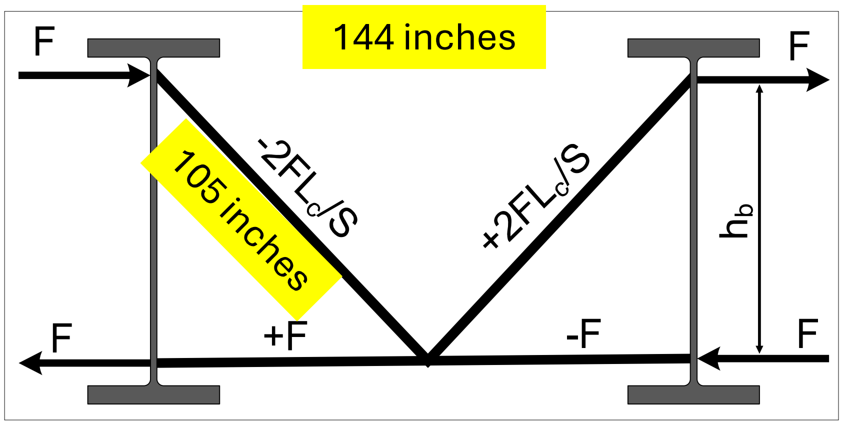
Stability bracing example

- Single span – 200 ft
- 4 beams @ 12 ft on center
- Web depth 86 inches (about L/28 for the web alone)
- Frame depth = 76 in. (web depth minus 10 in.) (88% of web depth)
- Factored construction moment at midspan = 14,448 ft-kips
- S/D = 144 / 86 = 1.67 > 1.5 a K-frame is recommended.
- Minimum size angle to meet kl/r = 6x6x3/8 for the top chord.
- Minimum size angle to meet kl/r = 5x5x5/8 for the diagonals.
- Cross-frames at 25 ft on center
- C_b = 1 for simplicity

Design Considerations

Stability bracing example

$$\beta_b = \frac{2ES^2h_b^2}{\frac{8L_d^3}{A_d} + \frac{S^3}{A_s}}$$



E = Modulus of elasticity (ksi)
L_c = Length of diagonal (in), **105 in.**
A_d = Area of diagonal member(s) (in), **5.9 sq in. times 0.65 = 3.84**
A_s = Area of horizontal member(s) (in), **4.38 sq in. times 0.65 = 2.85**
h_b = Height of brace system (in), **76 in.**
S = Spacing of girders (in), **144 in.**

Design Considerations

Stability bracing example

$$\beta_b = \frac{2ES^2h_b^2}{\frac{8L_d^3}{A_d} + \frac{S^3}{A_s}} = \frac{2(29000)(144^2)(76^2)}{\frac{8(105^3)}{3.84} + \frac{144^3}{2.85}} = 2.0x10^6$$

Design Considerations

Stability bracing example

$$\beta_g = \frac{24\left(n_g - 1\right)^2 S^2 E I_x}{n_g L^3}$$

$$= \frac{24(4-1)^2(144^2)(29000)(222{,}839)}{(4)(2400^3)} = 5.2\text{x}10^5$$

Design Considerations

Stability bracing example

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}\right)}$$

$$(\beta_T)_{act} = \frac{1}{\left(\frac{1}{2x10^6} + \frac{1}{\infty} + \frac{1}{5.2x10^5}\right)} = 4.1\text{x}10^5$$

Design Considerations

Stability bracing example

$$(\beta_T)_{act} \geq \frac{2.4L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b}\right)^2$$

$$4.1x10^5 \ \geq \ \beta_T = \frac{2.4(2400)(14{,}448 * 12)^2}{0.8(7)(1)(29000)(2693)} = 3.96x10^5$$

Design is satisfactory even with minimum kl/r angles

If it didn't work, solutions include:

- Increase the size of the angles
- Add a line of crossframes
- Switch to an X Frame
- Increase I_{xx} to increase the in-plane girder stiffness

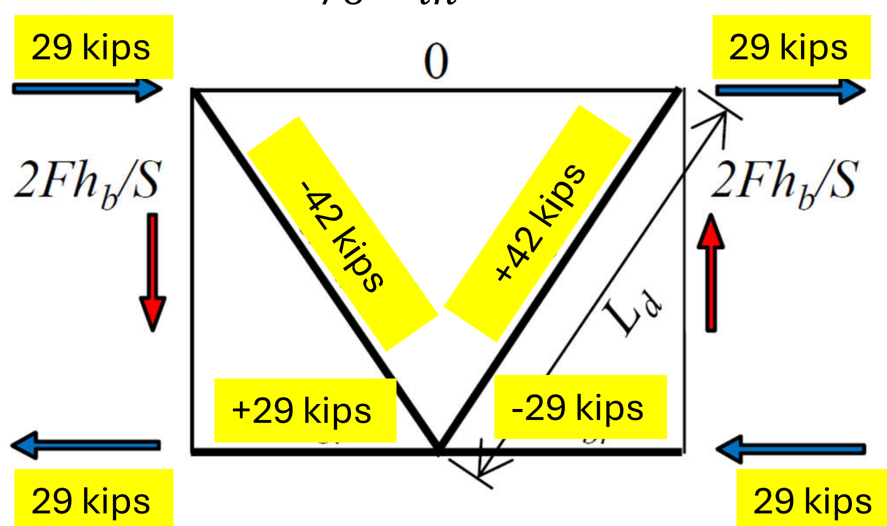
Design Considerations

Stability bracing example

$$M_{br} = M_{br} = \beta_T \theta_o = M_{br} = \frac{2.4L}{nEI_{yeff}}\left(\frac{M_u}{C_b}\right)^2\left(\frac{L_b}{500h_o}\right) = 2210\ in * kips$$

$$F_{br} = \frac{2210\ in - kips}{76 - in} = 29\ kips$$

Note: These are JUST the forces from stability bracing. To these, add in the chord forces from other construction loads



Design Considerations

Stability bracing summary

- AASHTO now has REQUIREMENTS (in the 10th edition) requiring that flexural members be braced with members of sufficient stiffness and strength
- Stiffness is required to control distortion (twist) in girders.
- Restraint of twist requires a strength design check of the bracing system
- **Calculations <u>on approximately 200 bridges</u> show that typical crossframes, designed for kl/r requirements meet or come close to meeting the stiffness and strength requirements for a skew up to 20 degrees.**

Design Considerations

Stability bracing summary

- But what if it doesn't work?
  - If it doesn't work, and it's close…
    - Wider / thinner flange if possible to increase I_{yeff}
    - Deepen the girder to increase I_{xx}
    - Add a line or two of bracing, to increase "n" in the stability equations
  - If it's "way off"
    - Add top flange level lateral bracing for one or two bays at the end of the span
    - And then remember to check the wind loads that will now accumulate at the braced end

Design Considerations

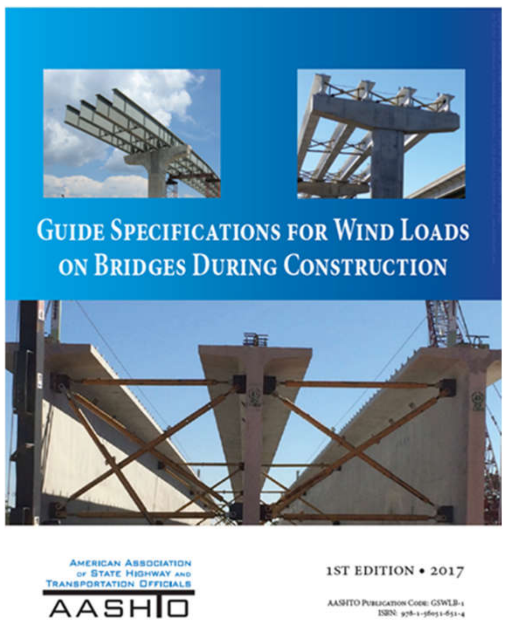
Stability bracing summary

- What have we learned?
  - Cross-frames using minimum kl/r bracing angles provide a reasonable solution in terms of stiffness and strength.
  - When a design does not satisfy the stiffness requirements it is usually because of the girder stiffness components
  - Considering the wind and stability topics together the following are important conclusions
    - Narrow thick flanges are not an efficient solution. Better performance all around is found with wider and thinner flanges

Design Considerations

Wind load during construction

Strength Loads

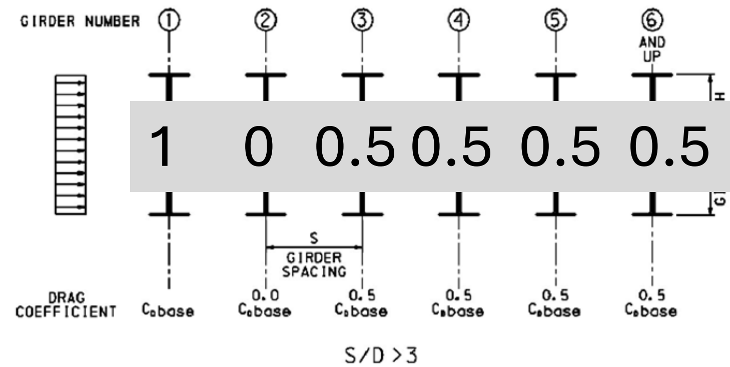


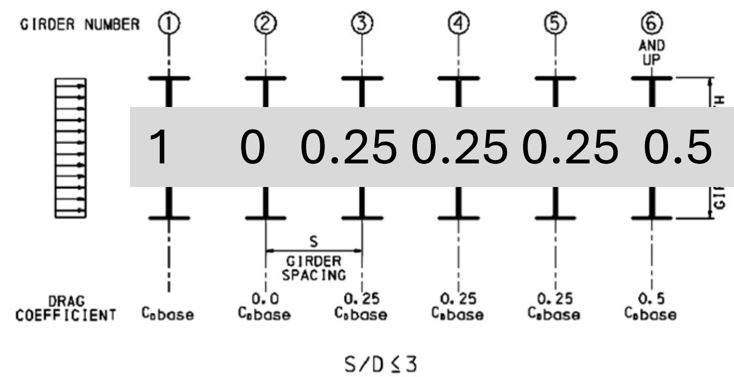
Service Loads

Design Considerations

Wind load during construction

- Guide specifications
  - Assumptions (things you need to know)
  - Inactive wind speed (115 mph)
  - R factor (reduction in wind risk due to period of exposure)
    - 0.73 used, 6 wks – 1 yr construction
  - Height corrections
    - 1.0 if less than 33 ft above ground
  - Drag coefficients
    - $C_{D,base}$ = 2.2 (plate girders during construction)
    - Sum of C_D for girders at that stage





Design Considerations

Wind load during construction

- Remember, stress might not be maximum at midspan
- Need to check combined stresses with steel self-wt

Design Considerations

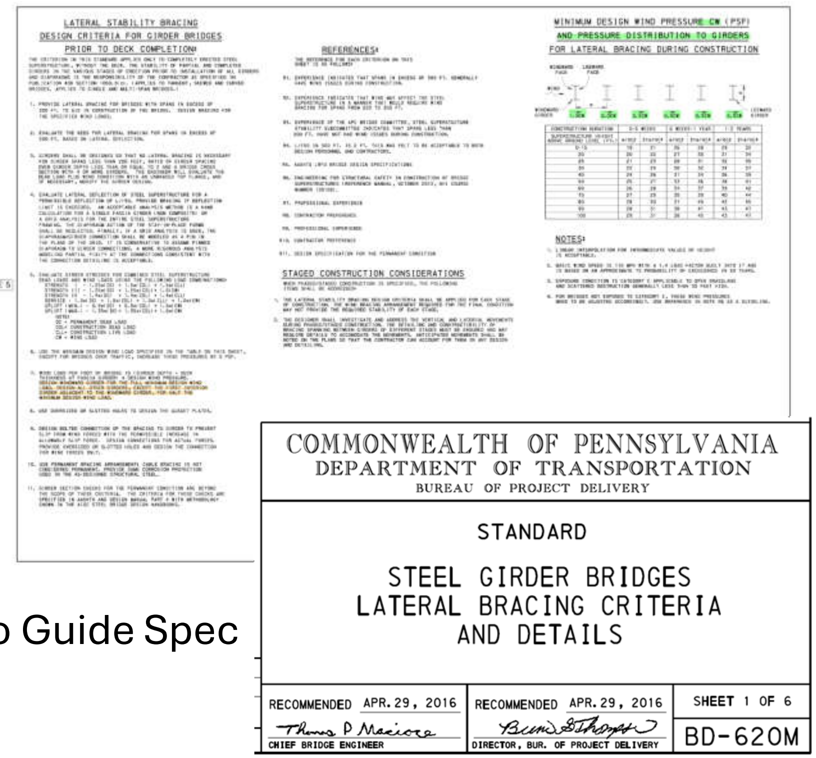
Comments and observations from standards development

- Unless your state requires this check, these are "optional" in the sense that a Guide Specifications is not a mandatory requirement.
- But what can we learn?
  - The development of standards assumed bridges "low to the ground", i.e. < 33 ft. Bridges much higher than this will change the outcomes
  - For single span bridges, Guide Spec wind load stresses are between 5 – 20 ksi
    - Recall a limit of 0.6 Fy, 30 ksi commonly for Gr 50W plate
    - For single span bridges, deflection became a problem before stress
  - For 2, 3, 4-span bridges, Guide Spec wind load stresses approached 0.6Fy for various designs
    - For continuous span bridges, deflection was almost never a problem

Design Considerations

Wind load during construction

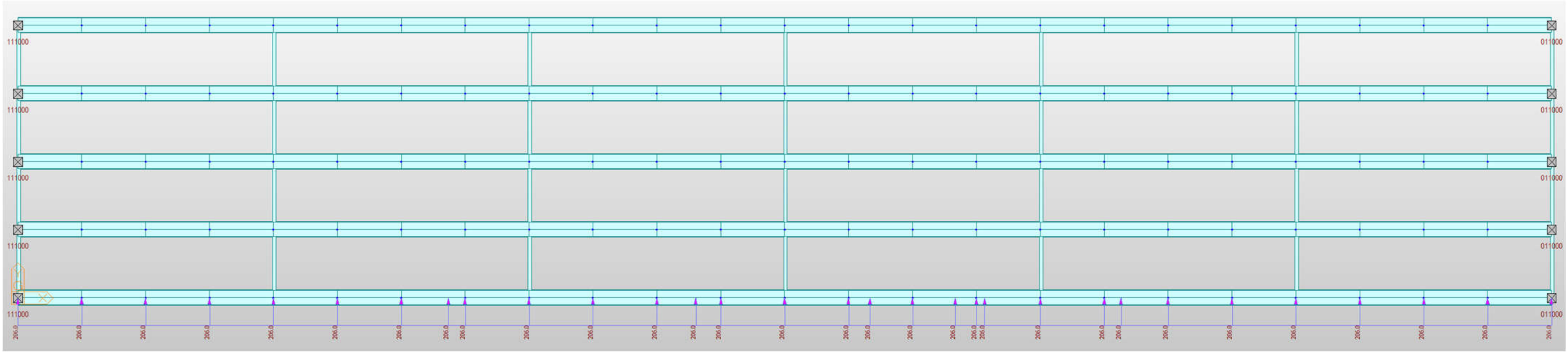
- PennDOT criteria
  - BD-620M
  - Design spans under 200 ft to not need it
  - Evaluate between 200-300 ft
  - Always provide over 300 ft
  - Permissible lateral deflection L/150
  - Changes
    - Old - only fascia girders are loaded
    - New (Nov. 2022) – all girders loaded similar to Guide Spec



Design Considerations

Wind load during construction

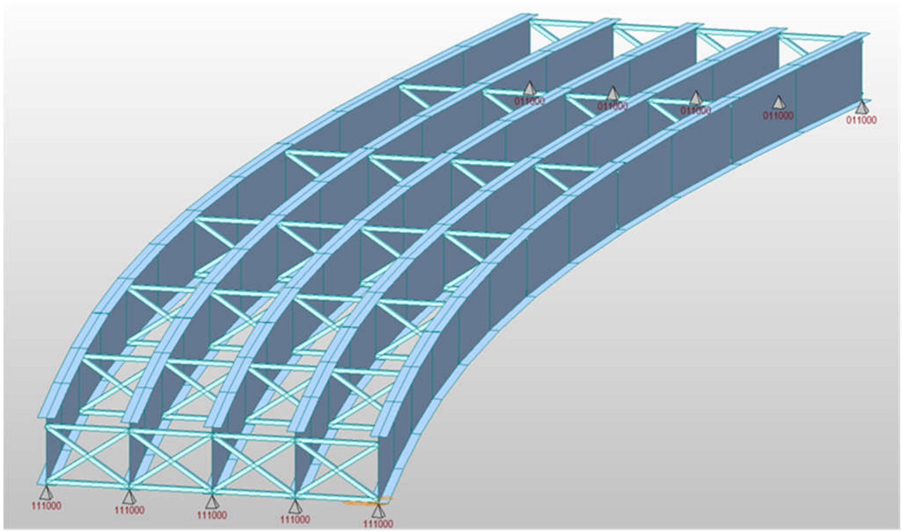
- Example bridge
    - 5 beams @ 8 ft centers
    - Span = 180 ft
    - Apply the PennDOT 32 psf wind load to fascia beam only



Design Considerations

Wind load during construction

- Example bridge
  - Midspan deflection = 14.9”
  - Permissible = L/150 = 14.4”
  - Slightly over
    - Solutions could be
      - Revise flanges
      - Add bracing

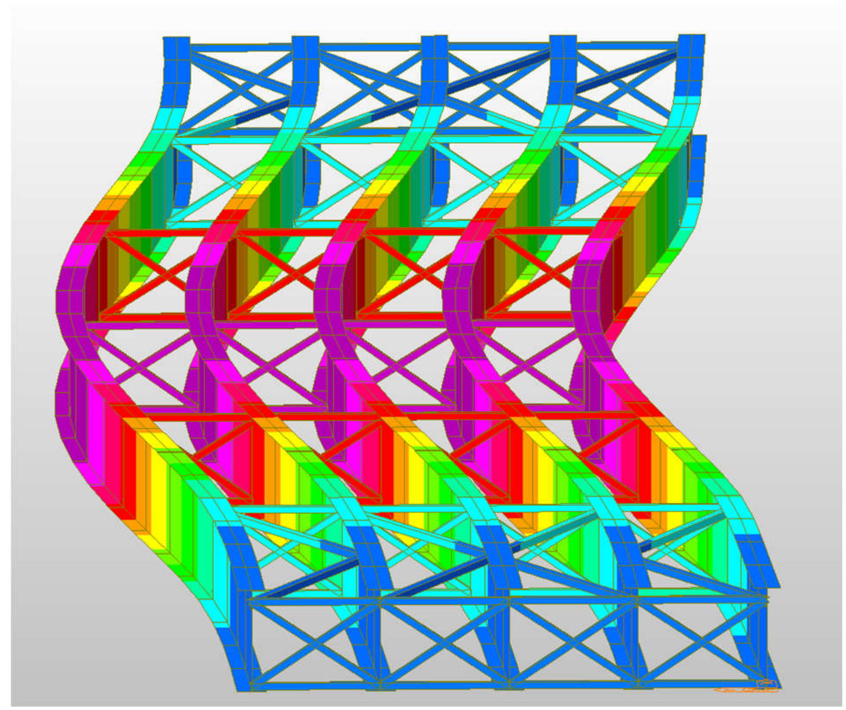


Examine what happens if we add WT bracing at the end

Design Considerations

Wind load during construction

- Example bridge
  - Midspan deflection = 1.3”
    - Previous 14.9”
  - What happened?
    - Bridge has 6 bays @ 30 ft each
    - 1 bay each end now has lateral bracing
    - “Free length” now 120 ft



- **A little bracing can go a long way…**

Design Considerations

Wind load during construction

- Some other recommendations
  - Do not rely on outer bay bracing only
  - Add bracing as the width of the bridge is built out
  - As a designer you have no idea what sequence will be used in construction
  - Provide the greatest degree of safety and stiffness by including bracing in all bays
  - Each bay added adds wind load area but also capacity
  - Look at this early on…wind and constructability can control.

6. Design Resources

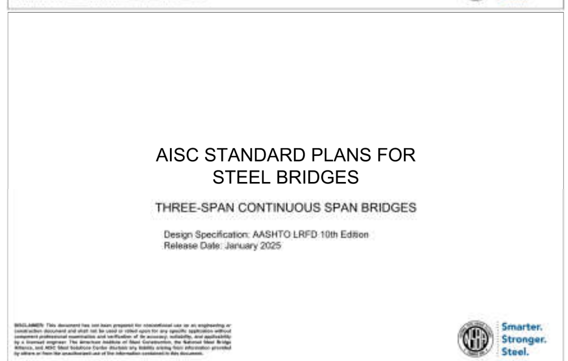
Design Resources

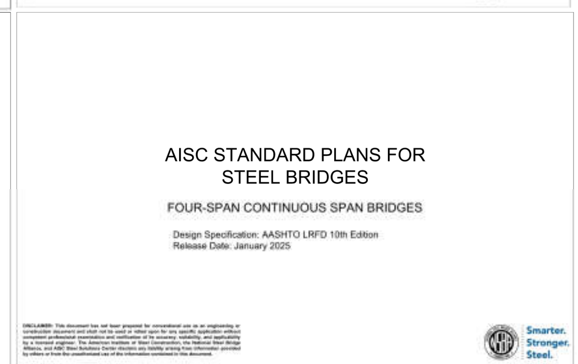
Standard Plans for Steel Bridges

- AASHTO 10th Edition
- 1-4 span bridges
- 80-300 ft span lengths
- 8, 10, 12, 14 ft spacing
- Comprehensive


AISC STANDARD PLANS FOR
STEEL BRIDGES
SINGLE SPAN BRIDGES AND MULTI-SPAN BRIDGES
WITH LINK SLABS
Design Specification: AASHTO LRFD 10th Edition
Release Date: January 2025


AISC STANDARD PLANS FOR
STEEL BRIDGES
TWO-SPAN CONTINUOUS SPAN BRIDGES
Design Specification: AASHTO LRFD 10th Edition
Release Date: January 2025


AISC STANDARD PLANS FOR
STEEL BRIDGES
THREE-SPAN CONTINUOUS SPAN BRIDGES
Design Specification: AASHTO LRFD 10th Edition
Release Date: January 2025


AISC STANDARD PLANS FOR
STEEL BRIDGES
FOUR-SPAN CONTINUOUS SPAN BRIDGES
Design Specification: AASHTO LRFD 10th Edition
Release Date: January 2025

https://www.aisc.org/nsba/design-and-estimation-resources/standard-bridge-plans/

Design Resources

Standard Plans for Steel Bridges

AISC STANDARD PLANS FOR
STEEL BRIDGES

THREE-SPAN CONTINUOUS SPAN BRIDGES

Design Specification: AASHTO LRFD 10th Edition
Release Date: January 2025

Smarter.
Stronger.
Steel.

Design Resources

Standard Plans for Steel Bridges

GENERAL NOTES:

Specifications:

AASHTO LRFD Bridge Design Specifications, 10th Edition.

AASHTO Guide Specifications for Wind Loads on Bridges During Construction, 1st Edition.

Materials:

Girder Webs and Flanges

ASTM A709 Gr 50W or Gr HPS 70W as noted in the plate size tables

Gr HPS 70W flanges are noted with a ▲

Stiffeners

A709 Gr 50W

Intermediate transverse shear stiffeners, single sided
Stiffener sizes shown as required by design, $\frac{1}{2}$ in. minimum thickness

Lateral Bracing and Diaphragm / Crossframe Members

ASTM A709 Gr 50W

Concrete Deck

f'_c = 4 ksi

Reinforcing Steel

F_y = 60 ksi

Bolts

ASTM F3125 Grade A325, diameter provided on detail sheets

Loading:

Live Load

Live load is the controlling force effects from:

HL93
EV3 - Present in multiple lanes
Fatigue design based on $ADTT_{sl}$ = 1000 trucks per day

Dead Load

Dead load assumptions:

For DC1

Slab thickness as shown in plans
Overhang thickness = slab thickness + 4 in.
Concrete haunch weight, 50 plf per beam
Stay-in-place form allowance, 15 psf
Miscellaneous steel weight:
8 ft girder spacing - 30 plf
10 ft girder spacing - 30 plf
12 ft girder spacing - 30 plf
14 ft girder spacing - 45 plf

Total DC1 loads shown on this sheet are computed with the above assumptions and assuming equal loading to all beams in the cross-section.

For DC2

Assumed single slope TL5 railing
600 plf divided to two beams

For DW

2 in. asphalt at 140 pcf

Final Design Dead Loads

8 ft girder spacing designs:

DC1 = 930 plf
DC2 = 300 plf
DW = 160 plf

10 ft girder spacing designs:

DC1 = 1,220 plf
DC2 = 300 plf
DW = 200 plf

12 ft girder spacing designs:

DC1 = 1,540 plf
DC2 = 300 plf
DW = 240 plf

14 ft girder spacing designs:

DC1 = 2,000 plf
DC2 = 300 plf
DW = 280 plf

Note: exterior girders also designed for flange lateral bending moments from overhang brackets and concrete deck finishing machine. Flange lateral bending moments for exterior beams are provided on the **Fascia Beam Design Criteria** sheet.

Wind Load

Wind on completed bridge 44 psf
Wind on open framing during construction, see **General Design Criteria** sheet.

GENERAL NOTES

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Design Resources

Standard Plans for Steel Bridges

Design Assumptions and Criteria, Continuous Span Bridges:

1. Girder Design
   a. All designs performed using NSBA LRFD SIMON.
   b. Interior and exterior beams were designed. In LRFD SIMON, the "BOTH" option is used for the LL distribution factors. This results in a single beam designed for the governing shear and moment distribution factors for an interior and exterior beam. The composite slab effective width is based on an exterior beam.
   c. Live load distribution follows AASHTO LRFD 4.6.2.2 for all beam spacings and span lengths. Designs where the AASHTO distribution factor equations are used beyond the range of applicability are noted in the design tables.
   d. A skew of 20 degrees from normal is assumed for all designs.
   e. Live load deflection satisfies AASHTO LRFD 2.5.2.6.2 Criteria for Deflection for vehicular bridges, L/800.
   f. Girder depth satisfies AASHTO LRFD 2.5.2.6.3 Optional Criteria for Span-to-Depth Ratios.
   g. Fatigue design based on Category C for shear studs welded to top flanges and Category C' for welded transverse stiffeners, $ADTT_{SL}$ = 1,000 vehicles per day and a 75-year design life.
   h. Maximum segment length, 140 feet.
   i. Three-span-continuous units are designed for end span lengths equal to 78% of the center span length.
   j. All continuous span bridges have field splices adjacent to each pier at approximately 0.7L of the end span.
   k. Some continuous span bridges have additional splices at approximately 0.25L of the end span to meet shipping length requirements. These are noted in the plans.
   l. Maximum shipping weight, 50 tons.
   m. Maximum web depth, 11 feet.
   n. Minimum top flange width, $b_{tfs} \geq L_{fs} / 85$ where L_{fs} is the field section length. AASHTO LRFD (C6.10.2.2-1).
   o. Flange widths held constant in a field section.
   p. Minimum flange thickness, 1 in. Maximum flange thickness, 3 in. Flange thickness increments, 1/4 in.
   q. Minimum web thickness, 1/2 in. Web thickness increments, 1/8 in.
   r. No more than two complete joint penetration flange butt welds per flange in any field section.
   s. When a single size flange is used in a field section, the weight reduction of a complete joint penetration transition was first evaluated and then eliminated based on weight, cost, and stress considerations.
   t. Single-sided transverse shear stiffeners are used when needed.
   u. Longitudinal stiffeners are not used.
   v. All girders are composite for positive and negative bending.
   w. Negative moment longitudinal deck reinforcing is 1% of the gross deck cross-section. This reinforcing extends at least between the field splices, or longer as required by AASHTO LRFD 6.10.1.7 for the Service II Limit State. Designer to determine if the factored deck casting and construction loads require this reinforcing steel to be extended. See the **Deck Details** sheets for additional details.
   x. Shear stud design based on LRFD SIMON and AASHTO LRFD 9th edition. For flanges ≤ 16 in. wide, three 7/8 in. diameter studs in a transverse row are used. All other flange widths use four studs in a transverse row.
2. Diaphragm and Cross-Frame Design
   a. Intermediate diaphragms and cross-frames are designed as below. End diaphragms or cross-frames that support the deck and/or expansion joint are not considered as part of these standards.
   b. Diaphragm and cross-frame spacing varies within in the span. Maximum spacing does not exceed 30 ft.
   c. Depth of bracing is at least 0.8 times girder web height.
   d. For cross-frame design, the effective depth of the chords was assumed to be 5 in. vertically from the top and bottom of web. This dimension is used for "D" in the S/D checks. For all S/D checks, "S" is S / Cosine 20 deg assuming a maximum 20 degree skew for all designs.
   e. Solid diaphragms are used when the girder spacing to web depth ratio, S/D > 3.5.
   f. K-frames are used when 1.5 < S/D ≤ 3.5.
   g. X-frames are used when S/D ≤ 1.5
   h. Angles are used for all cross-frame members.
   i. Cross-Frame members are designed as secondary members.
   j. Cross-Frame members are designed for tension / compression loading.
   k. Cross-frame member stiffness is based on 0.65AE stiffness reduction factor for eccentrically loaded angles, AASHTO LRFD C4.6.3.3.4.
   l. Diaphragms and cross-frames are designed for combined stability-induced loads along with simultaneous deck casting forces. The finishing machine is assumed to be centered at a brace point location.
3. Wind Load Design
   a. Lateral deflection and flange lateral bending stresses due to wind on the fully erected steel framing were evaluated. Lateral bracing is not required for the design conditions assumed in 3.1 and 3.2, below. Other conditions may require bracing for wind load deflection or stress.

3.1 Service Design Criteria
   a. Lateral deflections due to wind loads on the fully erected steel satisfy the Span / 150 requirement established by PennDOT BD-620M. All references to BD-620M are to the April 29, 2016 edition.
   b. For this deflection check, a 32 psf assumed pressure is applied to fascia beams only for a superstructure height = 30 ft. For other superstructure heights, refer to PennDOT BD-620M.

3.2 Strength Design Criteria
Girder flange lateral bending is checked for strength as follows:
   a. Maximum wind load positve and negative moment regions were checked. Check other plate transitions in final design.
   b. Fascia beam checked for global bending of the span and local bending between cross-frames.
   c. Wind loads on erected steel determined from the *AASHTO Guide Specification for Wind Loads on Bridges During Construction, 2017.*
      Inactive wind condition, V = 115 mph. Superstructure height, 30 ft
      Superstructure construction duration 6 weeks - 1 year, R = 0.73
      K_z = 1.0, C_d = 2.2 for fascia beam, per AASHTO Guide Specifications for other beams.

4. Bolted Field Splices
   a. All bolted field splices use 1 in. diameter ASTM F3125 Grade A325 bolts and standard sized holes.
   b. All connection and fill plates are Gr 50W.
   c. Slip resistance is based on a Class B surface condition.
   d. For connections where the bottom flange and a portion of the web are required to be in tension to resist the factored moments at the point of splice an additional check was made to determine if the slab has adequate compression strength. This check is not in AASHTO. If the slab is unable to provide a compression capacity equal to the tensile forces of the bottom flange and web in tension, the connection was designed as a noncomposite splice. If or when this situation occurs, these splices are noted "Non-Composite" in the **Bolted Field Splices** sheets. This condition was not encountered in any of the three-span standard designs.

NATIONAL STEEL BRIDGE ALLIANCE NSBA FOUNDED 1995	GENERAL DESIGN CRITERIA	
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Design Resources

Standard Plans for Steel Bridges

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3.2 Strength Design Criteria

noncomposite splice. If or when this situation occurs, these splices are noted "Non-Composite" in the **Bolted Field Splices** sheets. This condition was not encountered in any of the three-span standard designs.

GENERAL DESIGN CRITERIA

Issued January 2025
Revision 0

Sheet 3 of 32

3.1 Service Design Criteria

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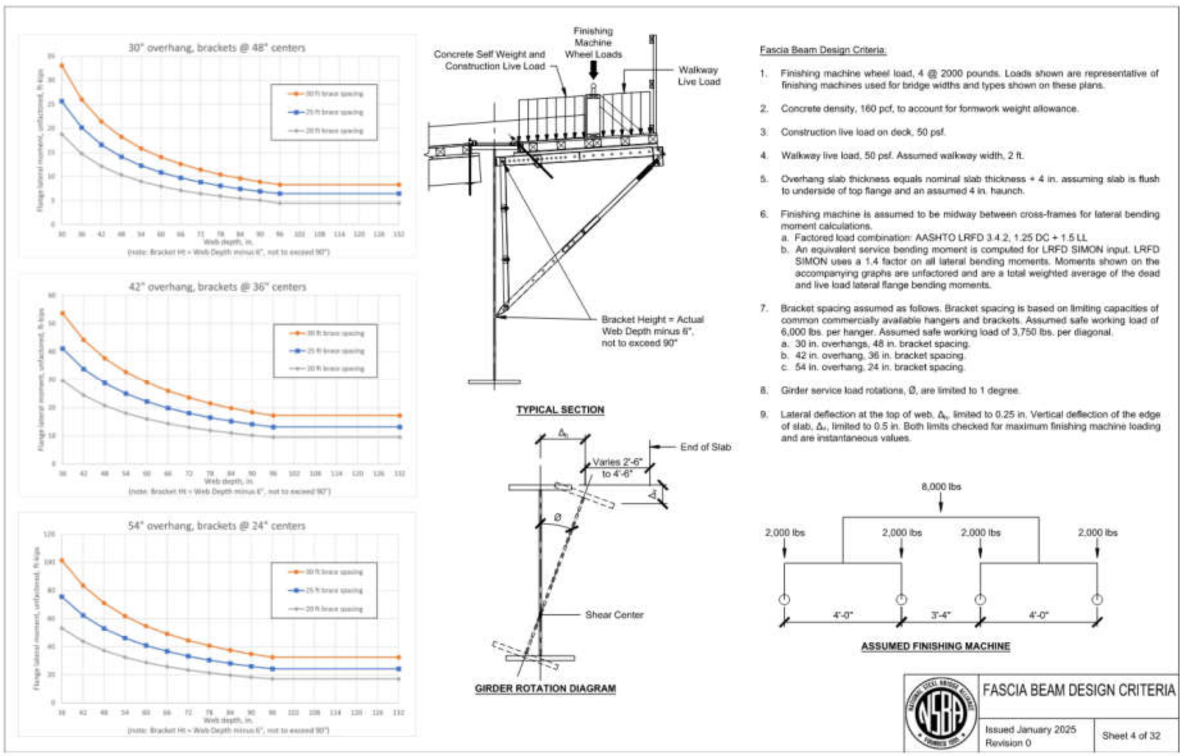
b. For this deflection check, a 32 psf assumed pressure is applied to fascia beams only for a superstructure height = 30 ft. For other superstructure heights, refer to PennDOT BD-620M.

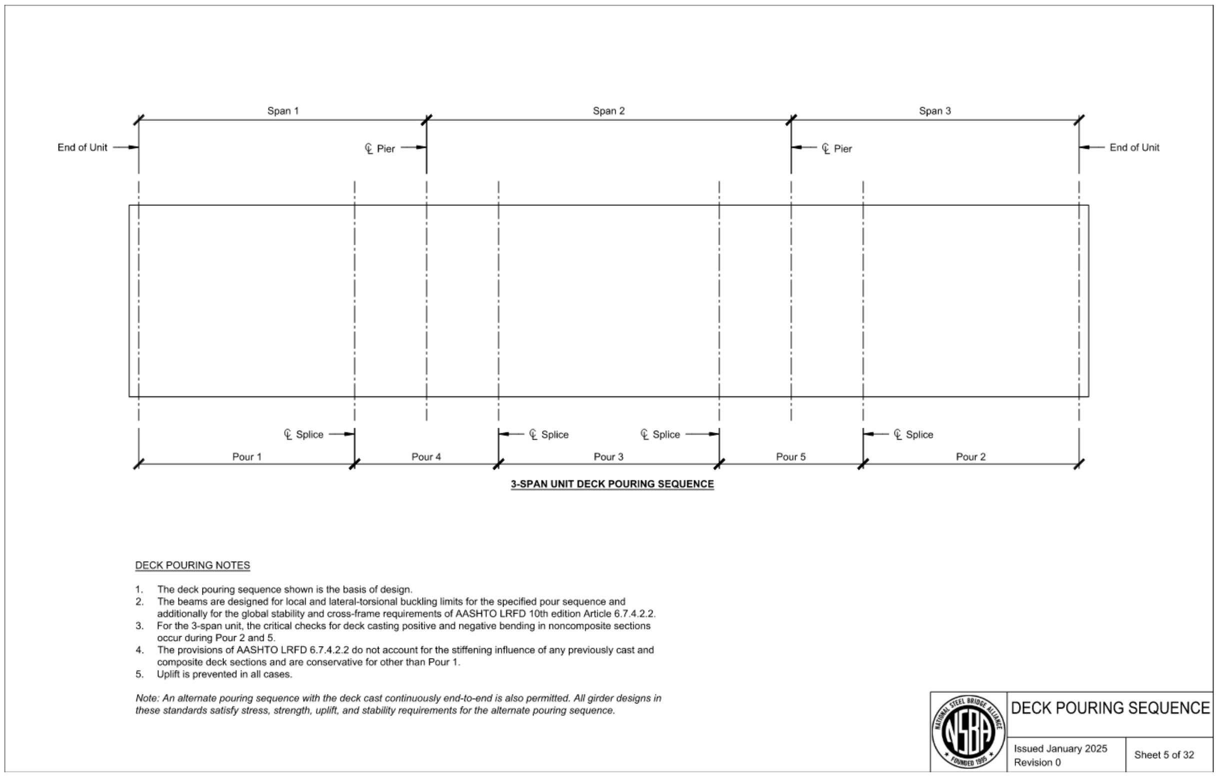
Design Resources

Standard Plans for Steel Bridges



Design Resources

Standard Plans for Steel Bridges



DECK POURING NOTES

1. The deck pouring sequence shown is the basis of design.
2. The beams are designed for local and lateral-torsional buckling limits for the specified pour sequence and additionally for the global stability and cross-frame requirements of AASHTO LRFD 10th edition Article 6.7.4.2.2.
3. For the 3-span unit, the critical checks for deck casting positive and negative bending in noncomposite sections occur during Pour 2 and 5.
4. The provisions of AASHTO LRFD 6.7.4.2.2 do not account for the stiffening influence of any previously cast and composite deck sections and are conservative for other than Pour 1.
5. Uplift is prevented in all cases.

Note: An alternate pouring sequence with the deck cast continuously end-to-end is also permitted. All girder designs in these standards satisfy stress, strength, uplift, and stability requirements for the alternate pouring sequence.

DECK POURING SEQUENCE

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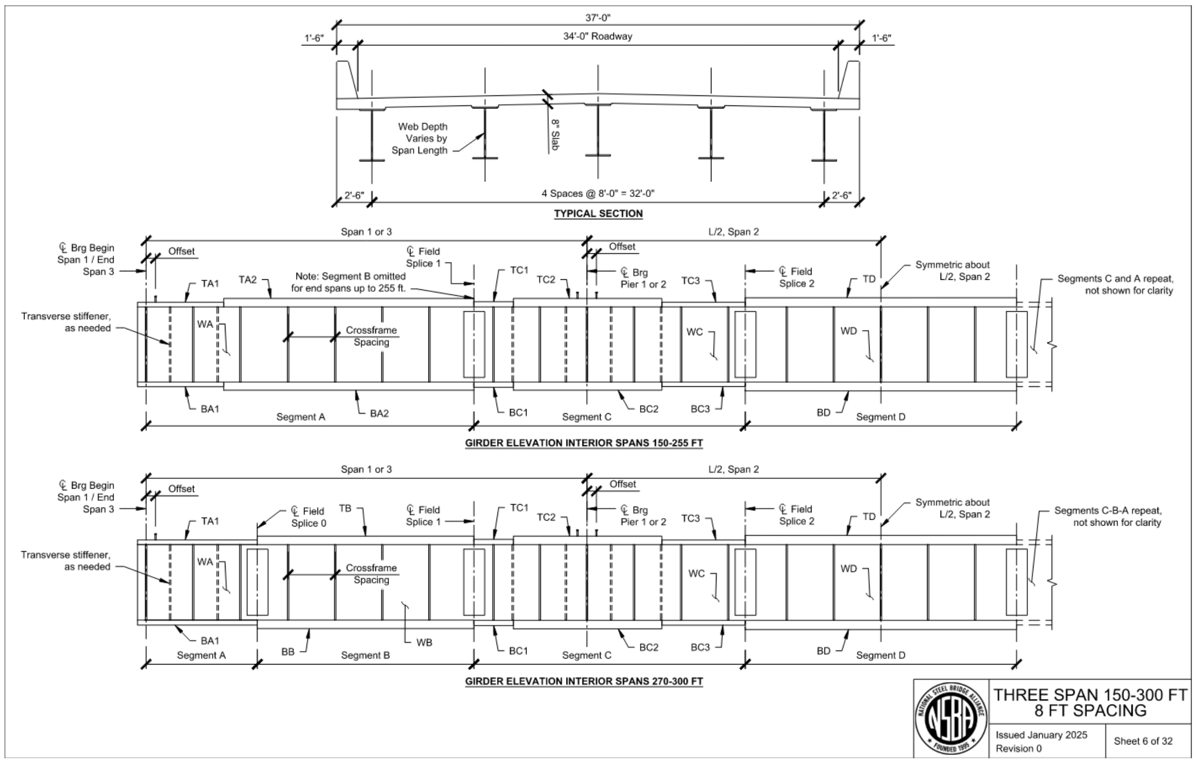
Design Resources

Standard Plans for Steel Bridges



Design Resources

Standard Plans for Steel Bridges

Span, ft. End-Int.-End	SEGMENT A					SEGMENT B (as needed)		
	WA (in. x in. x ft.)	TA1 (in. x in. x ft.)	TA2 (in. x in. x ft.)	BA1 (in. x in. x ft.)	BA2 (in. x in. x ft.)	WB (in. x in. x ft.)	TB (in. x in. x ft.)	BB (in. x in. x ft.)
117-150-117	54 x 0.5 x 79	---	16 x 1 x 79	---	16 x 1.25 x 79	---	---	---
129-165-129	60 x 0.5 x 89	---	16 x 1 x 89	---	16 x 1.25 x 89	---	---	---
141-180-141	66 x 0.5 x 98	---	16 x 1 x 98	---	16 x 1.25 x 98	---	---	---
153-195-153	72 x 0.625 x 106	---	18 x 1 x 106	---	18 x 1 x 106	---	---	---
164-210-164	76 x 0.625 x 113	---	18 x 1 x 113	---	18 x 1 x 113	---	---	---
176-225-176	82 x 0.625 x 122	---	18 x 1 x 122	---	18 x 1 x 122	---	---	---
188-240-188	88 x 0.625 x 130	---	20 x 1 x 130	---	20 x 1 x 130	---	---	---
199-255-199	92 x 0.625 x 138	---	20 x 1 x 138	---	20 x 1 x 138	---	---	---
211-270-211	96 x 0.75 x 51	20 x 1 x 51	---	20 x 1 x 51	---	96 x 0.75 x 100	20 x 1 x 100	20 x 1 x 100
223-285-223	102 x 0.75 x 51	22 x 1 x 51	---	22 x 1 x 51	---	102 x 0.75 x 110	22 x 1 x 110	22 x 1 x 110
234-300-234	108 x 0.75 x 54	22 x 1 x 54	---	22 x 1 x 54	---	108 x 0.75 x 120	24 x 1 x 120	24 x 1 x 120

Span, ft. End-Int.-End	SEGMENT C							SEGMENT D			Additional Footnotes
	WC (in. x in. x ft.)	TC1 (in. x in. x ft.)	TC2 (in. x in. x ft.)	TC3 (in. x in. x ft.)	BC1 (in. x in. x ft.)	BC2 (in. x in. x ft.)	BC3 (in. x in. x ft.)	WD (in. x in. x ft.)	TD (in. x in. x ft.)	BD (in. x in. x ft.)	
117-150-117	54 x 0.5 x 76	---	22 x 1.25 x 76	---	22 x 1 x 19	22 x 1.5 x 38	22 x 1 x 19	54 x 0.5 x 74	16 x 1 x 74	16 x 1 x 74	---
129-165-129	60 x 0.5 x 80	22 x 1 x 20	22 x 1.5 x 40	22 x 1 x 20	22 x 1 x 20	22 x 1.75 x 40	22 x 1 x 20	60 x 0.5 x 85	16 x 1 x 85	16 x 1 x 85	---
141-180-141	66 x 0.5 x 86	22 x 1 x 26	22 x 1.5 x 34	22 x 1 x 26	22 x 1 x 26	22 x 1.75 x 34	22 x 1 x 26	66 x 0.5 x 94	16 x 1 x 94	16 x 1 x 94	---
153-195-153	72 x 0.625 x 94	24 x 1 x 23	24 x 1.5 x 48	24 x 1 x 23	24 x 1 x 23	24 x 1.75 x 48	24 x 1 x 23	72 x 0.625 x 101	18 x 1 x 101	18 x 1 x 101	---
164-210-164	76 x 0.625 x 102	25 x 1.25 x 35	25 x 1.5 x 32	25 x 1.25 x 35	25 x 1.25 x 35	25 x 1.75 x 32	25 x 1.25 x 35	76 x 0.625 x 108	18 x 1 x 108	18 x 1 x 108	---
176-225-176	82 x 0.625 x 108	24 x 1 x 27	24 x 1.75 x 54	24 x 1 x 27	24 x 1 x 27	24 x 2 x 54	24 x 1 x 27	82 x 0.625 x 117	18 x 1 x 117	18 x 1 x 117	---
188-240-188	88 x 0.625 x 116	28 x 1.25 x 38	28 x 1.5 x 40	28 x 1.25 x 38	28 x 1.25 x 38	28 x 1.75 x 40	28 x 1.25 x 38	88 x 0.625 x 124	20 x 1 x 124	20 x 1 x 124	---
199-255-199	92 x 0.625 x 122	28 x 1.25 x 41	28 x 1.75 x 40	28 x 1.25 x 41	28 x 1.25 x 41	28 x 2 x 40	28 x 1.25 x 41	92 x 0.625 x 133	20 x 1 x 133	20 x 1 x 133	a
211-270-211	96 x 0.75 x 125	28 x 1.25 x 40	28 x 1.75 x 40	28 x 1.25 x 45	28 x 1.25 x 40	28 x 2 x 40	28 x 1.25 x 45	96 x 0.75 x 140	20 x 1 x 140	20 x 1 x 140	a
223-285-223	102 x 0.75 x 135	28 x 1.25 x 31	28 x 2 x 62	28 x 1.25 x 42	28 x 1.25 x 31	28 x 2 x 62	28 x 1.25 x 42	102 x 0.75 x 139	22 x 1 x 139	22 x 1 x 139	a
234-300-234	108 x 0.75 x 140	28 x 1.25 x 30	28 x 2 x 60	28 x 1.25 x 50	28 x 1.25 x 30	28 x 2 x 60	28 x 1.25 x 50	108 x 0.75 x 140	22 x 1 x 140	22 x 1 x 140	a

Note: All plates are A709 Gr 50W.

Footnotes:

a. AASHTO distribution factor equations were used with girder stiffness and / or span length exceeding AASHTO limits. Check with refined analysis.

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Span, ft. End-Int.-End	SEGMENT A WA (in. x in. x ft.)	TA1 (in. x in. x ft.)	TA2 (in. x in. x ft.)	BA1 (in. x in. x ft.)	BA2 (in. x in. x ft.)	SEGMENT B (as needed) WB (in. x in. x ft.)	TB (in. x in. x ft.)	BB (in. x in. x ft.)				
153-195-153	72 x 0.625 x 94	24 x 1 x 23	24 x 1.5 x 48	24 x 1 x 23	24 x 1 x 23	24 x 1.75 x 48	24 x 1 x 23	72 x 0.625 x 101	18 x 1 x 101	18 x 1 x 101	---
164-210-164	76 x 0.625 x 102	25 x 1.25 x 35	25 x 1.5 x 32	25 x 1.25 x 35	25 x 1.25 x 35	25 x 1.75 x 32	25 x 1.25 x 35	76 x 0.625 x 108	18 x 1 x 108	18 x 1 x 108	---
176-225-176	82 x 0.625 x 108	24 x 1 x 27	24 x 1.75 x 54	24 x 1 x 27	24 x 1 x 27	24 x 2 x 54	24 x 1 x 27	82 x 0.625 x 117	18 x 1 x 117	18 x 1 x 117	---
188-240-188	88 x 0.625 x 116	28 x 1.25 x 38	28 x 1.5 x 40	28 x 1.25 x 38	28 x 1.25 x 38	28 x 1.75 x 40	28 x 1.25 x 38	88 x 0.625 x 124	20 x 1 x 124	20 x 1 x 124	---
199-255-199	92 x 0.625 x 122	28 x 1.25 x 41	28 x 1.75 x 40	28 x 1.25 x 41	28 x 1.25 x 41	28 x 2 x 40	28 x 1.25 x 41	92 x 0.625 x 133	20 x 1 x 133	20 x 1 x 133	a
211-270-211	96 x 0.75 x 125	28 x 1.25 x 40	28 x 1.75 x 40	28 x 1.25 x 45	28 x 1.25 x 40	28 x 2 x 40	28 x 1.25 x 45	96 x 0.75 x 140	20 x 1 x 140	20 x 1 x 140	a
223-285-223	102 x 0.75 x 135	28 x 1.25 x 31	28 x 2 x 62	28 x 1.25 x 42	28 x 1.25 x 31	28 x 2 x 62	28 x 1.25 x 42	102 x 0.75 x 139	22 x 1 x 139	22 x 1 x 139	a
234-300-234	108 x 0.75 x 140	28 x 1.25 x 30	28 x 2 x 60	28 x 1.25 x 50	28 x 1.25 x 30	28 x 2 x 60	28 x 1.25 x 50	108 x 0.75 x 140	22 x 1 x 140	22 x 1 x 140	a

Note: All plates are A709 Gr 50W.

Footnotes:
a. AASHTO distribution factor equations were used with girder stiffness and / or span length exceeding AASHTO limits. Check with refined analysis.

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TRANSVERSE AND BEARING STIFFENERS

Span, ft. End-Int.-End	Transverse Stiffener Size and Location, Distance From End support, Each Span				Bearing Stiffeners, End		Bearing Stiffeners, Piers	
	Width in.	Thickness in.	Span 1 Location, ft.	Span 2 Location, ft.	Width in.	Thickness in.	Width in.	Thickness in.
117-150-117	5.5	0.5	90, 103.5	13.5, 27, 123, 136.5	7.25	0.75	10.25	1
129-165-129	5.5	0.5	7.5, 99, 114	15, 30, 135, 150	7.25	0.75	10.25	1
141-180-141	6	0.5	8.25, 24.75, 81.5, 98, 108, 124.5	16.5, 33, 43, 59.5, 120.5, 137, 147, 163.5	7.25	0.75	10.25	1
153-195-153	6	0.5	135	18, 177	8.25	0.75	11.25	1
164-210-164	6.25	0.5	145	19, 191	8.25	0.75	11.75	1.125
176-225-176	6	0.5	135, 155.5	20.5, 41, 184, 204.5	8.25	0.75	11.25	1
188-240-188	7	0.5	144, 166	22, 44, 196, 218	9.25	0.875	13.25	1.25
199-255-199	7	0.5	11.5, 153, 176	23, 46, 209, 232	9.25	0.875	13.25	1.25
211-270-211	7	0.5	187	24, 246	9	0.875	13	1.125
223-285-223	7	0.5	197.5	25.5, 259.5	10	0.875	13	1.125
234-300-234	7	0.5	180, 207	27, 54, 246, 273	10	0.875	13	1.125

DEAD AND LIVE LOAD REACTIONS

Span, ft. End-Int.-End	End Reaction				Pier 1 & 2 Reaction			
	DC kips	DW kips	Truck kips	Lane kips	DC kips	DW kips	Truck kips	Lane kips
117-150-117	60	7	75	30	222	24	130	80
129-165-129	66	7	75	33	247	27	134	88
141-180-141	73	8	76	36	270	29	136	96
153-195-153	81	9	76	38	305	31	138	104
164-210-164	88	9	77	41	331	34	139	112
176-225-176	94	10	77	44	359	37	140	120
188-240-188	103	11	77	47	390	39	141	128
199-255-199	109	11	77	50	418	41	142	136
211-270-211	119	12	77	52	455	43	142	143
223-285-223	128	13	77	55	492	46	142	151
234-300-234	137	13	77	57	522	48	141	157

Note: Truck and lane reactions include distribution factors, skew correction, and impact on the truck loading.

SHEAR STUD LAYOUT

Span, ft. End-Int.-End	Studs per row	Span 1										Span 2									
		Offset in.	Group 1			Group 2			Group 3			Offset in.	Group 1			Group 2			Group 3		
			Spaces	Pitch in.	Length ft.	Spaces	Pitch in.	Length ft.	Spaces	Pitch in.	Length ft.		Spaces	Pitch in.	Length ft.	Spaces	Pitch in.	Length ft.	Spaces	Pitch in.	Length ft.
117-150-117	4	0	18	12	18	53	16	70.67	7	48	28	20	7	48	28	68	16	90.67	7	48	28
129-165-129	4	0	20	12	20	52	18	78	7	48	28	12	8	48	32	99	12	99	8	48	32
141-180-141	4	0	14	12	14	62	18	93	8	48	32	0	9	48	36	75	18	108	9	48	36
153-195-153	4	0	8	12	8	72	18	108	9	48	36	36	9	48	36	78	18	117	9	48	36
164-210-164	4	0	9	12	9	71	18	106.5	12	48	48	24	10	48	40	84	18	126	10	48	40
176-225-176	4	12	82	18	123	4	24	8	11	48	44	48	10	48	40	91	18	136.5	11	48	44
188-240-188	4	0	19	18	28.5	57	24	114	11	48	44	30	11	48	44	73	24	146	11	48	44
199-255-199	4	0	14	18	21	65	24	130	12	48	48	6	12	48	48	79	24	158	12	48	48
211-270-211	4	0	7	18	10.5	74	24	148	13	48	52	24	13	48	52	81	24	162	13	48	52
223-285-223	4	0	18	23	34.5	57	28	133	13	48	52	6	14	48	56	74	28	172.67	13	48	52
234-300-234	4	0	18	24	36	56	30	140	14	48	56	24	14	48	56	73	30	182.5	14	48	56

GIRDER WEIGHT

Span, ft. End-Int.-End	Segment A tons	Segment B tons	Segment C tons	Segment D tons	Total tons
117-150-117	8.47	---	10.60	7.43	45.57
129-165-129	9.99	---	11.94	8.97	52.84
141-180-141	11.50	---	12.86	10.40	59.12
153-195-153	14.61	---	17.32	13.92	77.78
164-210-164	16.05	---	20.11	15.34	87.67
176-225-176	18.11	---	22.10	17.37	97.78
188-240-188	21.01	---	26.10	20.04	114.27
199-255-199	22.89	---	28.85	22.06	125.54
211-270-211	9.72	19.06	32.58	26.68	149.39
223-285-223	10.46	22.55	38.08	28.50	170.67
234-300-234	11.48	26.34	40.25	29.77	185.93

Note: Girder weight is total weight of web and flanges only measured between CL brg at each end. Does not include girder extension at end bearings, stiffeners, shear studs, bracing, or any other allowances.

CROSS-FRAME SPACING

Span, ft. End-Int.-End	End Span	Interior Span	Type
117-150-117	4 @ 20.5 + 2 @ 17.5 = 117	2 @ 17.5 + 3 @ 26.66 + 2 @ 17.5 = 150	K-Frame
129-165-129	4 @ 23 + 2 @ 18.5 = 129	2 @ 18.5 + 4 @ 22.75 + 2 @ 18.5 = 165	K-Frame
141-180-141	4 @ 25.25 + 2 @ 20 = 141	2 @ 20 + 4 @ 25 + 2 @ 20 = 180	K-Frame
153-195-153	5 @ 22 + 2 @ 21.5 = 153	2 @ 21.5 + 4 @ 27.25 + 2 @ 21.5 = 195	K-Frame
164-210-164	5 @ 23 + 3 @ 16.33 = 164	3 @ 16.25 + 5 @ 22.5 + 3 @ 16.25 = 210	K-Frame
176-225-176	5 @ 25 + 3 @ 17 = 176	3 @ 16.66 + 5 @ 25 + 3 @ 16.66 = 225	X-Frame
188-240-188	5 @ 26.5 + 3 @ 18.5 = 188	3 @ 17.91 + 5 @ 26.5 + 3 @ 17.91 = 240	X-Frame
199-255-199	6 @ 23.5 + 3 @ 19.33 = 199	3 @ 18.75 + 5 @ 28.5 + 3 @ 18.75 = 255	X-Frame
211-270-211	6 @ 24.67 + 3 @ 21 = 211	3 @ 21 + 6 @ 24 + 3 @ 21 = 270	X-Frame
223-285-223	6 @ 26.5 + 3 @ 21.33 = 223	4 @ 17.5 + 6 @ 24.16 + 4 @ 17.5 = 285	X-Frame
234-300-234	8 @ 23.25 + 3 @ 16 = 234	4 @ 19 + 6 @ 24.66 + 4 @ 19 = 300	X-Frame

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8 FT SPACING

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Design Resources

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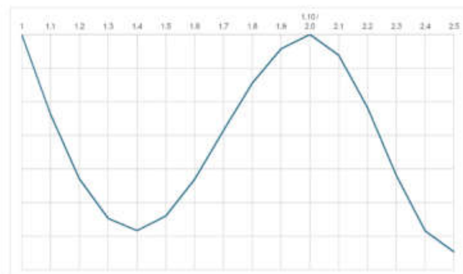
DEAD LOAD DEFLECTIONS, SPAN 1 AND L/2 SPAN 2 SHOWN, SYMMETRIC																	
Span, ft. End-Int.-End	Span Tenth Points and Deflections, in. Span 1											Span Tenth Points and Deflections, in. Span 2					
	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	1.10	2.0	2.1	2.2	2.3	2.4	2.5
117-150-117 ft. span - steel only, in.	0.00	0.13	0.24	0.31	0.33	0.32	0.26	0.18	0.09	0.03	0.00	0.00	0.05	0.16	0.28	0.38	0.41
slab, in.	0.00	0.53	0.96	1.25	1.35	1.26	1.02	0.69	0.34	0.09	0.00	0.00	0.25	0.74	1.30	1.73	1.89
barrier rails, in.	0.00	0.10	0.18	0.23	0.25	0.24	0.19	0.13	0.06	0.01	0.00	0.00	0.05	0.16	0.27	0.35	0.39
117-150-117 ft. span - total, in.	**0.00**	**0.75**	**1.38**	**1.79**	**1.94**	**1.82**	**1.47**	**1.00**	**0.50**	**0.13**	**0.00**	**0.00**	**0.35**	**1.06**	**1.85**	**2.46**	**2.68**
129-165-129 ft. span - steel only, in.	0.00	0.15	0.28	0.37	0.40	0.38	0.31	0.21	0.11	0.03	0.00	0.00	0.05	0.16	0.30	0.40	0.44
slab, in.	0.00	0.61	1.11	1.44	1.55	1.45	1.17	0.78	0.39	0.10	0.00	0.00	0.24	0.75	1.35	1.82	1.99
barrier rails, in.	0.00	0.12	0.21	0.28	0.30	0.28	0.23	0.15	0.08	0.02	0.00	0.00	0.06	0.17	0.29	0.39	0.43
129-165-129 ft. span - total, in.	**0.00**	**0.88**	**1.61**	**2.08**	**2.25**	**2.10**	**1.70**	**1.14**	**0.58**	**0.16**	**0.00**	**0.00**	**0.34**	**1.07**	**1.94**	**2.61**	**2.85**
141-180-141 ft. span - steel only, in.	0.00	0.19	0.35	0.45	0.49	0.47	0.38	0.26	0.14	0.04	0.00	0.00	0.06	0.21	0.38	0.51	0.56
slab, in.	0.00	0.72	1.32	1.70	1.84	1.72	1.39	0.93	0.46	0.12	0.00	0.00	0.28	0.92	1.66	2.23	2.43
barrier rails, in.	0.00	0.14	0.26	0.33	0.36	0.34	0.27	0.18	0.09	0.02	0.00	0.00	0.07	0.20	0.36	0.48	0.52
141-180-141 ft. span - total, in.	**0.00**	**1.05**	**1.92**	**2.49**	**2.69**	**2.53**	**2.05**	**1.38**	**0.68**	**0.18**	**0.00**	**0.00**	**0.40**	**1.33**	**2.40**	**3.21**	**3.51**
153-195-153 ft. span - steel only, in.	0.00	0.23	0.42	0.55	0.59	0.55	0.45	0.31	0.16	0.05	0.00	0.00	0.08	0.26	0.47	0.63	0.69
slab, in.	0.00	0.77	1.41	1.82	1.97	1.84	1.50	1.01	0.52	0.15	0.00	0.00	0.25	0.82	1.50	2.05	2.22
barrier rails, in.	0.00	0.16	0.29	0.38	0.41	0.39	0.31	0.21	0.11	0.03	0.00	0.00	0.06	0.19	0.34	0.46	0.50
153-195-153 ft. span - total, in.	**0.00**	**1.16**	**2.12**	**2.74**	**2.97**	**2.78**	**2.26**	**1.53**	**0.79**	**0.23**	**0.00**	**0.00**	**0.39**	**1.27**	**2.31**	**3.11**	**3.41**
164-210-164 ft. span - steel only, in.	0.00	0.28	0.51	0.66	0.71	0.66	0.54	0.36	0.18	0.05	0.00	0.00	0.11	0.35	0.62	0.82	0.90
slab, in.	0.00	0.89	1.63	2.11	2.27	2.13	1.72	1.16	0.58	0.15	0.00	0.00	0.32	1.04	1.87	2.50	2.73
barrier rails, in.	0.00	0.19	0.34	0.44	0.48	0.45	0.36	0.25	0.12	0.03	0.00	0.00	0.08	0.25	0.43	0.57	0.62
164-210-164 ft. span - total, in.	**0.00**	**1.35**	**2.47**	**3.20**	**3.46**	**3.24**	**2.62**	**1.76**	**0.89**	**0.22**	**0.00**	**0.00**	**0.52**	**1.64**	**2.91**	**3.89**	**4.25**
176-225-176 ft. span - steel only, in.	0.00	0.32	0.58	0.75	0.80	0.75	0.61	0.41	0.21	0.06	0.00	0.00	0.10	0.33	0.62	0.83	0.91
slab, in.	0.00	0.98	1.79	2.31	2.49	2.32	1.87	1.26	0.64	0.19	0.00	0.00	0.29	0.98	1.83	2.50	2.75
barrier rails, in.	0.00	0.21	0.38	0.49	0.53	0.50	0.40	0.27	0.14	0.04	0.00	0.00	0.08	0.24	0.44	0.59	0.64
176-225-176 ft. span - total, in.	**0.00**	**1.50**	**2.74**	**3.55**	**3.82**	**3.57**	**2.89**	**1.94**	**0.99**	**0.29**	**0.00**	**0.00**	**0.46**	**1.56**	**2.88**	**3.92**	**4.30**
188-240-188 ft. span - steel only, in.	0.00	0.36	0.56	0.86	0.93	0.87	0.71	0.48	0.25	0.07	0.00	0.00	0.13	0.43	0.76	1.02	1.12
slab, in.	0.00	1.02	1.37	2.43	2.62	2.46	1.99	1.35	0.70	0.20	0.00	0.00	0.33	1.12	2.03	2.73	3.00
barrier rails, in.	0.00	0.22	0.41	0.53	0.57	0.54	0.44	0.30	0.15	0.04	0.00	0.00	0.09	0.27	0.49	0.65	0.71
188-240-188 ft. span - total, in.	**0.00**	**1.61**	**2.34**	**3.81**	**4.12**	**3.87**	**3.14**	**2.13**	**1.10**	**0.31**	**0.00**	**0.00**	**0.55**	**1.82**	**3.28**	**4.40**	**4.82**
199-255-199 ft. span - steel only, in.	0.00	0.41	0.75	0.97	1.05	0.98	0.79	0.53	0.27	0.08	0.00	0.00	0.14	0.46	0.84	1.13	1.24
slab, in.	0.00	1.14	2.08	2.68	2.89	2.70	2.18	1.46	0.74	0.21	0.00	0.00	0.34	1.18	2.19	2.97	3.26
barrier rails, in.	0.00	0.25	0.46	0.59	0.64	0.60	0.49	0.33	0.17	0.05	0.00	0.00	0.09	0.30	0.54	0.72	0.79
199-255-199 ft. span - total, in.	**0.00**	**1.80**	**3.29**	**4.25**	**4.58**	**4.28**	**3.45**	**2.32**	**1.18**	**0.33**	**0.00**	**0.00**	**0.56**	**1.95**	**3.57**	**4.83**	**5.29**
211-270-211 ft. span - steel only, in.	0.00	0.51	0.93	1.20	1.29	1.21	0.97	0.65	0.32	0.07	0.00	0.00	0.20	0.63	1.12	1.50	1.64
slab, in.	0.00	1.24	2.26	2.93	3.16	2.95	2.38	1.59	0.79	0.20	0.00	0.00	0.42	1.39	2.52	3.39	3.72
barrier rails, in.	0.00	0.28	0.51	0.66	0.71	0.67	0.54	0.36	0.18	0.04	0.00	0.00	0.11	0.35	0.62	0.82	0.90
211-270-211 ft. span - total, in.	**0.00**	**2.02**	**3.70**	**4.78**	**5.16**	**4.82**	**3.89**	**2.60**	**1.29**	**0.31**	**0.00**	**0.00**	**0.73**	**2.38**	**4.26**	**5.71**	**6.25**
223-285-223 ft. span - steel only, in.	0.00	0.55	1.00	1.29	1.39	1.29	1.04	0.69	0.33	0.08	0.00	0.00	0.22	0.67	1.19	1.59	1.74
slab, in.	0.00	1.25	2.29	2.96	3.18	2.97	2.39	1.60	0.80	0.21	0.00	0.00	0.41	1.34	2.43	3.29	3.61
barrier rails, in.	0.00	0.29	0.53	0.68	0.74	0.69	0.56	0.38	0.19	0.05	0.00	0.00	0.11	0.35	0.62	0.83	0.90
223-285-223 ft. span - total, in.	**0.00**	**2.09**	**3.31**	**4.93**	**5.31**	**4.95**	**3.98**	**2.66**	**1.32**	**0.34**	**0.00**	**0.00**	**0.74**	**2.35**	**4.24**	**5.71**	**6.25**
234-300-234 ft. span - steel only, in.	0.00	0.60	1.10	1.41	1.52	1.41	1.14	0.75	0.37	0.09	0.00	0.00	0.24	0.77	1.37	1.82	1.99
slab, in.	0.00	1.28	2.34	3.01	3.23	3.01	2.42	1.61	0.79	0.20	0.00	0.00	0.48	1.53	2.77	3.73	4.09
barrier rails, in.	0.00	0.30	0.55	0.71	0.76	0.71	0.58	0.39	0.19	0.05	0.00	0.00	0.13	0.40	0.70	0.93	1.02
234-300-234 ft. span - total, in.	**0.00**	**2.18**	**3.98**	**5.12**	**5.51**	**5.14**	**4.13**	**2.75**	**1.35**	**0.34**	**0.00**	**0.00**	**0.85**	**2.70**	**4.84**	**6.49**	**7.10**



DEFLECTION VERSUS SPAN TENTH POINT, SYMMETRIC ABOUT L/2 SPAN 2

Exterior and First Interior Girder

Deflection Assumptions

"Steel Only" = self weight of girders

"Slab" = deflection due to user-input non composite uniform dead load (slab, haunch, allowance for bracing)

"Barrier Rails" = deflection due to barrier rail loading distributed evenly to exterior and first interior girder.

THREE SPAN 150-300 FT
8 FT SPACING

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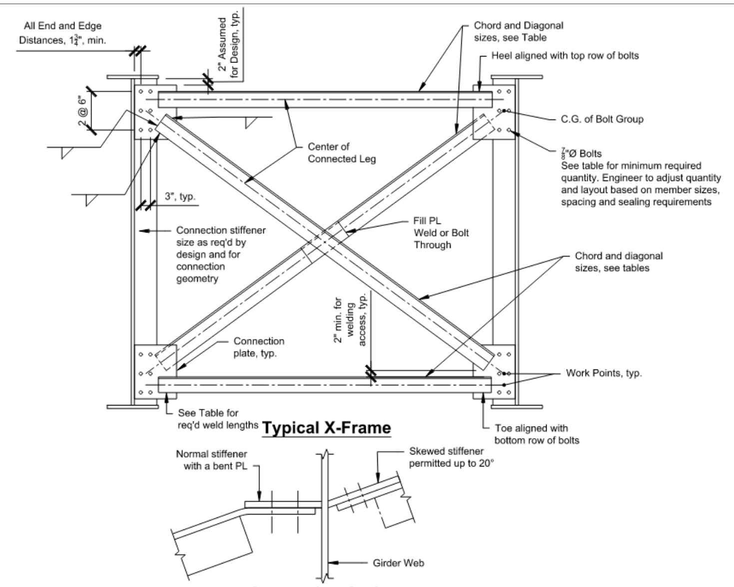
Design Resources

Standard Plans for Steel Bridges



Connection Options

Not to Scale

CROSS-FRAME DETAILS				
Beam Spacing, ft.	Span, ft. End-Int.-End	Type	Chord	Diagonal
8	117-150-117 through 164-210-164	K-Frame	L5X5X3/8	L5X5X3/8
	176-225-176 through 234-300-234	X-Frame	L5X5X3/8	L6x6x3/8
10	117-150-117 through 199-255-199	K-Frame	L5X5X3/8	L5X5X3/8
	211-270-211 through 234-300-234	X-Frame	L5X5X3/8	L6X6X3/8
12	All spans	K-Frame	L6X6X3/8	L5X5X3/8
14	141-180-141 through 234-300-234	K-Frame	L8X6X1/2	L5X5X3/8

CROSS-FRAME WELD DETAILS		
Angle Size	Toe Length	Heel Length
L5x5x3/8	2 in. min. See notes regarding toe weld length	4 in.
L6x6x3/8		4 in.
L8x6x1/2		4

CROSS-FRAME BOLTED CONNECTION DETAILS					
Beam Spacing, ft.	Type	Top Connection		Bottom Connection	
		Total Num Bolts	Vertical Spacing	Total Num Bolts	Vertical Spacing
8	K-Frame	6	6 in.	2	3 in.
	X-Frame	6	6 in.	6	6 in.
10	K-Frame	6	6 in.	2	3 in.
	X-Frame	6	6 in.	6	6 in.
12	K-Frame	6	6 in.	2	3 in.
14	K-Frame	8	4.75	4	4.75

Notes:

1. All bolts for bent plate diaphragms 7/8 in. diameter ASTM F3125 Grade A325 bolts assumed in single shear with threads in the shear plane.
2. All bolts for K and X cross-frames 7/8 in. diameter ASTM F3125 Grade A325 bolts assumed in single shear with threads in the shear plane.
3. All welds 5/16 in. fillet welds. The minimum heel and toe dimensions provided meet load and eccentricity requirements. The toe may be lengthened to equal the heel dimension provided in the tables; the resulting eccentricity was considered in design. Other weld geometries may be needed for dimensional or sealing requirements and are to be designed.
4. Member and connection designs based on stability, construction, and wind forces.
5. General layout and details follow industry preferences. Provide details in accordance with owner preferences and modify these details accordingly.
6. Determine cross-frame forces for specific designs and proportion members and connections accordingly. Bolt connection layout, quantity and spacing provided on this sheet are approximate based on member loads and several representative geometries. Deck cross-slope was not considered in developing the details. Given a wide range of beam depths and spacing, the geometry of each connection was not fully studied. A scale drawing of the connection including chosen work points should be used for layout of the members, final bolt patterns, and determination of connection plate sizes. The selection of work points, member axes and orientation shown represent one acceptable approach. Engineers may choose alternate work point locations and overall connection geometries that consider the effects of eccentricity on the welded and bolted connections.

CROSS-FRAME & DIAPHRAGM DETAILS 1

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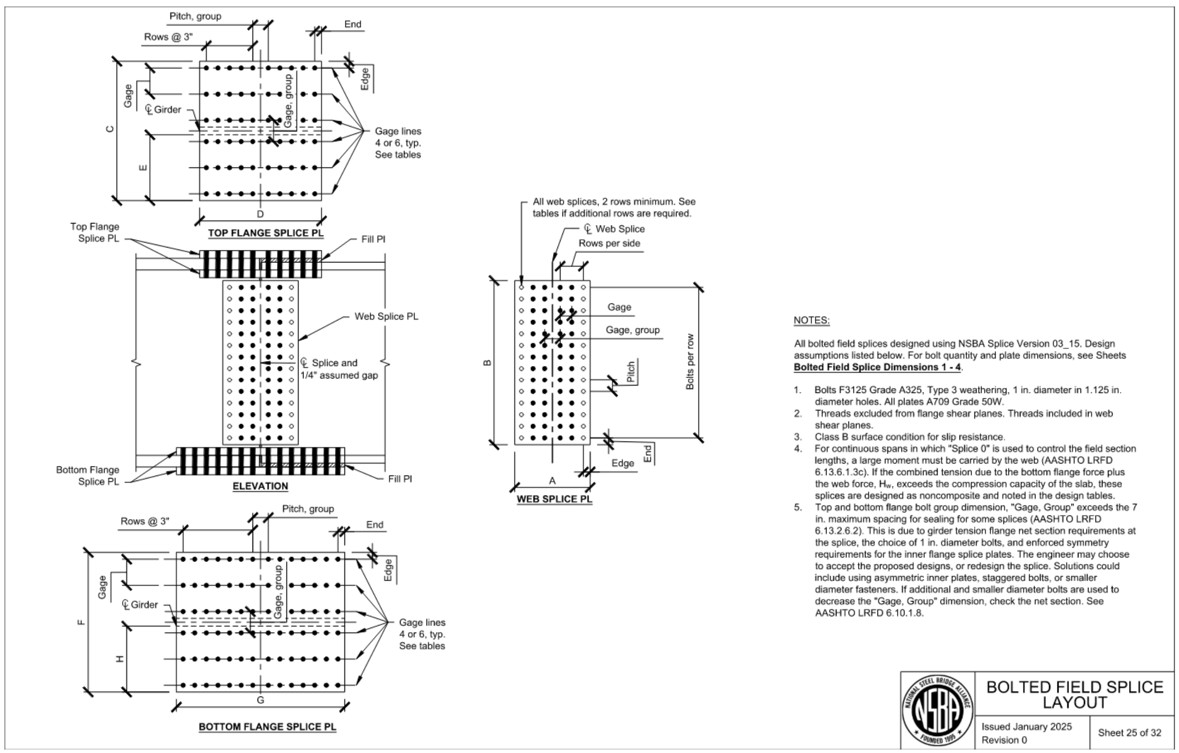
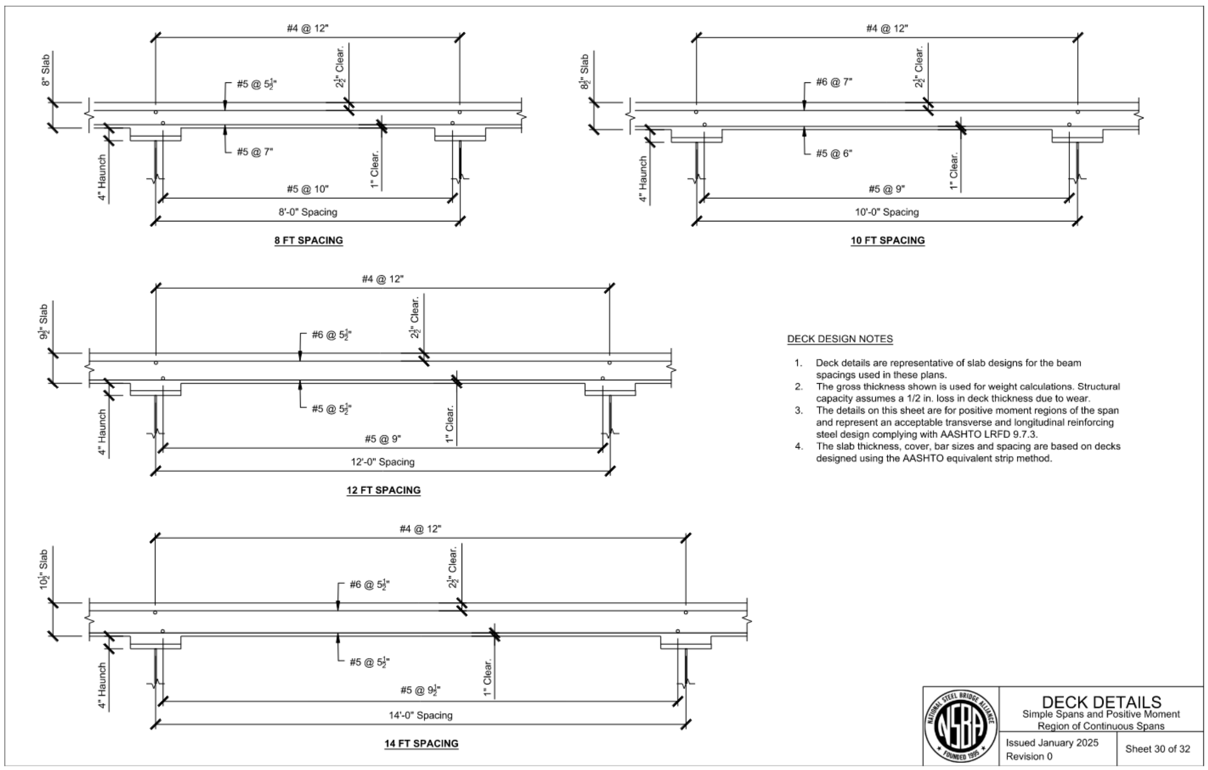
Design Resources

Standard Plans for Steel Bridges



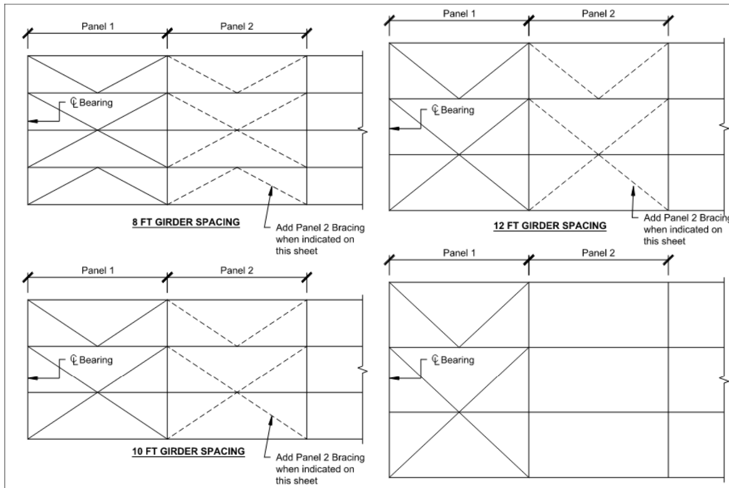
Design Resources

Standard Plans for Steel Bridges



Design Resources

Standard Plans for Steel Bridges

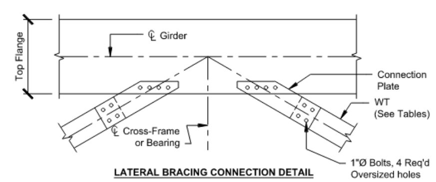


SPACING	SPAN LENGTH	NUMBER OF BRACED PANELS	BRACING SIZE
8	190	1	WT 7 x 17
8	200	1	WT 7 x 17
8	210	1	WT 7 x 24
8	220	1	WT 7 x 24
8	230	1	WT 7 x 24
8	240	1	WT 7 x 30.5
8	250	1	WT 7 x 30.5
8	260	1	WT 8 x 33.5
8	270	1	WT 8 x 33.5
8	280	2	WT 8 x 33.5
8	290	2	WT 8 x 33.5
8	300	2	WT 8 x 33.5

SPACING	SPAN LENGTH	NUMBER OF BRACED PANELS	BRACING SIZE
10	180	1	WT 7 x 17
10	190	1	WT 7 x 17
10	200	1	WT 7 x 19
10	210	1	WT 7 x 19
10	220	1	WT 7 x 24
10	230	1	WT 7 x 24
10	240	1	WT 8 x 33.5
10	250	1	WT 8 x 33.5
10	260	1	WT 8 x 33.5
10	270	1	WT 8 x 33.5
10	280	2	WT 8 x 33.5
10	290	2	WT 8 x 33.5
10	300	2	WT 8 x 33.5

SPACING	SPAN LENGTH	NUMBER OF BRACED PANELS	BRACING SIZE
12	220	1	WT 7 x 26.5
12	230	1	WT 7 x 26.5
12	240	1	WT 7 x 26.5
12	250	1	WT 8 x 33.5
12	260	1	WT 8 x 33.5
12	270	1	WT 8 x 33.5
12	280	2	WT 8 x 33.5
12	290	2	WT 8 x 33.5
12	300	2	WT 8 x 33.5

SPACING	SPAN LENGTH	NUMBER OF BRACED PANELS	BRACING SIZE
14	230	1	WT 7 x 24
14	240	1	WT 7 x 24
14	250	1	WT 7 x 24
14	260	1	WT 7 x 34
14	270	1	WT 7 x 34
14	280	1	WT 7 x 34
14	290	1	WT 8 x 33.5
14	300	1	WT 8 x 33.5



LATERAL BRACING CONNECTION DETAIL

NOTES AND DESIGN CRITERIA:

1. Lateral bracing is required for the indicated spans. Bracing shown is needed to control lateral deflections and flange stresses due to wind on the fully erected steel, for global stability during deck placement, or a combination of those factors.
2. Lateral deflections due to wind loads on the fully erected steel satisfy Span / 150 requirement established by PennDOT BD-620M. All references to BD-620M are to the April 29, 2016 edition.
   2.1. For this deflection check, a 32 psf assumed pressure is applied to fascia beams only for a superstructure height = 30 ft. For other superstructure heights, refer to PennDOT BD-620M.
3. Girder flange lateral bending is checked and lateral bracing is designed for strength as follows:
   3.1. Midspan region checked. Check other plate transitions in final design.
   3.2. Fascia beam checked for global bending of the span and local bending between cross-frames.
   3.3. Wind loads on erected steel determined from the *AASHTO Guide Specification for Wind Loads on Bridges During Construction, 2017.*
      3.3.1. Inactive wind condition, V = 115 mph. Superstructure height, 30 ft.
      3.3.2. Superstructure construction duration 6 weeks - 1 year, R = 0.73
      3.3.3. $K_z = 1.0$, $C_d = 2.2$ for fascia beam, per AASHTO Guide Specifications for other beams
4. Lateral bracing members were designed to transfer the *Guide Specification* wind load at each end of the fully erected span.
5. Bracing members were designed as eccentrically loaded WTs in compression using *Tables for Eccentrically Loaded WT Shapes in Compression*, AISC Engineering Journal, Second Quarter, 2010.
6. Lateral bracing bolts are designed as bearing type connections for the inactive wind condition, AASHTO LRFD 6.13.2.1.2. Lateral bracing bolts additionally designed to prevent slip during deck casting, AASHTO LRFD 6.13.2.1.1 and 6.10.3.1. For the determination of bolt slip resistance, oversized holes in one ply are permitted. Provide a minimum of a Class A surface condition.

LATERAL BRACING DETAILS

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Design Resources

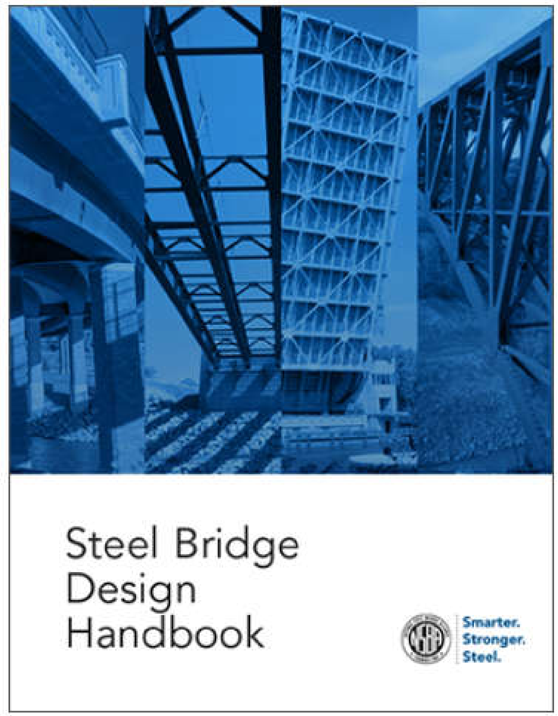
- Steel Bridge Design Handbook

STEEL BRIDGE DESIGN HANDBOOK

- Bridge Steels and Their Mechanical Properties - Chapter 1
- Steel Bridge Fabrication - Chapter 2
- Structural Steel Bridge Shop Drawings - Chapter 3
- Strength Behavior and Design of Steel - Chapter 4
- Selecting the Right Bridge Type - Chapter 5
- Stringer Bridges and Making the Right Choices - Chapter 6

DESIGN CONSIDERATIONS

- Loads and Load Combinations - Chapter 7
- Structural Analysis - Chapter 8
- Redundancy - Chapter 9
- Limit States - Chapter 10
- Design for Constructability - Chapter 11
- Design for Fatigue - Chapter 12
- Bracing System Design - Chapter 13
- Splice Design - Chapter 14
- Bearing Design - Chapter 15
- Substructure Design - Chapter 16
- Bridge Deck Design - Chapter 17
- Load Rating of Steel Bridges - Chapter 18
- Corrosion Protection of Steel Bridge - Chapter 19



https://www.aisc.org/nsba/design-and-estimation-resources/steel-bridge-design-handbook/

Design Resources

- Steel Bridge Design Handbook

DESIGN EXAMPLE APPENDICES

- Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
- Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
- Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- Design Example 3: Three-Span Continuous Horizontally Curved Composite Steel I-Girder Beam Bridge
- Design Example 4: Three-Span Continuous Straight Composite Steel Tub-Girder Bridge
- Design Example 5: Three-Span Continuous Horizontally Curved Composite Steel Tub-Girder Bridge

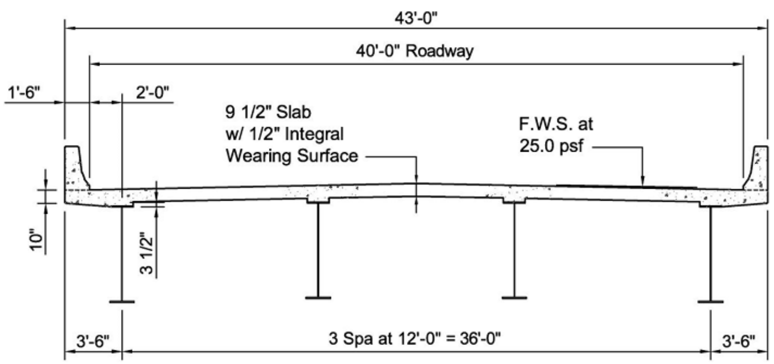


Figure 1: Typical Bridge Cross-Section

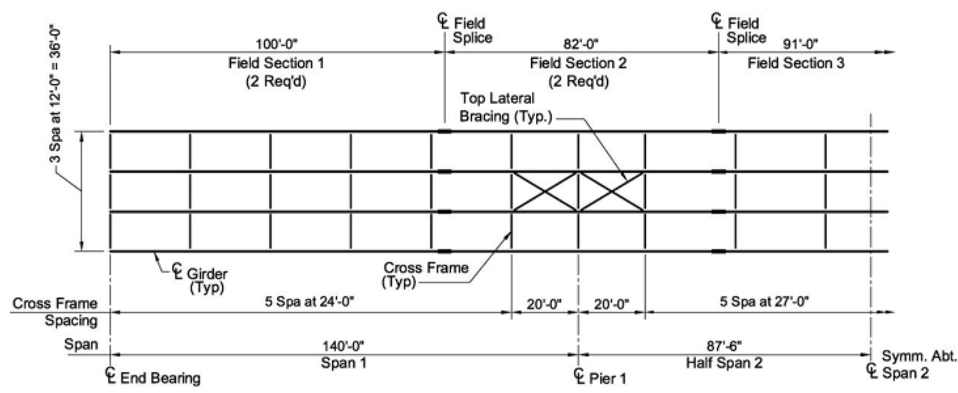


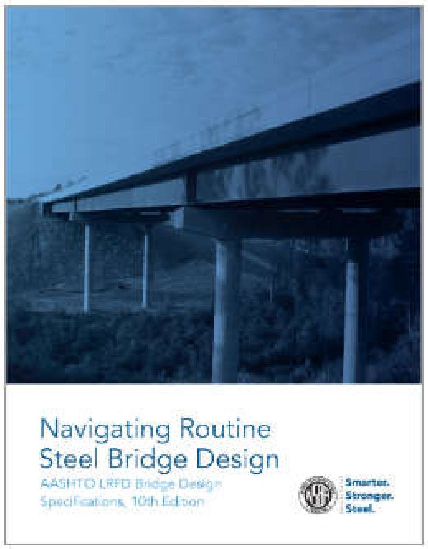
Figure 2: Framing Plan

Note: Top Lateral Bracing members are shown only for reference and are not included in the final design.

Design Resources

- Routine Steel Bridges

NSBA Guide to Navigating Routine Steel Bridge Design



Routine steel I-girder bridges--straight bridges with little or no skew, span lengths up to 200 ft, and routine framing and girder configurations--are the workhorses of the steel bridge world.

That's why the National Steel Bridge Alliance (NSBA) has made it much easier and faster to design and check these common bridges. We teamed up with two leading consultants, HDR and MA Grubb and Associates, to make sure that the guide is accurate and practical for design engineers, DOTs, and fabricators.

Navigating Routine Steel Bridge Design complements the 10th Edition *AASHTO LRFD Bridge Design Specifications* and addresses the design of steel superstructures for routine steel I-girder bridges. This streamlined design guide is a series of hyperlinked checklists that walk engineers and designers through the process, step by step, focusing on the specific provisions of the AASHTO *Specifications* that apply to these routine bridges.

Download the guide today to start designing and checking routine steel I-girder bridges more efficiently. We recommend you use Adobe Acrobat to view it.

https://www.aisc.org/nsba/design-resources/nsba-guide-to-navigating-routine-steel-bridge-design/

Design Resources

- Routine Steel Bridges
    - AASHTO references
        - Complete discussion of code provisions
    - Industry references
        - Links to industry publications
    - Useful tools
        - Design resources

GIRDER SHEAR DESIGN

Quick links to applicable AASHTO LRFD BDS provisions, with Discussion

Design girders for shear to meet the requirements of the strength limit state, considering the following:

- General provisions (6.10.9.1), Flowchart (Figure C6.10.9.1-1)
- Nominal resistance of unstiffened webs (6.10.9.2)
- Nominal resistance of stiffened webs
    - General provisions (6.10.9.3.1)
    - Nominal resistance of interior panels (6.10.9.3.2)
    - Nominal resistance of end panels (6.10.9.3.3)

Quick links to helpful industry design guidelines, references, and examples

For more explanation and examples of girder shear design, see:

- The Reference Manual for NHI Course 130081, Load and Resistance Factor Design (LRFD) for Highway Bridge Superstructures
    - Section 6.5.7 (LRFD Strength Limit State Design for Shear)
- NSBA's Steel Bridge Design Handbook
    - Design Example 1: Three-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2A: Two-Span Continuous Straight Composite Steel I-Girder Bridge
    - Design Example 2B: Two-Span Continuous Straight Composite Steel Wide-Flange Beam Bridge
- The AASHTO-NSBA Steel Bridge Collaboration Guidelines
    - G12.1-2020 Guidelines for Design for Constructability

Quick links to useful tools

NSBA's LRFD Simon line-girder analysis and design software. Simon is available for free download from the NSBA website is also a valuable tool for the design of routine steel I-girder bridges. It calculates the design shear loads, and the corresponding shear resistances, in accordance with the provisions of the AASHTO LRFD BDS, greatly reducing the time and effort required of the designer. Other commercial software packages with the ability to analyze and design routine steel I-girder bridges are also available.

Users should verify the capabilities, assumptions, and general correctness of any program's calculations prior to initial use.

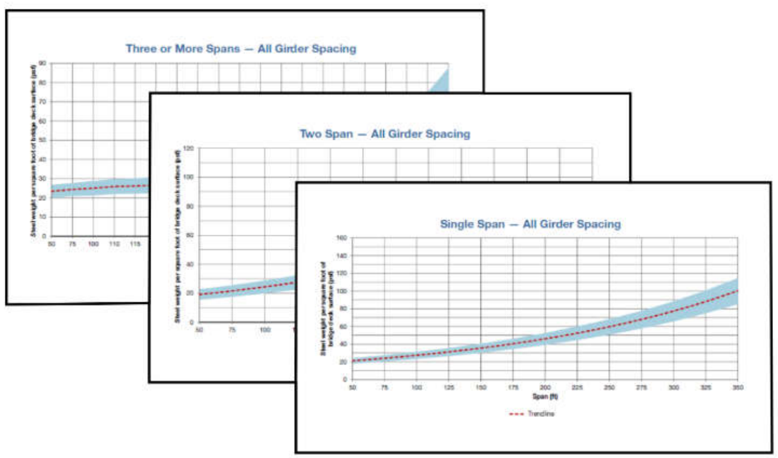
Design Resources

- Steel Span to Weight Curves

The Steel Span to Weight Curves are the quickest way to determine the weight of steel per square foot of bridge deck for straight, low skew, plate girder bridges. The Curves are organized by span arrangement (1, 2 or 3 or more span bridges) and girder spacings.

To use the graphs first determine the bridge span arrangement, then, utilizing the maximum span, find that value along the "x"-axis. Draw a line straight up until reaching the curve. Follow that line over to the "y"-axis to find the steel weight per square foot of bridge deck.

The curves are great for comparing various span arrangements and girder spacings. With some additional information the weight per square foot can easily be converted to a potential dollar value for the steel superstructure. The curves are based upon over 800 preliminary designs the NSBA has done through the years. In each instance the design was optimized for economics and is based upon standard AASHTO loading.



https://www.aisc.org/nsba/design-resources/span-to-weight-curves/

Design Resources

- LRFD Simon
  - LRFD Simon is a steel line-girder analysis and design software program for straight steel I-and box-girder bridges with limited or no skew.
  - AASHTO LRFD 9th Edition
  - Great set of release notes
  - FREE!

https://www.aisc.org/nsba/design-resources/simon/

Design Resources

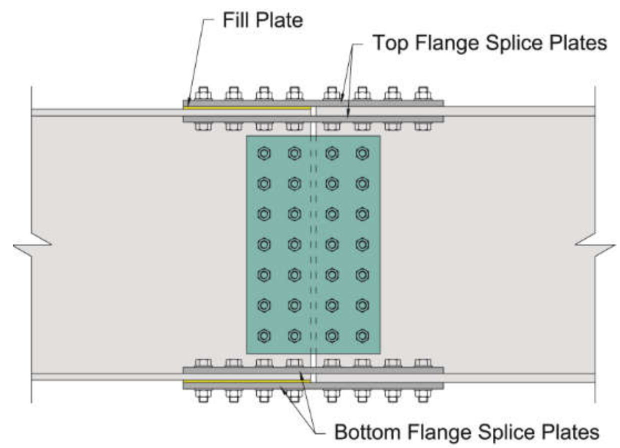
- NSBA Splice

UPDATED: NSBA Splice

NSBA Splice takes the time-consuming task of designing and checking a bolted splice connection and rewrites the process with a simple input page and output form. NSBA Splice can be incorporated as a design tool on plate girder bridges allowing the designer to quickly analyze various bolted splice connections to determine the most efficient bolt quantity and configuration. Based upon the updated AASHTO LRFD 8th Edition, NSBA Splice allows the user to explore the effects of bolt spacing, bolt size, strength, and connection dimensions on the overall splice design.

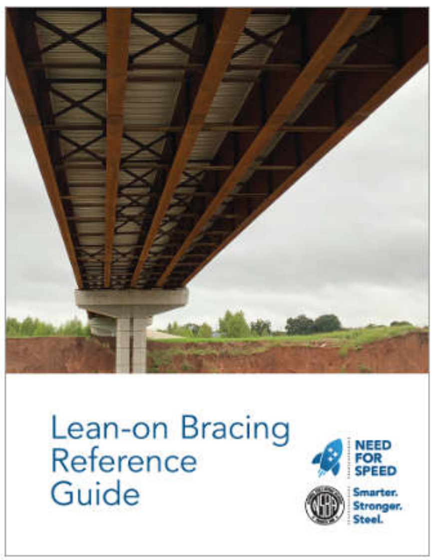
NSBA Splice is presented in an easy to understand Microsoft Excel spreadsheet format. The download includes a blank design spreadsheet as well as two completed examples drawn from the inputs and solutions for Examples 1 and 2 presented in **Bolted Field Splices for Steel Bridge Flexural Members** design guide. Both the design guide and spreadsheet are current to the AASHTO LRFD 9th Edition specification.

https://www.aisc.org/nsba/design-resources/nsba-splice/



Design Resources

- Lean-on bracing reference guide



Cross-frames are one of the costliest elements in a steel bridge on a per-pound basis. Reducing the number and complexity of cross-frames can have a significant impact on the speed of fabrication, speed of erection, and overall bridge cost. Lean-on bracing represents a way to do just that and potentially eliminate 50% or more of the full cross-frames required for a routine steel I-girder bridge without adding any cost to the girders.

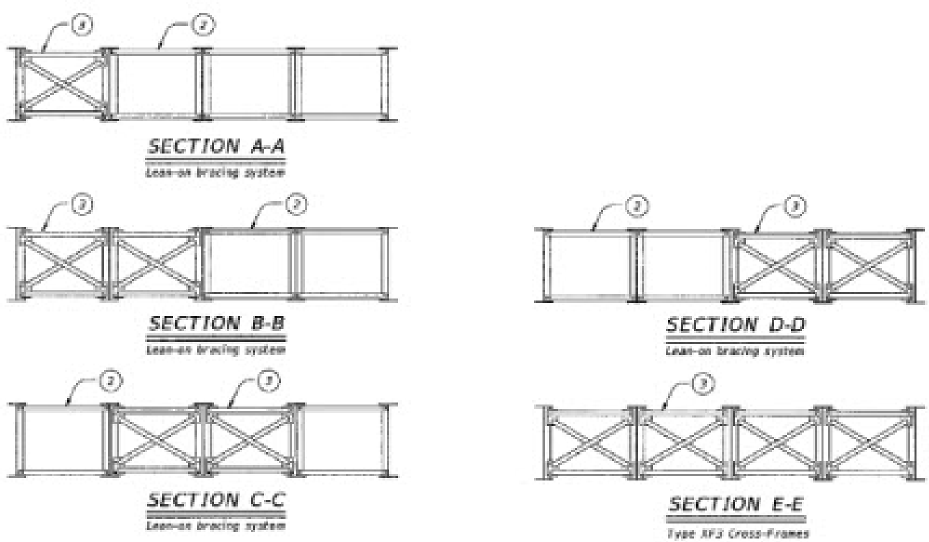
Academic research, including real bridge demonstration projects, has already been completed; however, the industry has been slow to adopt the concept. The *Lean-on Bracing Reference Guide* is written to educate designers and allay concerns about stability and strength implications. The guide provides design criteria, commentary, and example designs. It is also intended to show bridge designers how to implement lean-on bracing in routine bridge designs with confidence and with minimal computational effort beyond that required for a traditional bracing system.

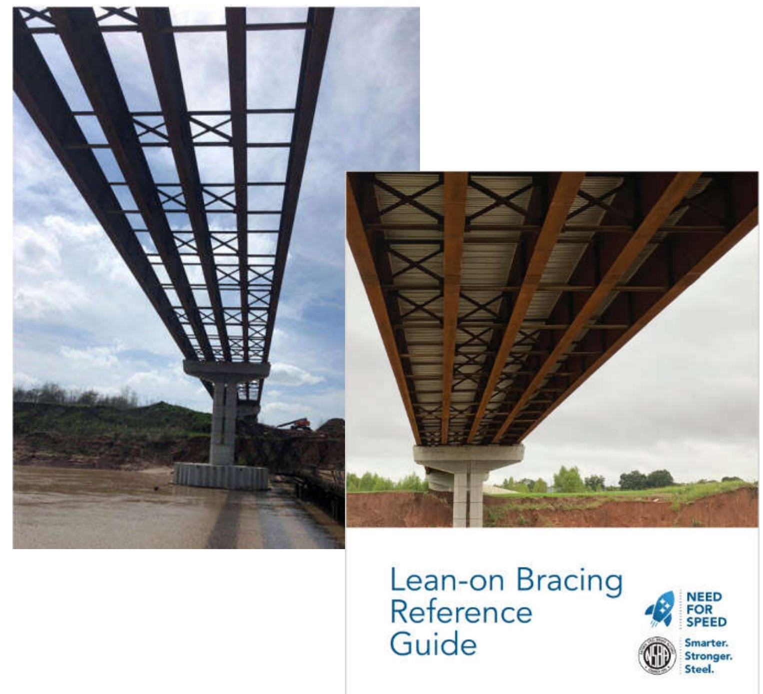
DOWNLOAD THE GUIDE

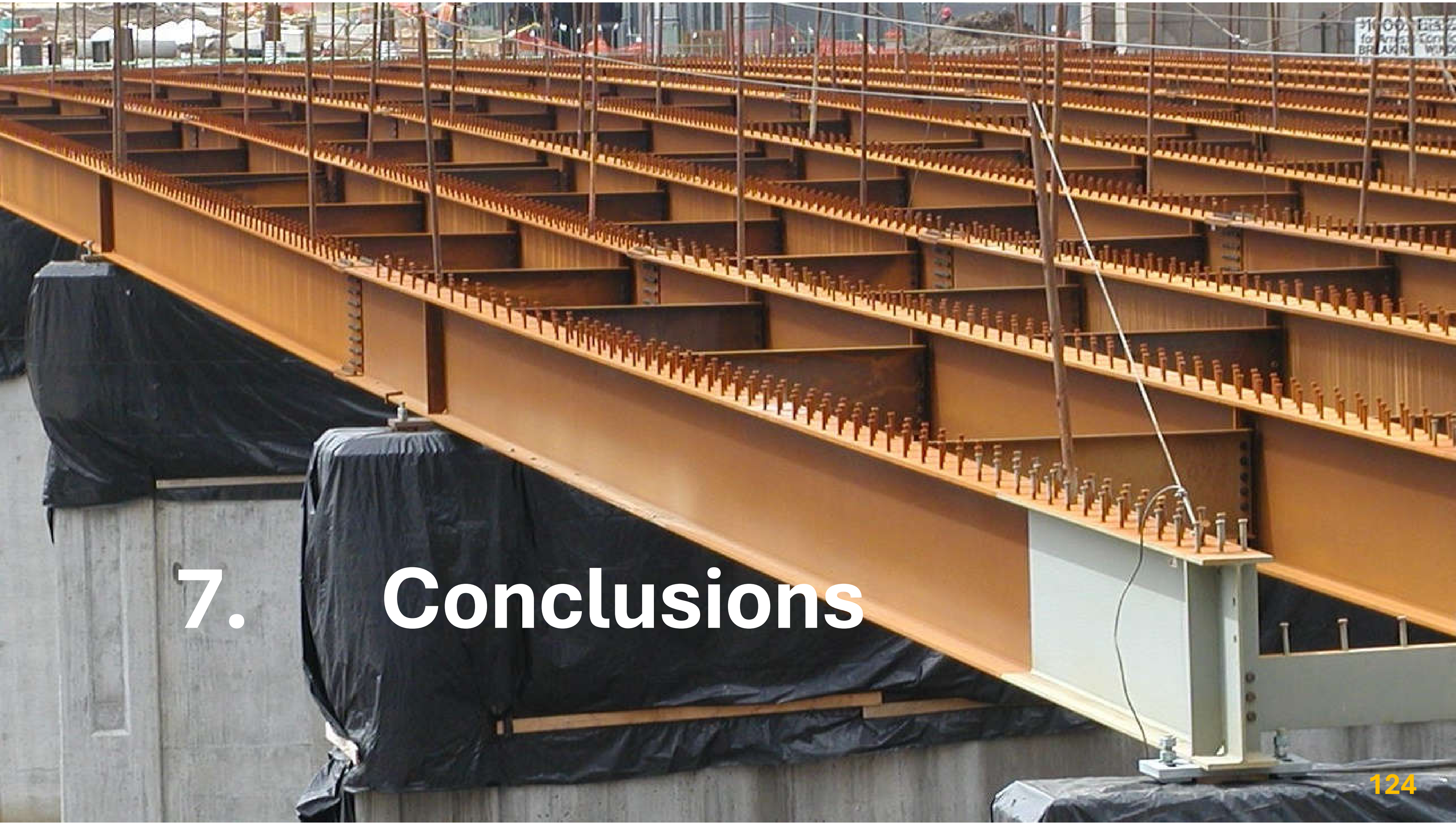
https://www.aisc.org/nsba/design-resources/lean-on-bracing-reference-guide/

Design Resources

- Lean-on bracing



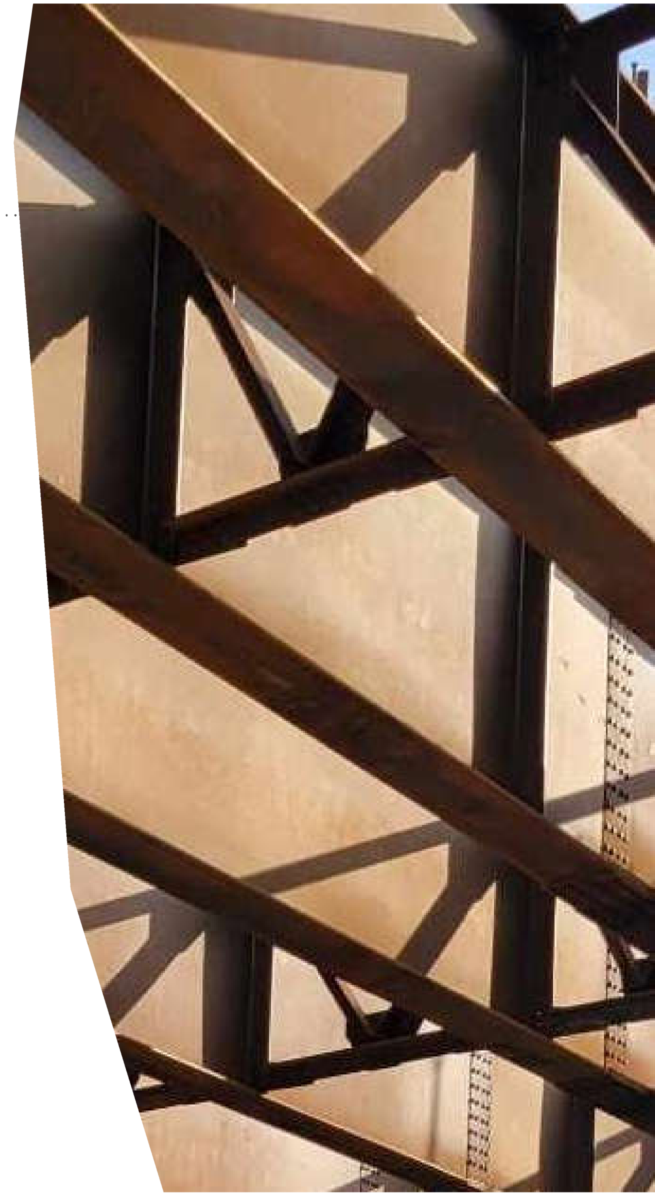




7. Conclusions

Steel Bridge Design Basics

1. Use the resources available
2. Engage fabricators with questions
3. NSBA can help!
4. Remember the rules of thumb
5. Stability needs to be looked at early
6. Wind during construction – stress and deflections
7. Use the new steel bridge standards



Questions?