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Course Description

Design of Bolted Girder Field Splices
March 14, 2017

The 8th Edition *AASHTO LRFD Bridge Design Specifications* contains several changes that impact steel connection design. This webinar will introduce the new simplified design procedure for bolted splices and will present updates on the changes to the shear and slip capacity of bolted connections.

New Method Highlights:

- Ensures that the splice can develop the yield strength of the flange and shear capacity of the web and also has the strength to resist the applied design forces
- Moment capacity is checked using a lower bound equilibrium solution, based on the flange yield capacity

The behavior of a connection designed using the new method will be compared to the older method in a finite element analysis. Design examples for a composite plate girder and a composite trapezoidal box girder will be presented. A brief introduction to a spreadsheet solution available on the NSBA website will also be included.

Learning Objectives

- Identify key changes to bolt capacity in the 8th Edition *AASHTO LRFD Bridge Design Specifications*.
- Identify key aspects to the new design method for bolted field splices introduced in the *8th Edition AASHTO LRFD Bridge Design Specifications*.
- Describe differences observed in a finite element analysis between bolted girder splices designed using the old method versus the new method.
- Locate additional resources and design aids for bolted girder field splices.

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New AASHTO Design Method for Bolted Field Splices

Presented by
Karl H. Frank
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Austin, Texas

structural STEEL

Outline

- Update on ASTM Specifications
- Changes to Bolt Design Shear Capacity
- Bolted Field Splice Design
  - Overview of Method
  - Plate Girder Splice Design
    - Comparison with FEA Analysis- Old vs. New
  - Trapezoidal Box (Tub) Girder Splice
- Additional Resources
  - Paper with Detailed Design Examples
  - Spread Sheet

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High Strength Bolts

- New Specification Combines 4 Specifications into 1 for both buildings and bridges-**F3125** (published 1/15)
  - A325 Standard Hex Bolt
  - F1852 (A325 Tension Control)
  - A490 Standard Hex Bolt
  - F2280 (A490 Tension Control)
  - + Metric
- These old names become Grades

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Significant Changes

- Grade A325- F_u = 120 ksi for all diameters (results in an increase in shear capacity and pretension for bolts > 1 in.)
- Annex A1- Table gives permitted coatings and over tapping required for nuts
  - No hot dip or mechanical galvanizing of Grade A490 bolts
  - F1136 and F2833 Zinc/Aluminum Allowed on **all** Grades A325 and A490
- Rotational Capacity Test in Appendix A2

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AASHTO Specifications

- Updated to New Specification
  - Pretension Increased for A325 Bolts>1 in.
  - Bolts Referred to as ASTM F3125 Grade A325, etc.
- Standard hole size for bolts 1 inch in diameter or larger increased to bolt diameter+1/8 in.
  - Provides clearance for longitudinal seam developed during hot forging of bolt
  - Smaller diameter bolts typically cold forged

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AASHTO LRFD Changes

- Bolt Shear Strength
- Slip Critical Categories
- Standard Hole Sizes
- Girder Field Splice Design

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Shear Strength

AASHTO 6.13.2.7

- Initial Length Reduction
  - Changed from 0.8 to 0.9
  - Long Joint from 50 to 38 in.
- Bolts with threads in the shear plane: (web bolts)
  - $\emptyset_s R_n = \emptyset 0.45\ A_b F_u$ *(old value 0.38)*
- Bolts with threads excluded from the shear plane:
  - $\emptyset_s R_n = \emptyset 0.56\ A_b F_u$ *(old value 0.48)*
- The nominal shear resistance of a bolt in lap tension connections greater than 38 in. in length shall be taken as 0.83 times the values above.
- $\emptyset_s$=0.80

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Slip Critical Connections

Class	Typical Surface	Slip Coefficient	
		Old Specification	New Specification
A	Mill Scale	0.33	0.30
B	Zinc Rich Paint, Metalized, and Blasted	0.50	0.50
C	Galvanized	0.33	0.30*
D	Organic Zinc Rich	-	0.45

*Do not wire brush the surface
Required tension for A325 > 1 in. diameter increased

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Bolt Design Values

	Diameter (in.)	0.625	0.75	0.875	1	1.125	1.25	1.375
	A_b (in^2)	0.307	0.442	0.601	0.785	0.994	1.227	1.485
	A325 Bolt							
	F_{ub} (ksi)	120	120	120	120	120	120	120
	F_{ub} A_b (kip)	36.8	53.0	72.2	94.2	119.3	147.3	178.2
(installation tension)	P_t (kip)	19	28	39	51	56	71	85
	Type	$\phi_s R_n$ (kip)						
(friction)	A325F	9.5	14.0	19.5	25.5	28.0	35.5	42.5
(threads in shear plane)	A325N	13.3	19.1	26.0	33.9	42.9	53.0	64.1
(threads excluded)	A325X	16.5	23.8	32.3	42.2	53.4	66.0	79.8
	A490 Bolt							
	F_{ub} (ksi)	150	150	150	150	150	150	150
	F_{ub} A_b (kip)	46.0	66.3	90.2	117.8	149.1	184.1	222.7
	P_t (kip)	24	35	49	64	80	102	121
	Type	$\phi_s R_n$ (kip)						
	A490F	12.0	17.5	24.5	32.0	40.0	51.0	60.5
	A490N	16.6	23.9	32.5	42.4	53.7	66.3	80.2
	A490X	20.6	29.7	40.4	52.8	66.8	82.5	99.8

Joints less than 38 in. long and Class B faying surfaces

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Footnote on Bolting

- New Hole Size
  - 1 inch and greater: Standard hole = diameter of fastener +1/8 in.
- Misdrilled holes- fill with fully tensioned high strength bolt (Category B fatigue strength)
- New electric wrenches can be programmed for required turn of the nut

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Expensive and Slow to Erect Field Splice

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The Problem: Tub Girder Splice

Field Splice
36 each top flange
80 bolts in each web
85 bolts bottom flange
634 bolts
1,902 holes

Bolts: 634x$20= $12,680
Labor: 634x10 min= 106 field hours each splice

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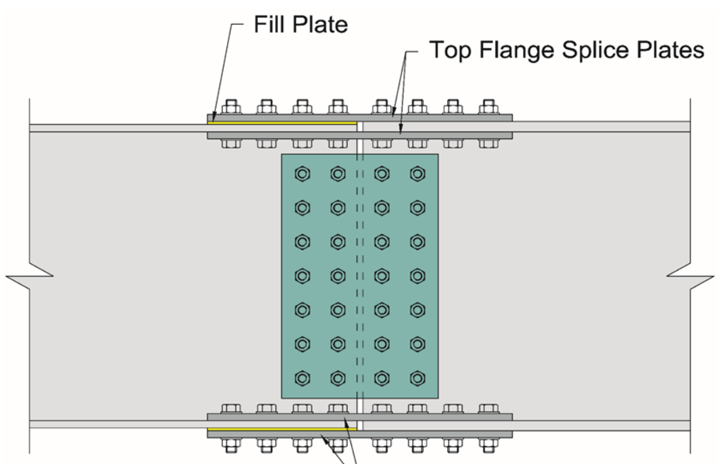
New Bolted Field Splice Design Method



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Outline of Method

- Design connection to develop yield strength of the flange
- Design the web connection to carry shear capacity of the web plus moment not resisted by flanges
- Check moment capacity of connection using flange yield strength
- If flanges are not sufficient to develop the moment- the needed additional moment is added to the design of the web connection

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Splice Design Procedure- Strength Limit State

1. Design Flange Connection to Develop the Smallest Design Yield Resistance of the Connected Flanges

Design Yield Resistance: $P_{fy} = F_{yf} A_e$

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yf}} \right) A_n \leq A_g$$

ϕ_u = 0.80 resistance factor for fracture of tension members Article 6.5.4.2
ϕ_y = 0.95 resistance factor for yielding of tension members Article 6.5.4.2
A_n = net area of the flange Article 6.8.3 (in.^{2})
A_g = gross area of the flange (in.^{2})
F_{yf} = yield strength of the flange Table 6.4.1-1 (ksi)
F_u = tensile strength of the flange Table 6.4.1-1 (ksi)

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Effective Net Area Coefficient

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yf}}\right) A_n = 0.84\left(\frac{F_u}{F_{yf}}\right) A_n \le A_g$$

ASTM A709 Grade	F_y	F_u	$0.84\left(\frac{F_u}{F_y}\right)$
36	36	58	1.35
50	50	65	1.09
50W and HPS 50W	50	70	1.18
HPS 70W	70	85	1.02
HPS 100W	100	110	0.92

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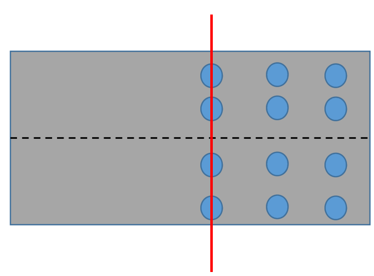
Trial Flange Net Section
Assuming 4 holes in a row across flange

$$A_n = t_f\left[b_f - 4d_h\right]$$



where:
t_f = flange thickness (in.)
b_f = flange width (in.)
d_h = diameter of standard-size bolt hole Table 6.13.2.4.2-1 (in.)

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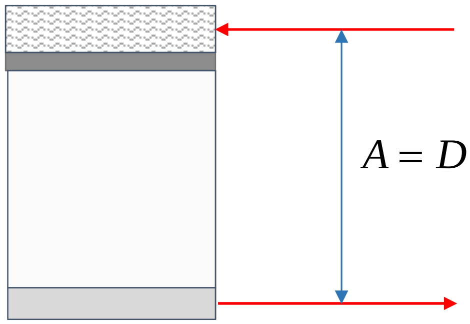
Positive Moment Capacity

(Composite Girder)

Bottom Flange in Tension



$$A = D + \frac{t_{ft}}{2} + t_{haunch} + t_{fc} + \frac{t_s}{2}$$

$$P_{fy} = F_{yf} A_e$$

Moment Capacity:

P_{fy} for the Bottom Flange x Moment Arm to Mid- Depth of Deck

= (F_{yf} x A_e) x A

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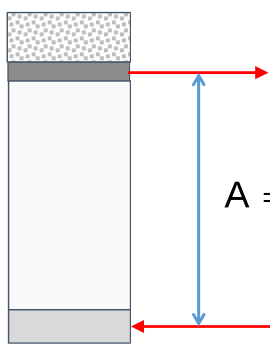
Negative Moment Capacity Check

Ignore Tensile Strength of Reinforcement



$$P_{fy}(\text{top}) = F_{yf} A_e$$

$$A = D + \frac{t_{ft}}{2} + \frac{t_{fc}}{2}$$

$$P_{fy}(\text{bot.}) = F_{yf} A_e$$

Moment Capacity=

Smallest Value of P_{fy} x Distance Between Flange Centroids

(F_{yf} x A_e) x A

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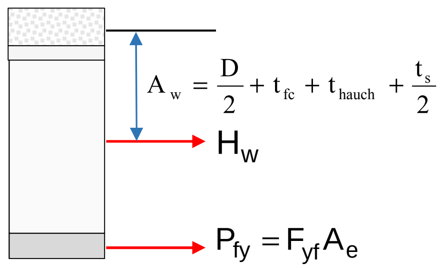
Calculation of Moment Carried by Web

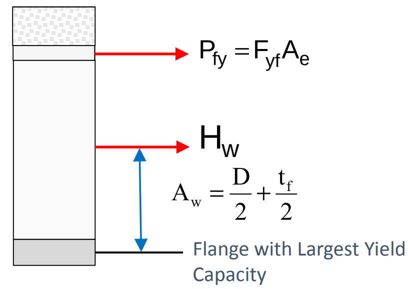
Web Moment = Factored Design Moment – Flange Moment (P_{fy} x A)

Web Moment = H_w x A_w (web force at mid depth)

Additional Web Connection Force: H_w= Web Moment/ A_w



Positive Moment
Take Moments about mid thickness of deck



Negative Moment
Take Moments about larger flange

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Splice Design Procedure- Strength Limit State

2. Design Web Connection to Develop the Smallest Factored Shear Resistance of the Connected Webs

$$V_r = \phi_v V_n$$

ϕ_v = 1.0 the resistance factor for shear
V_n =shear resistance of the web Article 6.10.9 or 6.11.9

Plus additional force H_w to resist design moment

Two rows of bolts minimum on each side of the splice the full depth of the web

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Resultant Web Force Shear and Moment

H_w = (Additional required web moment) / A_w

$$R = \sqrt{(V_r)^2 + (H_w)^2} = \sqrt{(\phi_v V_n)^2 + (H_w)^2}$$

Number Bolts Required = R / Bolt Capacity

Minimum of Two Rows each side of splice

Normally Maximum Spacing and 2 Row Requirement Controls Web Bolts

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Design Example 1

- Bolted field splice in the end span of a bridge with 140-175-140 spans
  - Use 7/8 in. F3125 Grade A325 bolts with standard 15/16 in. holes
- Splice near point of dead load contraflexure
- Design moments:

M_{DC1} = +248 kip-ft
M_{DC2} = +50 kip-ft
M_{DW} = +52 kip-ft
M_{+LL+IM} = +2,469 kip-ft
M_{-LL+IM} = -1,754 kip-ft
$M_{deck\ casting}$ = +1,300 kip-ft

- The factored Strength I design moments at the point of splice are computed as follows:
  - Positive Moment = 1.25(248+50) + 1.5(52) + 1.75(2,469) = **+4,771 kip-ft**
  - Negative Moment = 0.9(248+50) + 0.65(52) + 1.75(-1,754) = **-2,768 kip-ft**

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Girder Cross Section

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	16 [a]	1 [a]	69	½	18 [a]	$1\text{-}^{3}/_{8}$ [a]
Right	18 [b]	1 [b]	69	$^{9}/_{16}$	20 [b]	1 [b]

[a] – Flange is ASTM A709 Grade 50W
[b] – Flange is ASTM A709 Grade HPS 70W

3/8 filler required

Thickness of Composite Deck= 9 in. with no haunch

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Top Flange Splice

- Left side 1x16 in. Grade 50W (smallest area and strength-controls connection design)
  - Effective Net Section assuming 4 bolt holes in a row

$$A_e = 1.18(1)[16\text{-}4(^{15}/_{16})] = 14.4 \text{ in.}^2 < 1(16) \text{ in.}^2 = 16.0 \text{ in.}^2$$

$$P_{fy} = 50(14.4) = \mathbf{720\ kips}$$

- Number of Bolts Required: N = 720/(2x32.3) = 11.1
- Use 3 rows of 4 = 12 bolts on each side of the splice.

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Bottom Flange Splice

- Need to check which side controls splice design
- Left Side

  $A_e = 1.18(1.375)[18\text{-}4(^{15}/_{16})] = 23.1 \text{ in.}^2 < 1.375(18) \text{ in.}^2 = 24.75 \text{ in.}^2$

  $P_{fy} = 50(23.1) = 1{,}152 \text{ kips}$
- Right Side

  $A_e = 1.02(1)[20\text{-}4(^{15}/_{16})] = 16.6 \text{ in.}^2 < 1.0(20) \text{ in.}^2$

  $P_{fy} = 70(16.6) = 1{,}162 \text{ kips} > 1{,}152 \text{ kips}$

  (therefore the smaller strength left side controls)

  $P_{fy} = 1{,}152$ kips

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Bottom Flange Splice Filler Plate Shear Reduction

- Due to change in flange thickness a 3/8 in. filler is required on the right side which reduces the shear capacity of the bolt. The reduction factor in AASHTO is:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right]$$

AASHTO Eq. 6.13.6.1.4-1

$$\gamma = A_f/A_p$$

  - A_f =sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})
  - A_p=smaller of either the connected plate area on the side of the connection with the filler or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

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Filler Plate Shear Reduction

- Normally the filler and flange have the same width and the splice plate areas will be larger than flange area
- The ratio, $\gamma = A_f/A_p$, can be written in terms of the fill plate and flange thickness

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right] = \left[\frac{1+\frac{t_{filler}}{t_{flange}}}{1+\frac{2xt_{filler}}{t_{flange}}}\right]$$

$$R = \frac{1+\frac{0.375}{1}}{1+\frac{2(0.375)}{1}} = 0.79$$

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Bottom Flange Splice Design

P_{fy} = 1,152 kips

- Number of Bolts Required: N = 1,152/(2x0.79x32.3) = 22.6
- Use 6 rows of 4 = 24 bolts on each side of the splice.

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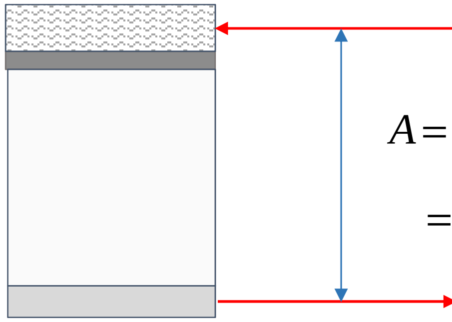
Positive Moment Capacity

(Composite Girder)

Bottom Flange in Tension



$$A = D + \frac{t_{ft}}{2} + t_{haunch} + t_{fc} + \frac{t_s}{2}$$

$$= 69 + 1.375/2 + 1 + 9/2 = 75.2\,\text{in.}$$

$$P_{fy} = F_{yf} A_e = 1{,}152\ \text{kips}$$

M_{flange} = 1,152 x (75.2/12) = 7,218 kip-ft > 4,771 kip-ft ok

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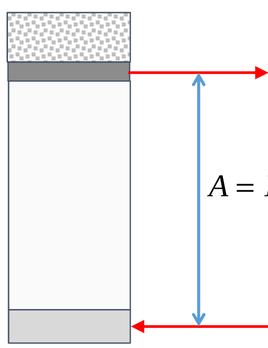
Negative Moment Capacity Check

Ignore Tensile Strength of Reinforcement



$$P_{fy}\,(\text{top}) = F_{yf} A_e = 720\ \text{kips}$$

$$A = D + \frac{t_{ft}}{2} + \frac{t_{fc}}{2} = 69 + (1.375 + 1)/2 = 70.2\ \text{in.}$$

$$P_{fy}\,(\text{bot.}) = F_{yf} A_e = 1{,}152\ \text{kips}$$

M_{flange} = 720 x (70.2/12) = 4,211 kip-ft > |-2,768 kip-ft| ok

Moment Capacity of Flanges are Sufficient for both positive and negative Strength I design moments

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Design Web Bolts

(Design for Shear Only, Flanges Can Develop Required Moment Capacity)

- The factored shear resistance, V_r, of the smaller ½" x 69" Grade 50W web on the left side of the splice with a transverse-stiffener spacing of 3 times the web depth is from AASHTO Article 6.10.9:

$$V_r = \phi_v V_n = 468 \text{ kips}$$

- The number of bolts with threads in the shear plane of each side of splice required to develop the shear strength is:

N=468/(2x26.0)=9.0 bolts

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Web Splice Bolt Spacing

- AASHTO requires two rows of bolts on each side of splice and a full depth web connection.
  - The spacing and number of web bolts will be controlled by these requirements.
  - Spacing $s \leq (4.0 + 4.0t) \leq 7.0$ in., where t is the thickness of the splice plate (AASHTO 6.13.2.6.2)
  - Assuming the splice plate is ½ the web plate thickness plus 1/16 in. equaling 5/16 in.

  The maximum vertical bolt spacing is:

  s ≤ 4.0+4x5/16=5.25 in.

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Web Bolt Spacing

- Using a 3 in. clearance top and bottom to provide clearance for bolt installation, the available web depth is:

  69-(2x3)=63 in.

  Using a the maximum spacing of 5.25 in. the number of bolts required based on spacing requirements is:

  1+63/5.25=<u>13 bolts</u> in two vertical rows each side of web

  Total Number of bolts each side of web splice =26>9 exceeds design requirements

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Check of Flange Slip Resistance at Service II Moment

- Service II Positive Moment = 1.0(+248+50) + 1.0(+52) + 1.3(+2,469) = +3,560 kip-ft
- Assuming Class B slip coefficient

R_n = 19.5 kip

Connection Slip Capacity:

P_t=24 (2x19.5 kips/bolt) = 936 kips

Moment Arm centroid of lower flange to centroid of deck (same as strength calculation):

A=75.2 in.

Flange Slip Moment:

M_{flange} = 936 x (75.2/12) = 5,866 kip-ft > 3,560 kip-ft ok

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Check of Service II Negative Moment

- Service II Negative Moment = 1.0(+248+50) + 1.0(+52) + 1.3(-1,754) = -1,930 kip-ft
- Use flange with smallest number of bolts to calculate flange slip capacity

  Top Flange (12 bolts): P_t=12 (2x19.5 kips/bolt) = 468 kips

Moment Arm distance between centroids of flanges (same as strength calculation): A=70.2 in.

- M_{flange} = 468 x (70.2/12) = 2,738 kip-ft > |-1,930|kip-ft ok

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Check of Slip Resistance Deck Casting

- $M_{deck\ casting}$ = 1.4(+1,300) = +1,820 kip-ft
- Non Composite Section
  - Use smallest flange slip capacity

    Top Flange (12 bolts): P_t=12 (2x19.5 kips/bolt) = 468 kips
  - Distance between flange centroids

    A=70.2 in.

    M_{flange} = 468 x (70.2/12) = 2,738 kip-ft > |1,820 | kip-ft ok

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Check of Web Service II and Deck Casting

- Slip Resistance of Web Connection (26 bolts each side of connection):

V_t=26(2x19.5 kips/bolt) = 1,014 kips

Shear Capacity of Web:

$V_r = \phi_v V_n$ = 468 kips < slip capacity = 1,014 kips

Therefore slip will not occur under any limit state since slip capacity of web bolts exceed the shear capacity of the web.

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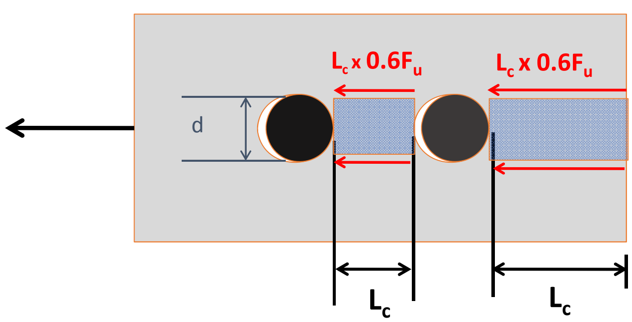
Bearing Strength of Connected Material Standard Holes



if $L_c \geq 2d$

$R_n = 2.4\,d\,t\,F_u$

for $L_c < 2d$

$R_n = 1.2\,L_c\,t\,F_u$

Φ_{bb}=0.80

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Bearing Strength Versus Hole Spacing
F_u= 65ksi

Diameter (in.)	0.625	0.75	0.875	1	1.125	1.25	1.375
Typical Spacing (in.)	2	2.5	3	3	3.5	3.75	4.25
Clear Distance (in.)	1.313	1.688	2.063	1.938	2.313	2.438	2.813
$f_{bb}R_n$ (kip/in. of thickness)	78	93.6	109.2	120.9	140.4	152.1	171.6
2xA325X/Bearing (in.)	**0.42**	**0.51**	**0.59**	**0.70**	**0.76**	**0.87**	**0.93**
2xA490X/Bearing (in.)	**0.53**	**0.63**	**0.74**	**0.87**	**0.95**	**1.08**	**1.16**
Typical End Dist. (in.)	1	1.25	1.5	1.625	1.75	2	2.25
Clear Distance (in.)	0.66	0.84	1.03	1.09	1.16	1.34	1.53
$f_{bb}R_n$ (kip/in. of thickness)	40.95	52.65	64.35	68.25	72.15	83.85	95.55
2xA325X/Bearing (in.)	**0.81**	**0.90**	**1.00**	**1.24**	**1.48**	**1.57**	**1.67**
2xA490X/Bearing (in.)	**1.01**	**1.13**	**1.26**	**1.55**	**1.85**	**1.97**	**2.09**
	Hole Size= Diameter+1/16 in.			Hole Size= Diameter+1/8 in			

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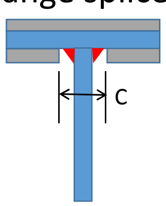
Splice Plate Design Guides

- Splice Plate Thickness:

$$t_{splice} = \frac{t_{flange\ or\ web}}{2} + \frac{1}{16}$$

Flange or web will control bearing and block shear.

- Inside flange splice plate



$$C \geq t_{web} + 2\left[weld\ size + \frac{1}{8}\right]$$

$$0.9 \leq \frac{\sum \text{Area}_{Inner\ splice\ plates}}{Area_{outer\ splice\ plate}} \leq 1.10$$

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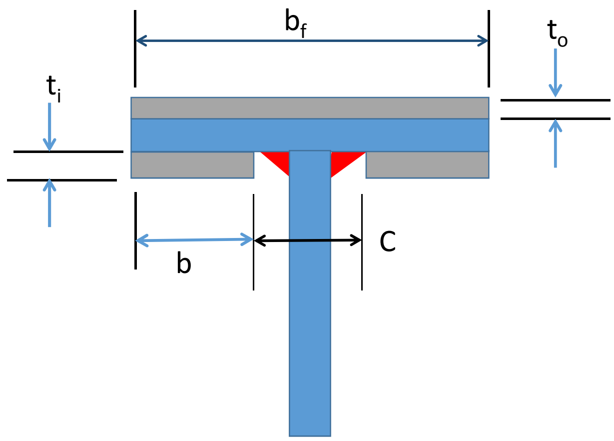
Proportioning Splice to Provide Double Shear Flange Connection



10% Rule

$$0.9\, b_f t_o \le 2bt_i \le 1.1\, b_f t_o$$

Substituting

$$b = \frac{(b_f - C)}{2}$$

Yields

$$0.9t_o \le \left(1 - \frac{c}{b_f}\right) t_i \le 1.1t_o$$

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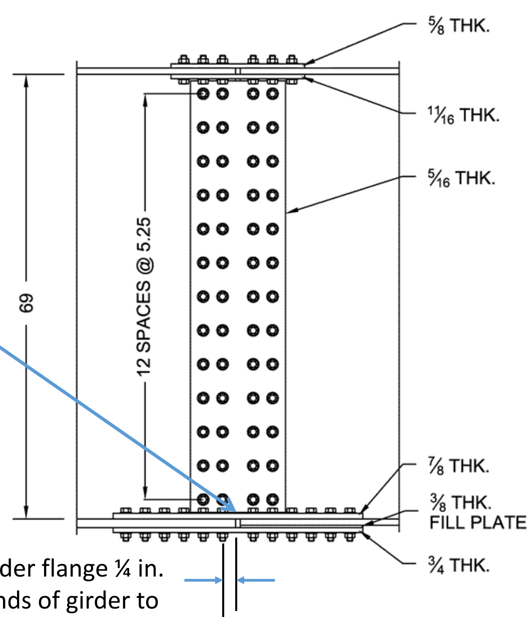
Girder Gap and End Distance

½ in. gap between girder to provide drainage and fit up



Increase end distance on girder flange ¼ in. from design value at both ends of girder to allow for girder trim (top and bottom)

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Summary of Design

- Flanges Adequate to Carry Strength I and Service II Design Moments
- Web Connection Controlled by Maximum Bolts Spacing, Two Row Requirement, and Full Depth Connection
- Need to Check Bolt Bearing, Net Section, and Block Shear on Flanges and Splice Plates
  - Note we have assumed the bolts are in double shear- This assumption requires sum of inside flange splice plates and outer splice plate areas are within 10%. If do not meet this requirement you need to proportion shear on each plane in accordance with ratio of the area individual splice plate to the total splice plate area. This is not required for checking slip only for strength limit states.

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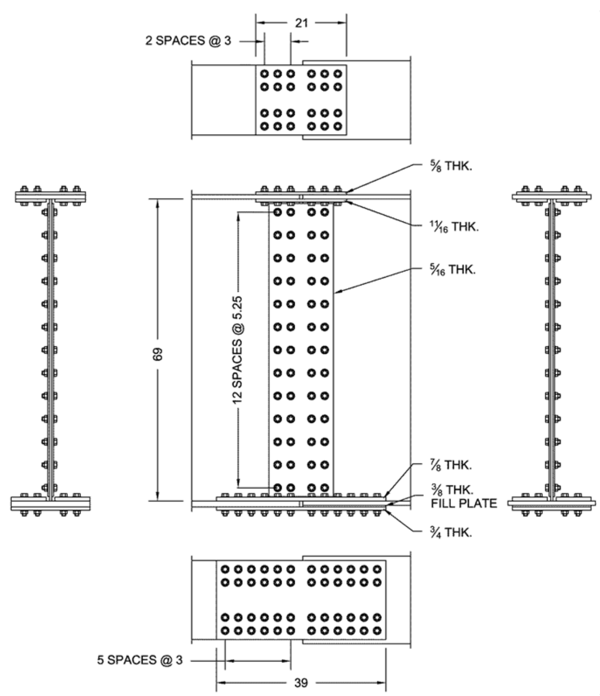
Connection Design

top flange: 24
web: 52
bot. flange: 48
Total:124 bolts



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Design Example 2: Deep Girder

- Bolted field splice in the end span of a bridge with 234-300-234 spans
- Use 7/8 in. F3125 Grade A325 bolts with standard 15/16 in. holes
- Splice near point of dead load contraflexure
- Design moments

M_{DC1} = -1,564 kip-ft
M_{DC2} = -242 kip-ft
M_{DW} = -315 kip-ft
M_{+LL+IM} = +5,627 kip-ft
M_{-LL+IM} = -7,117 kip-ft
$M_{deck\ casting}$ = +3,006 kip-ft

- The factored Strength I design moments at the point of splice are computed as follows:
  - Positive Moment = 0.9(-1,564 + -242) + 0.65(-315) + 1.75(+5,627) = **+8,017** kip-ft
  - Negative Moment = 1.25(-1,564 + -242) + 1.5(-315) + 1.75(-7,117) = **-15,185** kip-ft

Girder Cross Section (all material A709 Grade 50)

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	19	1	109	0.75	19	1
Right	22	2	109	0.75	22	2

- The deck thickness is 8.0 and the haunch height is ignored in this example.
- By inspection, the left side has the smallest flanges which will control the design.
- Also, 1-inch fillers are required for both top and bottom flange splices.

Flange Splice Design

- Left Side Controls- Top and Bottom Flange are Identical
- Flange Effective Area, 4 bolts in row across flange

A_e = 1.09(1)[19-4($^{15}/_{16}$)] = 16.6 in.^{2} < 1(19) in.^{2} = 19.0 in.^{2}

P_{fy} = 50(16.6) = 830 kips

Reduction in Shear Capacity Due to 1 in. filler:

$$R = \frac{1+\frac{1}{1}}{1+\frac{2(1)}{1}} = 0.67$$

Number of Bolts Required: N = 830/(0.67x2x32.3) = 19.2

Use 5 rows of 4 = 20 bolts on each side of the splice in both top and bottom flanges.

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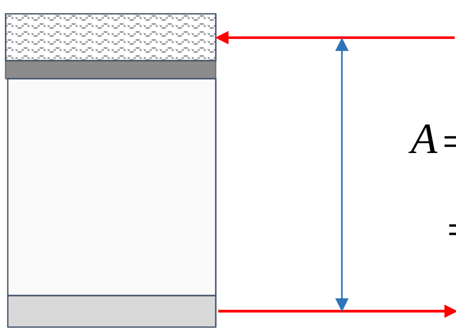
Positive Moment Capacity

(Composite Girder)

Bottom Flange in Tension



$$A = D + \frac{t_{ft}}{2} + t_{haunch} + t_{fc} + \frac{t_s}{2}$$

$$= 109 + 1/2 + 1 + 8/2 = 114.5\text{in.}$$

$$P_{fy} = F_{yf} A_e = 830\text{ kips}$$

M_{flange} = 830 x (114.5/12) = 7,948 kip-ft < 8,017 kip-ft

Difference=8,017-7,948= 69 kip-ft must be carried by the web

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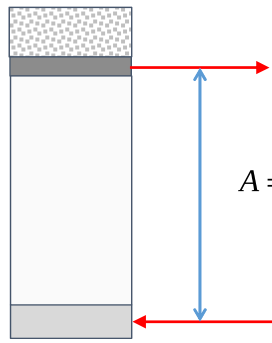
Negative Moment Capacity Check

Ignore Tensile Strength of Reinforcement



$$P_{fy}\,(top) = F_{yf}A_e = 830 \text{ kips}$$

$$A = D + \frac{t_{ft}}{2} + \frac{t_{fc}}{2} = 109 + (1+1)/2 = 110 \text{ in.}$$

$$P_{fy}\,(bot.) = F_{yf}A_e = 830 \text{ kips}$$

M_{flange} = 830 x (110/12) = 7,608 kip-ft < |-15,185 kip-ft|

Difference=15,185-7,608= 7,577 kip-ft must be carried by the web

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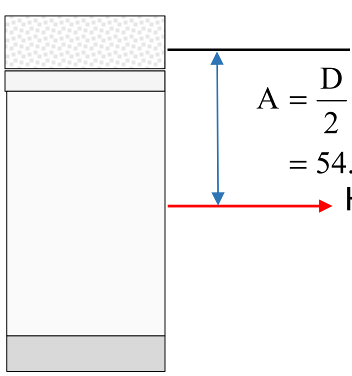
Calculation of Moment Carried by Web

Positive Moment



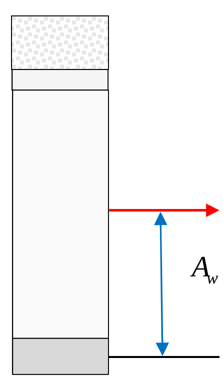
$$A = \frac{D}{2} + t_{fc} + \frac{t_s}{2}$$

$$= 54.5 + +1 + 4 = 59.5 \text{ in.}$$

H_w

$$H_w = \frac{69x12}{59.5} = 13.9 \text{ kips}$$

Negative Moment



H_w

$$A_w = \frac{D}{2} + \frac{t_f}{2}$$

$$= 54.5 + 0.5 = 55 \text{in.}$$

$$H_w = \frac{7{,}577x12}{55.0} = 1{,}653 \text{ kips}$$ **Controls**

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Design Web Bolts

(Design for Shear Strength Plus H_w)

- The factored shear resistance, V_r, of the smaller 3/4" x 109" Grade 50 web with a **transverse-stiffener** spacing of 2 times the web depth is (AASHTO Article 6.10.9):

$$V_r = \phi_v V_n = 1{,}312 \text{ kips}$$

- The resultant force on the web connection is:

$$R = \sqrt{(V_r)^2 + (H_w)^2} = \sqrt{(1{,}312)^2 + (1{,}653)^2} = 2{,}110 \ kips$$

- The number of bolts with threads in the shear plane of each side of splice required to develop the shear strength is:

N=2,110/(2x26.0)=40.6 bolts

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Geometry of Web Splice Bolt Group

- Assuming the splice plate is ½ the web plate thickness plus 1/16 in. equaling 7/16 in.

  The maximum vertical bolt spacing is:

  s ≤ 4.0+4x7/16=5.75 in.

- Try 5-1/8" spacing with 3-1/4" gap at top and bottom
  - Web depth available for bolts=109-2x3.25=102.5
  - Number of bolts at 5-1/8in spacing:

  $N_b = 1 + \frac{102.5}{5.125} = 21$ bolts in two vertical rows each side of the splice

  42 bolts total> 40.7 OK

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Check of Slip Resistance at Service II

- Service II Positive Moment = 1.0(-1,564 + -242) + 1.0(-315) + 1.3(+5,627) = +5,194 kip-ft
- Assuming Class B slip coefficient

R_n = 19.5 kip

Connection Slip Capacity:

P_t=20 (2x19.5 kips/bolt) = 780 kips

Moment Arm centroid of lower flange to centroid of deck (same as strength calculation):

A=114.5 in.

Flange Slip Moment:

M_{flange} = 780 x (114.5/12) = 7,442 kip-ft > 5,194 kip-ft ok

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Check of Service II Negative Moment

- Service II Negative Moment = 1.0(-1,564 + -242) + 1.0(-315) + 1.3(-7,117) = -11,373 kip-ft
- Use flange with smallest number of bolts (equal in this section) to calculate flange slip capacity

Top Flange (12 bolts): P_t=12 (2x19.5 kips/bolt) = 780 kips

Moment Arm distance between centroids of flanges (same as strength calculation): A=110 in.

M_{flange} = 780 x (110/12) = 7,150 kip-ft < |-11,373|kip-ft

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Web Force for Service II Slip Resistance

$$H_W = \frac{\text{Service II Moment} - \text{Flange Moment}}{\frac{D_W}{2} + \frac{t_f}{2}}$$

$$= \frac{(|-11{,}373| - 7{,}150) \times 12}{54.5 + 0.5} = 921 \text{ kips}$$

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Check of Slip Resistance Deck Casting

- $M_{deck\ casting}$ = 1.4(+3,006) = +4,208 kip-ft
- Non Composite Section
  - Use smallest flange slip capacity

    Top Flange (12 bolts): P_t=12 (2x19.5 kips/bolt) = 780 kips

  Distance between flange centroids

    A=110 in.

    M_{flange} = 780 x (110/12) = 7,150 kip-ft > 4,208 kip-ft ok

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Check of web- Service II and Deck Casting

Unfactored shears at splice

- V_{DC1} =-147 kips
- V_{DC2} =-28 kips
- V_{DW} =-37 kips
- V_{+LL+IM} = +19 kips
- V_{-LL+IM} =-126 kips
- $V_{deck\ casting}$= -79 kips

By inspection service II negative greater than positive shear controls:

$$= 1.0(-147 + -28) + 1.0(-37) + 1.3(-126) = -376 \text{ kips}$$

Deck casting=1.4x -79=-111 kips <-376 does not control

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Check of Slip of Web Connection Shear and Moment Service II

- Slip Resistance of Web Connection (42 bolts each side of connection):

$$V_t=42(2x19.5 \text{ kips/bolt}) = 1,638 \text{ kips}$$

- Web Resultant Service II Negative Moment and Negative Shear

$$R = \sqrt{(-376)^2 + (921)^2} = 995 \text{ kips} < 1,638 \text{ kips OK}$$

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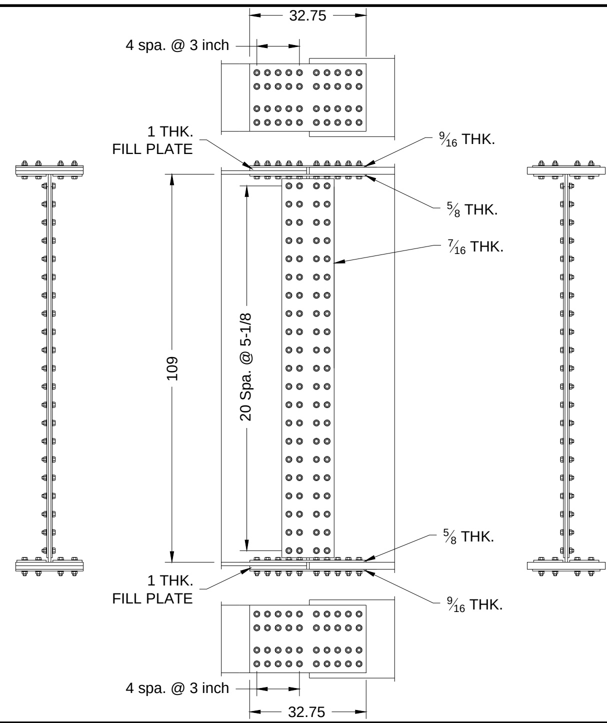
Connection Design

top flange: 40
web: 84
bot. flange: 40
Total:164 bolts



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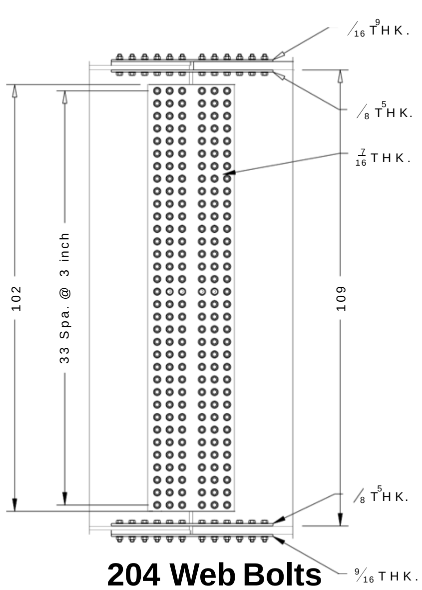
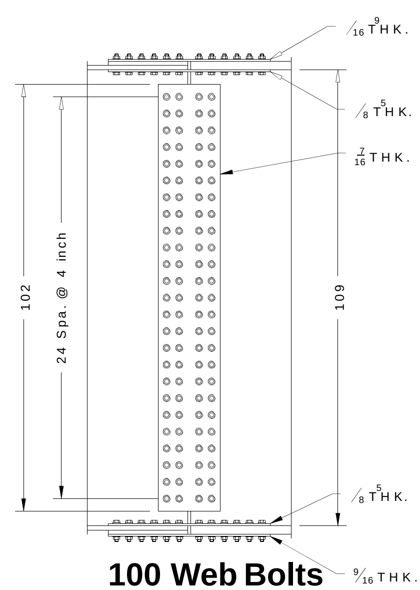
Validation Finite Element Analysis



204 Web Bolts



100 Web Bolts

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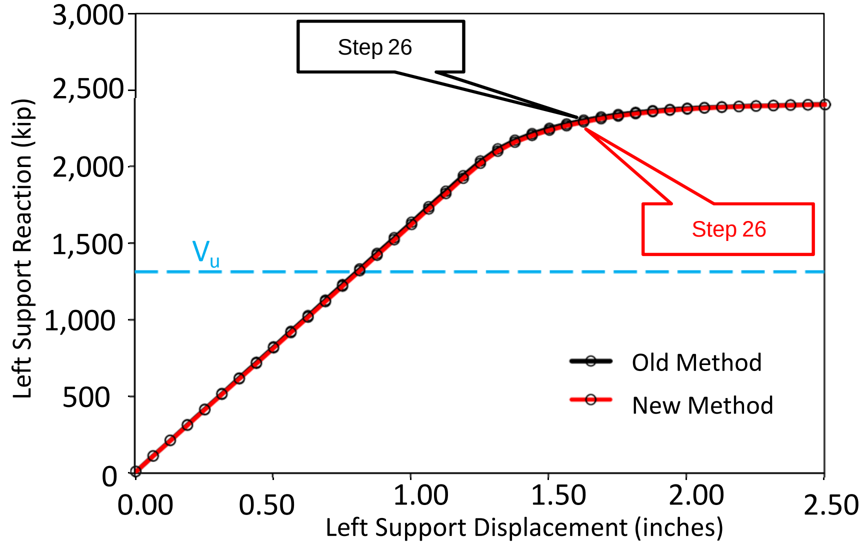
Results – High Shear



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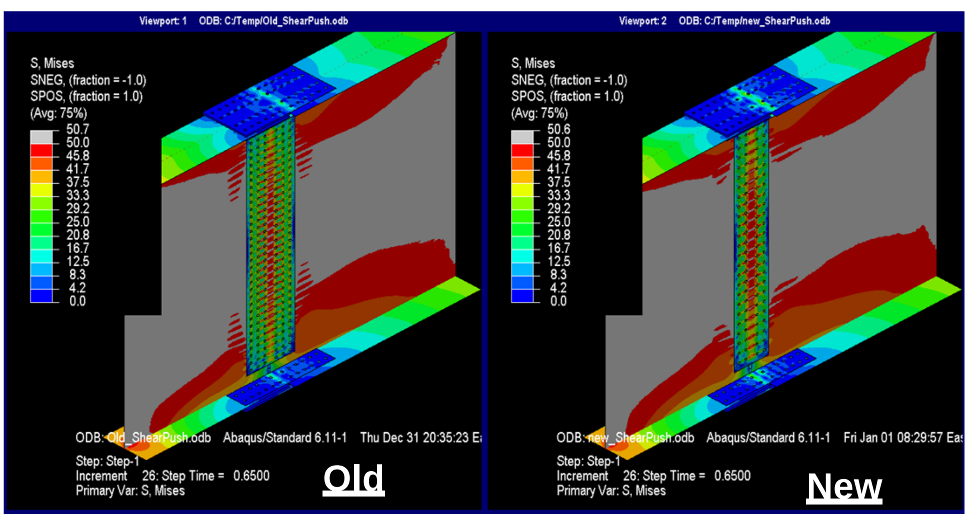
Results – High Shear

Von Mises Stresses @ Step 26



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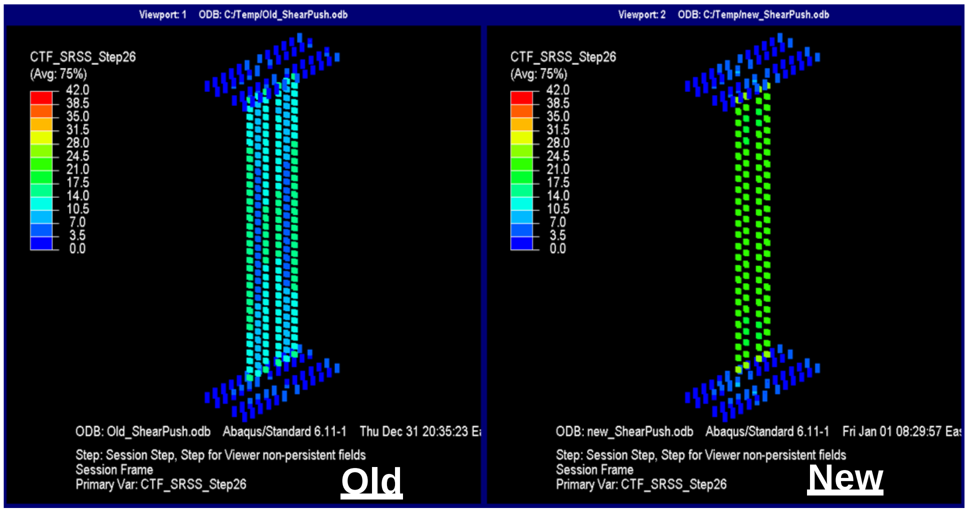
Results – High Shear

Bolt Shear Forces @ Step 26



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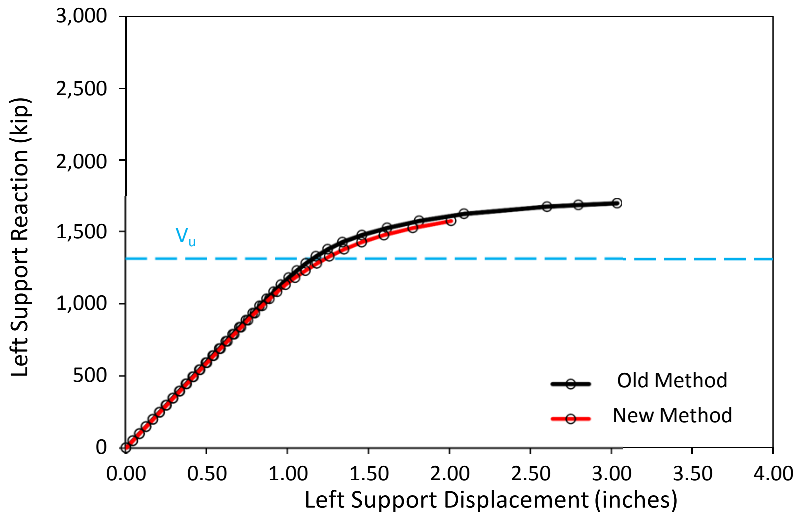
Proportional Negative Moment & Shear



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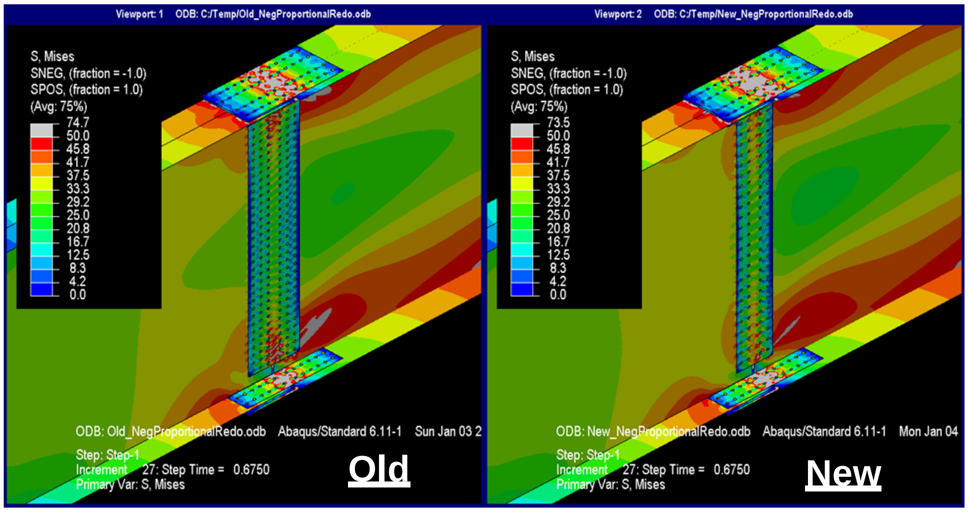
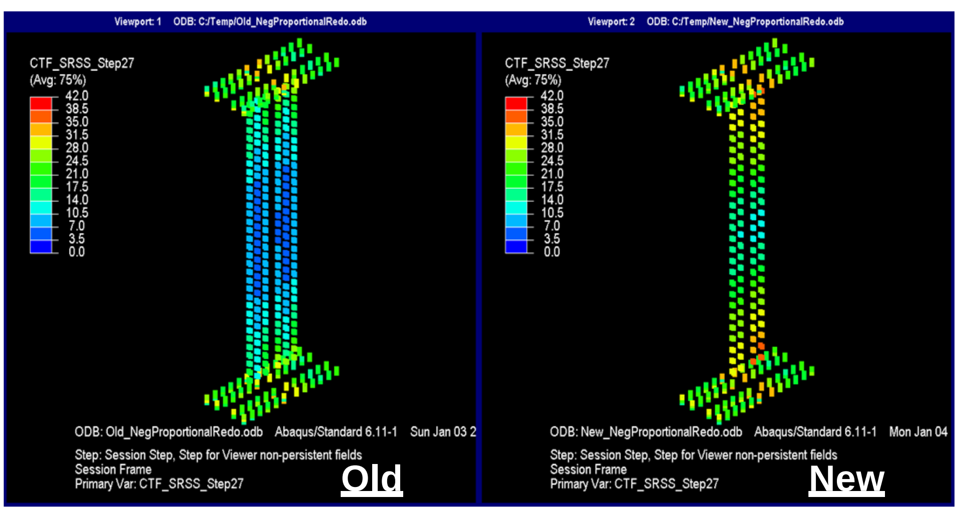
Proportional Negative Moment & Shear

Von Mises Stresses @ V_u



Proportional Negative Moment & Shear

Bolt Shear Forces @ V_u



Design Example 3 Curved Trapezoidal Tub Girder

- Bending strength design only
- Demonstrate inclusion of St. Venant torsion force in lower flange splice design
- Web design same as plate girders- no need to consider torsion forces in web since the web connection is designed for webs full design strength

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Design Example 3: Curved Trapezoidal Tub Girder

- Bolted field splice in a curved girder with spans of 150-200-150 feet with a radius of 550 feet
  - Use 7/8 in. F3125 Grade A325 bolts with standard 15/16 in. holes
- Design moments

M_{DC1} = +2,417 k-ft	T_{DC1}=-252 k-ft
M_{DC2} = +251 k-ft	T_{DC2}=-51 k-ft
M_{DW}= +339 k-ft	T_{DW}=-39 k-ft
M_{+LL+IM} = +5,066 k-ft	T_{+LL+IM}=+309 k-ft
M_{-LL+IM} = -2,926 k-ft	T_{-LL+IM} =-537 k-ft
$M_{deck\ casting}$= +4,082 kip-ft	$T_{deck\ casting}$=-217 k-ft

- The factored Strength I design moments at the point of splice are computed as follows:

Positive Moment = 1.25(2,417 + 251) + 1.5(339) + 1.75(5,066) = **+12,709** k-ft

Negative Moment = 0.90(2,417 + 251) + 0.65(339) + 1.75(-2,926) = **-2,499** k-ft

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Tub Girder Dimensions

A709 Grade 50W Steel
9.5 in. deck with a 5 in. haunch from the top of the web
effective deck width= 234 in.

Side of Splice	Top Flange Width (in.)	Top Flange Thickness (in.)	Web Depth (in.)	Web Thickness (in.)	Bottom Flange Width (in.)	Bottom Flange Thickness (in.)
Left	18	1	80.39 [a]	0.625	76	0.75
Right	20	1.25	80.39 [a]	0.625	76	1.25

[a] – Web depth measured along the web slope; the vertical web depth is 78.0 in.

By inspection smaller left side controls connection design
Fillers required: ¼ in. filler top flange, ½ in. filler bottom flange

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Flange Strengths

- Bottom Flange
  - Assume 21 bolts in a row across the flange (20 spaces at 3-3/8 in.)
  - $A_e = 1.18(0.75)[76\text{-}21(^{15}/_{16})] = 49.8 \text{ in.}^2 < 0.75(76) \text{ in.}^2 = 57.0 \text{ in.}^2$
  - $P_{fy} = 50(49.8) = \mathbf{2{,}490}$ kips

- Top Flange
  - Assume 4 bolts in row across flange
  - $A_e = 1.18(1)[18\text{-}4(^{15}/_{16})] = 16.8 \text{ in.}^2 < 1(18) \text{ in.}^2 = 18.0 \text{ in.}^2$
  - $P_{fy} = 50(16.8) = \mathbf{840}$ kips

Reduction in Shear Capacity Due to 1/4 in. filler:

$$R = \frac{1 + \frac{0.25}{1}}{1 + \frac{2(0.25)}{1}} = 0.83$$

Number of Bolts Required: N = 840/(0.83x2x32.5) = 15.7
Use 4 rows of 4 = 16 bolts on each side of the splice in both top flanges.

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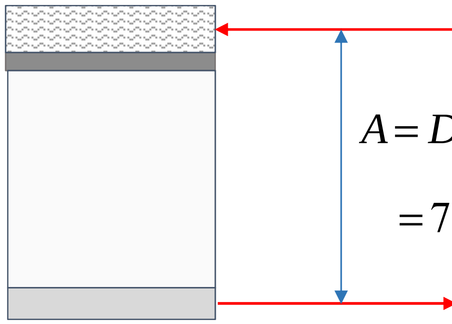
Positive Moment Capacity

(Composite Girder)

Bottom Flange in Tension



$$A = D + \frac{t_{ft}}{2} + t_{haunch} + t_{fc} + \frac{t_s}{2}$$

$$= 78 + 0.75/2 + 5 + 9.5/2 = 88.1\,\text{in.}$$

$$P_{fy} = F_{yf} A_e = 2{,}490\,\text{kips}$$

M_{flange} = 2,490 x (88.1/12) = 18,281 kip-ft > 12,709 kip-ft OK

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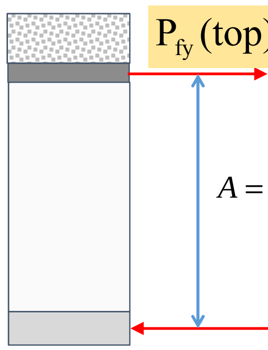
Negative Moment Capacity Check

Ignore Tensile Strength of Reinforcement

$$\mathrm{P_{fy}\,(top)} = 2x\mathrm{F_{yf}A_e} = 2x840 = 1{,}680\ \text{kips}$$



$$A = D + \frac{t_{ft}}{2} + \frac{t_{fc}}{2} = 78 + (0.75 + 1)/2 = 78.9\ \text{in.}$$

$$\mathrm{P_{fy}\,(bot.)} = \mathrm{F_{yf}A_e} = 2{,}490\ \text{kips}$$

M_{flange} = 1,680 x (78.9/12) = 11,046 kip-ft > |-2,499 kip-ft| OK

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Torsional Shear in Lower Flange Strength I Non Composite Section DC1 Load

Shear flow in tub Section:

$$f = \frac{T}{2A_o}$$

$$A_o = \frac{(111+72)}{2} * (78.0 + 0.375 + 0.5) * \frac{1\ ft^2}{144\ in.^2} = 50.1\ ft^2$$

$$f = \frac{1.0(1.25)(-252)}{2(50.1)} = -3.14\ kips/ft$$

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Torsional Shear in Lower Flange Strength I

Torsional Shear Forces on Lower Flange Splice:

Shear flow in tub Section:

$$f = \frac{T}{2A_o}$$

$$A_o = \frac{(111+72)}{2} * (78.0 + 0.375 + 5.0 + \frac{9.5}{2}) * \frac{1\ ft^2}{144\ in.^2} = 56.0\ ft^2$$

$$f = \frac{1.0\left|1.25(-51) + 1.5(-39) + 1.75(-537)\right|}{2(56.0)} = -9.48\ kips/ft$$

$$f_{total} = -3.14 + -9.48 = -12.62\ kips/ft$$

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Combined Flexure and Torsional Connection Design Forces

- Torsional shear force on connection:

$$V_{SV} = f_{total} b_f = \left|-12.62\right| \frac{72.0}{12} = 75.7\ kips$$

- Resultant shear force on flange connection:

$$R = \sqrt{\left(P_{fy}\right)^2 + \left(V_{sv}\right)^2} = \sqrt{\left(2{,}490\right)^2 + \left(75.7\right)^2} = 2{,}491\ kips$$

Connection of Bottom Flange

½ in. filler

- Filler bolt shear reduction:

$$R = \frac{1 + \frac{0.5}{0.75}}{1 + \frac{2(0.5)}{0.75}} = 0.71$$

- Number of bolts required:

  N = 2,491/(0.71x2x32.3) = 54.3

- 3 rows of 21 bolts=63 bolts each side of splice

Summary of Design Method

- Design flange splice for yield capacity of flange
- Calculate positive and negative moment that that can be developed by the flanges
- If flanges have adequate moment capacity- design web for shear strength
- If moment capacity flanges less than required design web for shear strength of web and required moment
- Calculation of lateral bending of flanges not required since designed for yield capacity (similar to a welded splice)
- Bottom flange splice of tub girder must consider St. Venant torsion
- No torsion check of web required since designed splice for shear capacity of the web.

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Results of Design Method

- Fewer bolts in the web
- Slightly more in the flanges
- Simpler direct design method

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Additional Resources

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Design Tool
Plate Girder Splice Design Spreadsheet

NSBA Bolted Splice Designer - Plate Girder

Design Input

Unfactored Loads - Splice Centerline

	Moment (kip-ft)	Shear (kip)
Noncomposite Dead Load (DC_1)	248.00	-82.00
Superimposed Composite Dead Load (DC_2)	50.00	-12.00
Future Wearing Surface (DW)	52.00	-11.00
Positive Live Load plus Impact (LL^+ + I)	2469.00	19.00
Negative Live Load plus Impact (LL^- + I)	-1754.00	-112.00
Deck Casting	1300.00	-82.00

Girder Properties

	Left	Right
Top Flange Material	Grade 50W	HPS Grade 70W
Top Flange Thickness (in)	1	1
Top Flange Width (in)	16	18
Web Material	Grade 50W	Grade 50W
Web Thickness (in)	1/2	9/16
Web Depth (in)	69	
Bottom Flange Material	Grade 50W	HPS Grade 70W
Bottom Flange Thickness (in)	1 3/8	1
Bottom Flange Width (in)	18	20

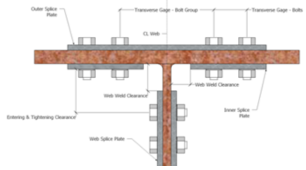
Splice Plate Properties

Bolt Properties

Bolt Type	A325	
Bolt Diameter (in)	7/8	
Web Threads	Included	
Flange Threads	Excluded	
Surface Condition Factor (K_s)	B	
Hole Size Factor (K_h)	Standard	
Top Flange Longitudinal Rows	4	OK
Web Rows - Initial	2	OK
Bottom Flange Longitudinal Rows	4	OK

Concrete Deck Properties

Composite	Composite
Thickness (in)	9
Haunch (in)	0



Coming Soon!

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Complete Splice Details With all Calculations

BOLTED FIELD SPLICES FOR STEEL BRIDGE FLEXURAL MEMBERS
OVERVIEW & DESIGN EXAMPLES

Coming Soon!

Michael A. Grubb, P.E.
M.A. Grubb & Associates, LLC

Karl H. Frank, Ph.D.
Hirschfeld Industries

Justin M. Ocel, Ph.D.
Federal Highway Administration

Paper and Spreadsheet available for download at NSBA Website
www.aisc.org/nsba/design-resources

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Quiz Question

Using the 8th Edition *AASTHO LRFD Bridge Design Specifications*, the web connection in a bolted field splice should be designed to carry (choose all that apply):

a. Shear capacity of the web
b. Shear capacity of the flanges
c. Factored design moment
d. Moment not carried by flanges (factored design moment minus flange moment)

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Questions?

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PDH Certificates

Within 2 business days…

- You will receive an email on how to report attendance from: registration@aisc.org.
- Be on the lookout: Check your spam filter! Check your junk folder!
- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

PDH Certificates

Within 2 business days…

- Reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

There's always a solution in steel.

Thank You

Please give us your feedback!
Survey at conclusion of webinar.

structural STEEL