Steel Bridge Connection Design

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Outline

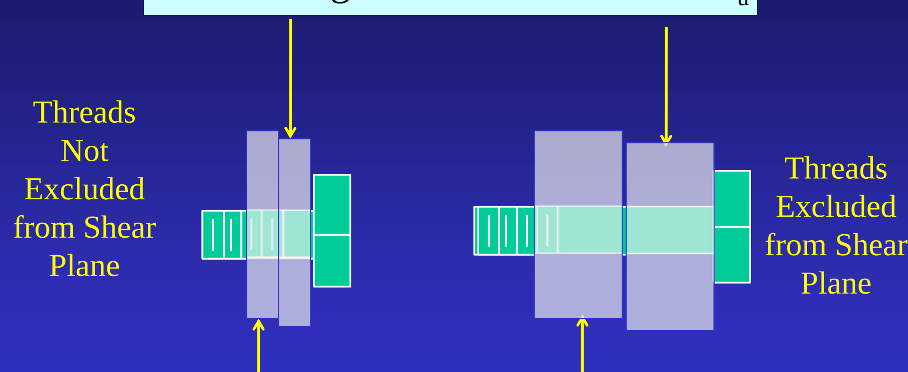
- Connection Limit States-Shear Connections
- Connection Design Forces-Composite Girders
- Design Example

Limit States- Shear Connections

- Bolt Strength
  - Shear
    - Threads Out of the Shear Plane
    - Threads in the Shear Plane
  - Slip
    - Bolt Tension
    - Slip Coefficient
- Connected Material
  - Bolt Bearing
  - Net and Gross Section
  - Block Shear

Bolt Shear Strength

Shear Strength of Material = 0.60 x F_u



Bolt Shear Strength = 0.8 x Shear Area x Shear Strength

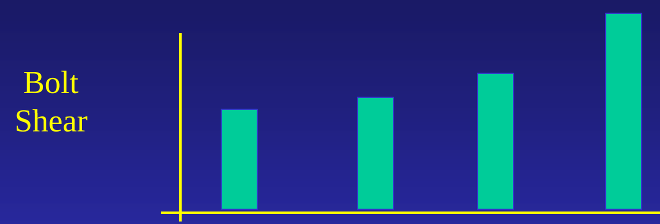
0.80 - Connection Reduction Factor for Joints Up to 50 inches in Length

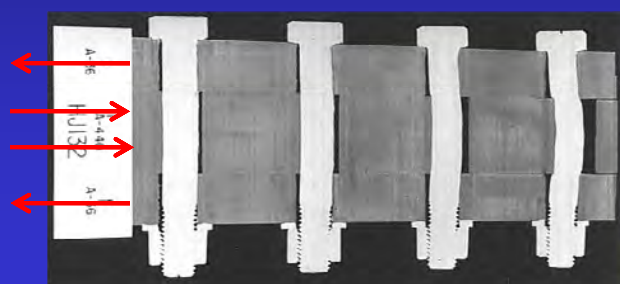
Shear Area :

Threads Exlcuded = A_{bolt}

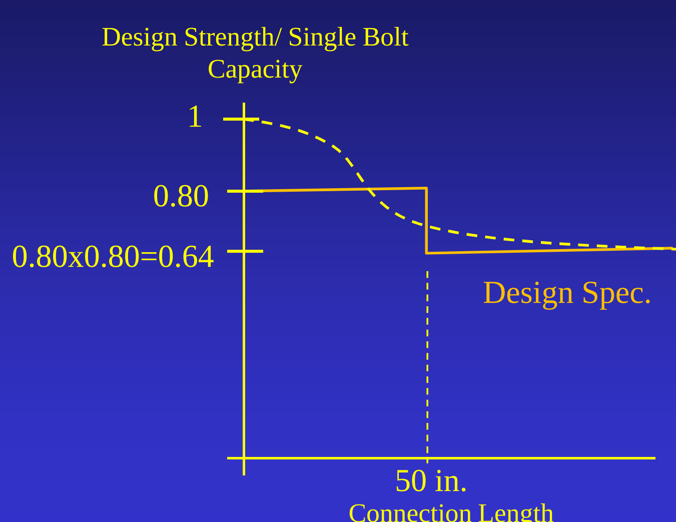
Threads Not Excluded = 0.8 A_{bolt}

Unequal Bolt Shear In Long Joints



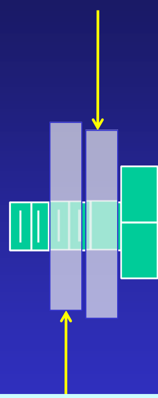


Joint Length Shear Strength Reductions



Bolt Shear Strength

Threads Not Excluded from Shear Plane

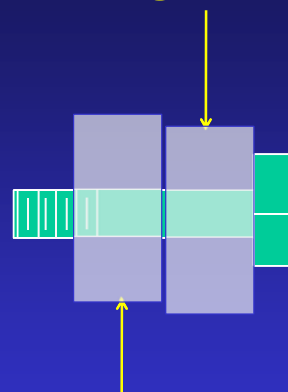


Shear Strength :

$0.8 \times 0.60 \times F_u \times 0.8 \times A_{bolt}$

$0.38\, F_u \times A_{bolt}$

Threads Excluded from Shear Plane



Shear Strength :

$0.8 \times 0.60 \times F_u \times A_{bolt}$

$0.48\, F_u \times A_{bolt}$

Φ_s=0.80

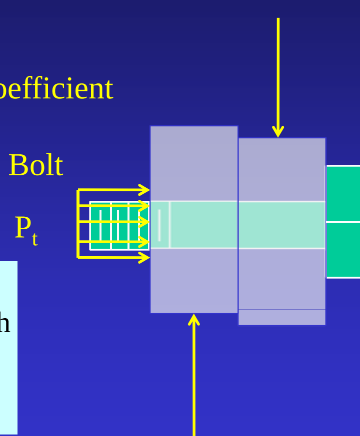
$\text{Slip Capacity} = R_n = K_h\, K_s\, N_s P_t$

Φ_s=1.0 (Art.6.13.2.2)

K_h= Hole Factor
= 1 (normal size holes)

K_s= Surface Condition Slip Coefficient
=0.5(blasted or Zinc Rich)

N_s=Number of Slip Planes per Bolt

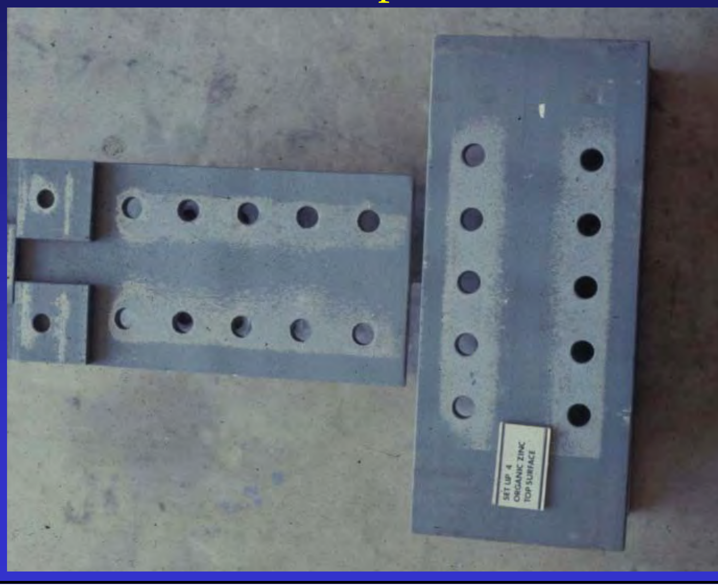


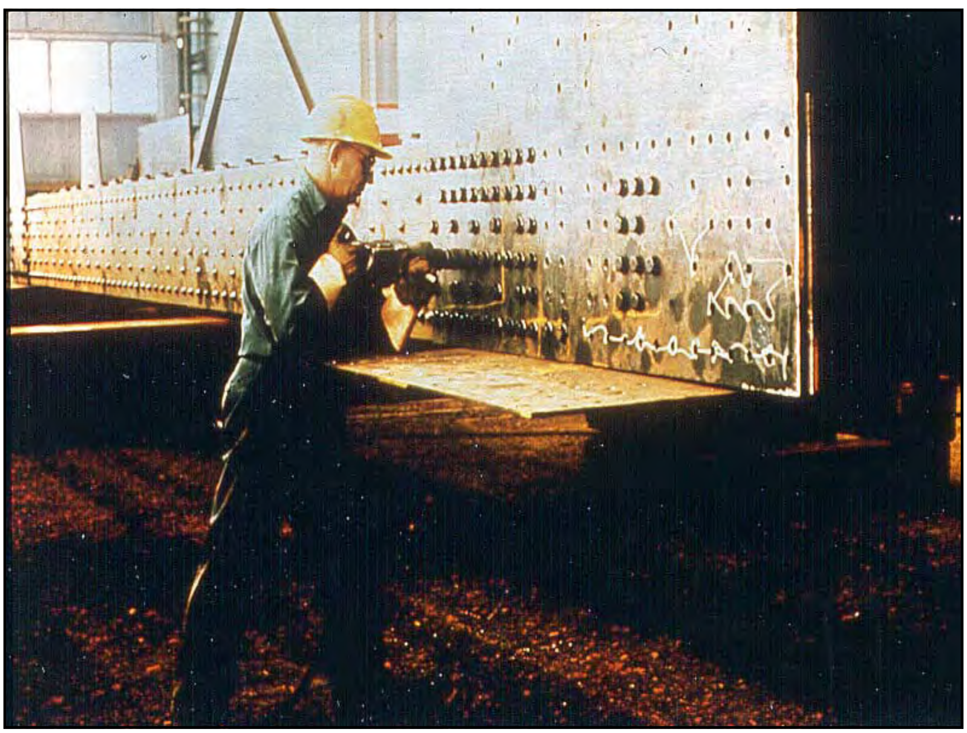
Bolt Installed Tension

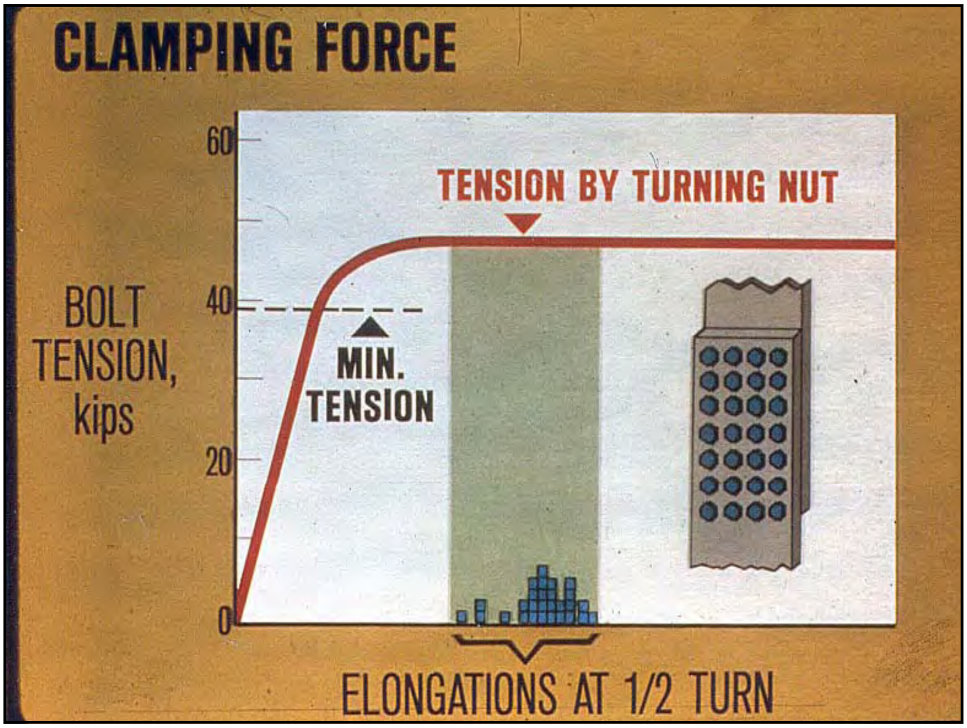
$P_t = 0.70 \times \text{Tensile Strength}$

$= 0.70 \times A_{tensile} \times F_u$

Post Slip Examination of Zinc Rich Paint Specimen



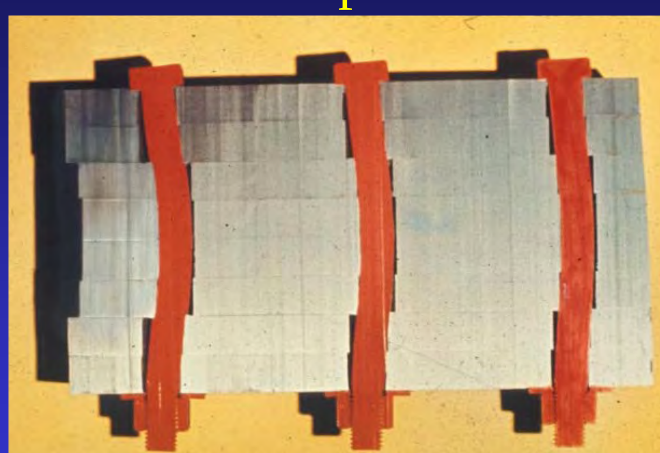




Diameter (in.)	0.625	0.75	0.875	1	1.125	1.25	1.375
A_b (in^2)	0.307	0.442	0.601	0.785	0.994	1.227	1.485
A325 Bolt							
F_{ub} (ksi)	120	120	120	120	105	105	105
$F_{ub} A_b$ (kip)	36.8	53.0	72.2	94.2	104.4	128.9	155.9
P_t (kip)	19	28	39	51	56	71	85
Type	$\phi_s R_n$ (kip)						
A325F	9.5	14.0	19.5	25.5	28.0	35.5	42.5
A325N	11.2	16.1	21.9	28.7	31.7	39.2	47.4
A325X	14.1	20.4	27.7	36.2	40.1	49.5	59.9

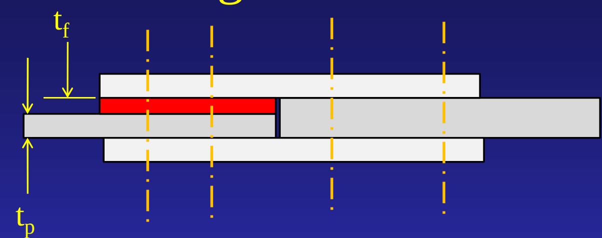
Diameter (in.)	0.625	0.75	0.875	1	1.125	1.25	1.375
A_b (in^2)	0.307	0.442	0.601	0.785	0.994	1.227	1.485
A490 Bolt							
F_{ub} (ksi)	150	150	150	150	150	150	150
F_{ub} A_b (kip)	46.0	66.3	90.2	117.8	149.1	184.1	222.7
P_t (kip)	24	35	49	64	80	102	121
Type	$\phi_s R_n$ (kip)						
A490F	12.0	17.5	24.5	32.0	40.0	51.0	60.5
A490N	14.0	20.1	27.4	35.8	45.3	56.0	67.7
A490X	17.7	25.4	34.6	45.2	57.3	70.7	85.5

Undeveloped Filler



Bending of Bolts Reduces Shear Strength

Filler at Flange Thickness Transition



Shear Strength Reduction due to Filler:

if $t_f \geq 0.25$in.

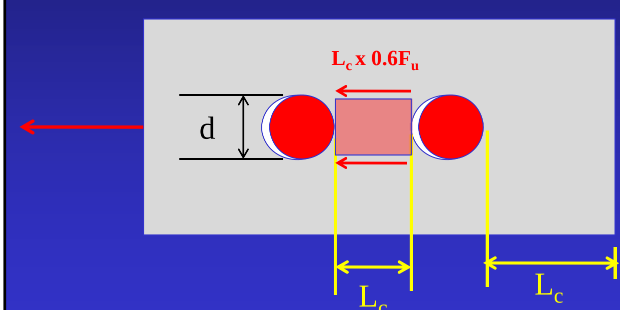
$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right]$$

$$\gamma = \frac{A_f}{A_p} \text{ normally} = \frac{t_f}{t_p}$$

A_f= Sum of the Area of the Filler Plates

A_p= Smaller of Connected Plate Area or Sum of Splice Plate Area

Bearing Strength of Connected Material Standard Holes



if $L_c \geq 2\,d$

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

for $L < 2\,d$

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

Φ_{bb}=0.80

AASHTO Bearing Strength

F_u = 65 ksi Hole Size=Dia.+0.0625

Diameter (in.)	0.625	0.75	0.875	1	1.125	1.25	1.375
Min. Spacing (in.)	1.875	2.25	2.625	3	3.375	3.75	4.125
Clear Distance (in.)	1.188	1.438	1.688	1.938	2.188	2.438	2.688
$\phi_{bb}R_n$ (kip/in. of thickness)	74.1	89.7	105.3	120.9	136.5	152.1	167.7
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
$\phi_{bb}R_n$ (kip/in. of thickness)	81.9	105.3	128.7	124.8	144.3	156	175.5
A325X/Bearing	0.17	0.19	0.22	0.29	0.28	0.32	0.34
A490X/Bearing	0.22	0.24	0.27	0.36	0.40	0.45	0.49
Min. End Distance (in.)	0.875	1	1.125	1.25	1.5	1.625	1.75
Clear Distance (in.)	0.53	0.59	0.66	0.72	0.91	0.97	1.03
$\phi_{bb}R_n$ (kip/in. of thickness)	33.15	37.05	40.95	44.85	56.55	60.45	64.35

Φ_{bb}=0.80

Design Conditions

- Dead Load and Service Load II- Design Against Slip
- Strength I- Design for Shear, Bearing, Block Shear, and Net Section
- Minimum Connection- Controls for many field Splices

Minimum Connection- Web Shear Article 6.13.6.1.4b

- If $V_u < 0.5\ \phi_v\ V_n$ (Shear at Strength I less ½ shear capacity)
  - $V_{uw} = 1.5 \times V_u$
- Otherwise:

$$V_{uw} = \frac{\left(V_u + \varphi_v\ V_n\right)}{2}$$

Minimum Flange Force Controlling Flange (largest stress)

$$\text{Minimum Flange Force} = F_{cf}\ \text{x}\ A_e$$

$$F_{cf} = \frac{\left(\left|\frac{f_{cf}}{R_h}\right| + \alpha\ \varphi_f\ F_{yf}\right)}{2} \geq 0.75\ \alpha\ \varphi_f\ F_{yf}$$

$$A_e = A_g \text{compression flange}$$

$$= \left(\frac{\varphi_u\ F_u}{\varphi_y\ F_{yt}}\right) A_{net} \leq A_g \text{tension flange}$$

Controlling Flange Requirements $\alpha = R_h = 1$

$$\text{Minimum Flange Force} = F_{cf} \text{ x } A_e$$

$$F_{cf} = \frac{(f_{cf} + F_{yf})}{2} \geq 0.75 \; F_{yf}$$

$$A_e = A_g \text{compression flange}$$

$$= 1.09 \; A_{net} \leq A_g \text{ Grade 50 tension flange}$$

$$= 1.18 \; A_{net} \leq A_g \text{ Grade 50W tension flange}$$

Non Controlling Flange

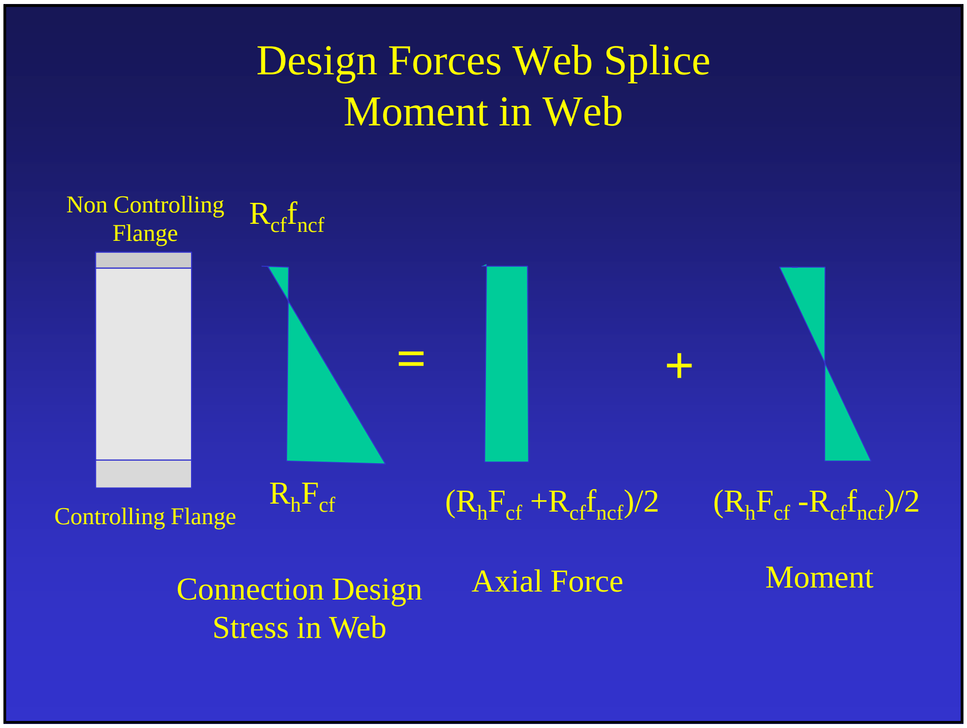
$$\text{Minimum Flange Force} = F_{nf} \text{ x } A_e$$

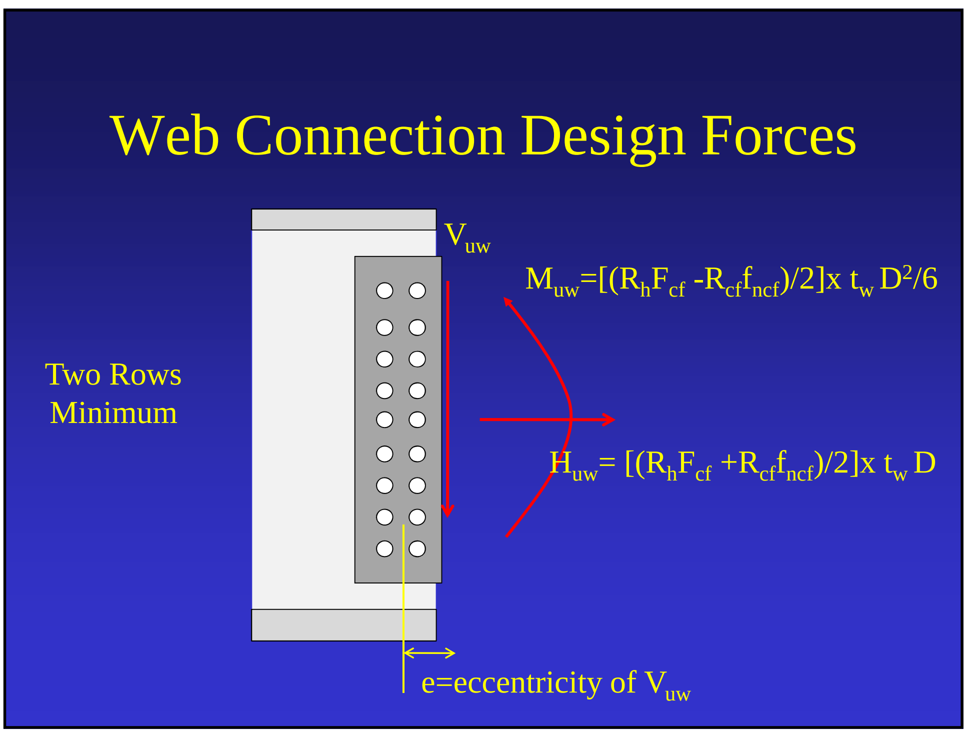
$$F_{nf} = R_{cf} \left| \frac{f_{nf}}{R_h} \right| \geq 0.75 \, \alpha \, \varphi_f \; F_{yf}$$

$$R_{cf} = \frac{F_{cf}}{f_{cf}} \text{ increase in design stress for controlling flange}$$

$$A_e = A_g \text{compression flange}$$

$$= \left(\frac{\varphi_u \; F_u}{\varphi_y \; F_{yt}} \right) A_{net} \leq A_g \text{tension flange}$$


Design Forces Web Splice
Moment in Web
Non Controlling Flange
$R_{cf}f_{ncf}$
Controlling Flange
R_hF_{cf}
=
+
$(R_hF_{cf} + R_{cf}f_{ncf})/2$
$(R_hF_{cf} - R_{cf}f_{ncf})/2$
Connection Design Stress in Web
Axial Force
Moment


Web Connection Design Forces
V_{uw}
$M_{uw}=[(R_hF_{cf} - R_{cf}f_{ncf})/2]x\ t_w\ D^2/6$
Two Rows Minimum
$H_{uw}= [(R_hF_{cf} + R_{cf}f_{ncf})/2]x\ t_w\ D$
e=eccentricity of V_{uw}

Web Connection Design Force

Moment:

$M = V_{uw} x\ e + M_{uw}$

Vertical Force:

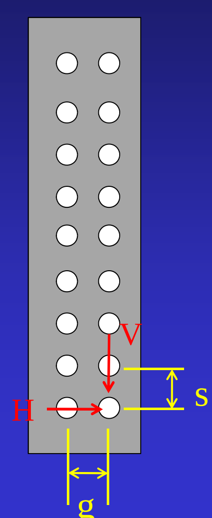
$V = V_{uw}$

Horizontal Force:

$H = H_{uw}$

- Calculate Force on Corner Fastener Using Rigid Body Equations Assuming Rotation About Centroid of Bolt Group

Web Corner Bolt Shear



$$I_p = \frac{n\,m}{12}\left[s^2\left(n^2-1\right)+g^2\left(m^2-1\right)\right]$$

n= number of bolts in one vertical row

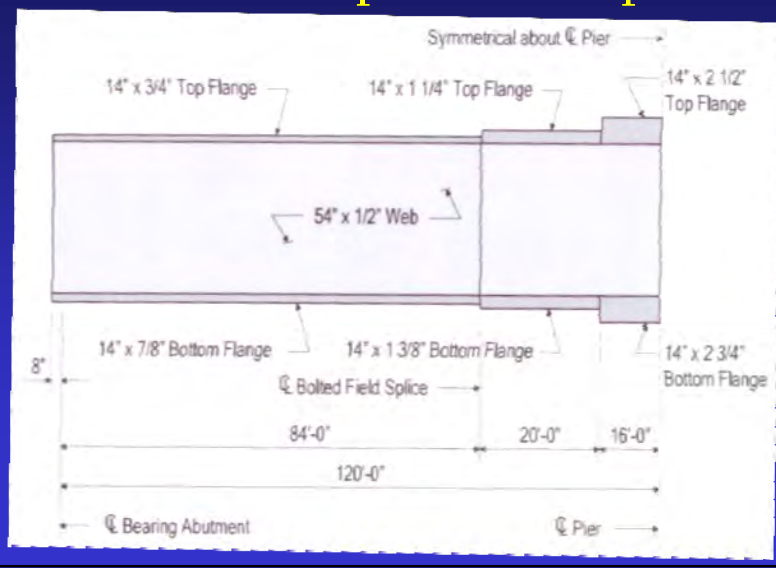
m= number of vertical rows

$$V = \text{Vertical Force} = \frac{V_{uw}}{nm} + \frac{\left(e\,V_{uw} + M_{uw}\right)}{I_p}\left[\frac{(m-1)}{2}g\right]$$

$$H = \text{Horizontal Force} = \frac{H_{uw}}{nm} + \frac{\left(e\,V_{uw} + M_{uw}\right)}{I_p}\left[\frac{(n-1)}{2}s\right]$$

$$\text{Maximum Bolt Shear} = \sqrt{V^2 + H^2}$$

NHI Girder Splice Example



Nominal Forces

Force	Service II	Strength I	A_{gross}	
Web Shear Force	-198.53 kip	262.2 kip		
Stress Top Flange Mid Thickness	-0.73 ksi	-1.32 ksi	14x3/4= 10.50	
Stress Bottom Flange Mid Thickness	15.1 ksi	20.69 ksi	14x7/8= 12.25	Controls

Controlling Flange Requirements Tension Flange, $\alpha = R_h = 1$

$$F_{cf} = \frac{(f_{cf} + F_{yf})}{2} = \frac{(20.69 + 50)}{2} = 35.3 < 0.75\ F_{yf} = 37.5\ \text{ksi}$$

Two Rows of $\frac{7}{8}$ in. bolts each side of web, standard holes.

$$A_{net} = 12.25 - 4\ \text{x} \left(\frac{15}{16}\right) \text{x} \frac{7}{8} = 8.97\ \text{in}^2$$

$$A_e = 1.09\ A_{net} = 1.09\text{x}8.75 = 9.77 \le A_g = 12.25\ \text{in}^2$$

Splice Design Force = 37.5 x 9.77 = 366.6 kip

Non Controlling Compression Flange

$$R_{cf} = \frac{F_{cf}}{f_{cf}} = \frac{37.5}{20.69} = 1.81\ \text{Ratio of Increase Controlling Flange}$$

Minimum Flange Force = F_{nf} x A_e

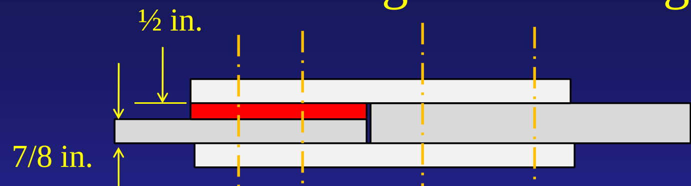
$$F_{nf} = R_{cf} \left|\frac{f_{nf}}{R_h}\right| = 1.81 \left|\frac{-1.32}{1}\right| = 2.18 < 0.75\, \alpha\, \varphi_f\ F_{yf}$$

$$= 0.75\ \text{x}\ 1\ \text{x}\ 1\ \text{x}\ 50 = 37.5\ \text{ksi}$$

$A_e = A_g = 10.5\ \text{in}^2$ compression flange

Splice Design Force = 37.5 x 10.5 = 394.8 kip

Bottom Flange Bolt Strength



Shear Strength Reduction due to Filler since $t_f \geq 0.25\text{in.}$:

$$t_f = \frac{7}{8}\text{in} \quad t_s = 1\frac{3}{8} - \frac{7}{8} = \frac{1}{2}\text{in}$$

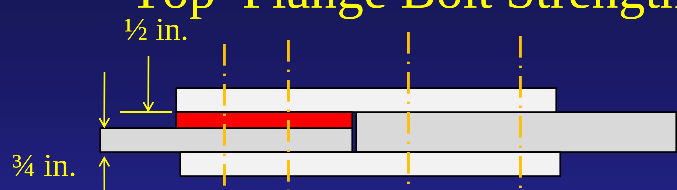
$$\gamma = \frac{0.5}{0.875} = 0.57$$

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right] = \left[\frac{(1+0.57)}{(1+2 \text{ x } 0.57)}\right] = 0.73$$

$\frac{7}{8}$ in A325 Bolt with Threads Excluded

Bolt Shear Strength = 0.73 x 27.7 = 20.2 kip

Top Flange Bolt Strength



Shear Strength Reduction due to Filler since $t_f \geq 0.25\text{in.}$:

$$t_f = \frac{3}{4}\text{in} \quad t_s = 1\frac{1}{4} - \frac{3}{4} = \frac{1}{2}\text{in}$$

$$\gamma = \frac{0.5}{0.75} = 0.67$$

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right] = \left[\frac{(1+0.67)}{(1+2 \text{ x } 0.67)}\right] = 0.71$$

$\frac{7}{8}$ in A325 Bolt with Threads Excluded

Bolt Shear Strength = 0.71 x 27.7 = 19.8 kip

Bolt Requirements

Bearing Based on 1.5 in. End Distance

Connection	Splice Design Force kip	Bolt Shear Capacity kip	Number of Bolts Required	Number Provided	Bearing Force/Bolt kip	Bearing End Bolt Flange kip
Top Flange	**394.8**	**19.8**	**394.8/(2x19.8)= 9.96**	**12**	**394.8/12= 32.9**	**64.35x ¾ = 48.3 OK**
Bottom Flange	**357.8**	**20.2**	**366.6/(2x20.2)= 9.07**	**12**	**366.6/12= 30.5**	**64.35x 7/8 = 56.3 OK**

t_{splice}= should be greater than half the smaller flange thickness

Top Flange t_{splice}>= 0.5x3/4 use 7/16 in.

Bottom Flange t_{splice}>= 0.5x7/8 use 7/16 in.

Web Connection Shear

- Unstiffened Web-k=5, D=54,t_w=0.5, D/t_w =108
  - V_n= C V_p , V_p= 0.58x50x54x0.5=783 kip

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2}\left(\frac{Ek}{F_{yw}}\right) = \frac{1.57}{\left(\frac{54}{0.5}\right)^2}\left(\frac{29{,}000x5}{50}\right)$$

$$= 0.39$$

$$V_n = 0.39 \text{ x } 783 = 305.4 \text{ kip}$$

$$V_u = 262.2 > 0.5\,\varphi_v V_n = 152.7 \text{ kip}$$

$$V_{uw} = \frac{(V_u + \varphi_v V_n)}{2} = \frac{(262.2 + 305.4)}{2} = 283.8 \text{ kips}$$

Web Connection Moment:

M_{uw}=[($R_h F_{cf}$ -$R_{cf} f_{ncf}$)/2]x t_w D^2/6

=[1x37.5-1.81x-1.32]x 0.5 x 54^2/12

= 4,847 k-in

V_{uw}x e = 283.8 x 3.25= 922.4 k-in

e=1.5 in. hole edge distance in web+ half the 1/2 in. space between webs(1/4 in.) + half the 3 in. row spacing in connection (1.5 in.) =3.25

Total Moment=4,847+922.4=5,769 k-in

Web Connection Horizontal Force:

H_{uw}= [($R_h F_{cf}$ +$R_{cf} f_{ncf}$)/2]x t_w D

= [(1x37.5+1.81x-1.32)/2]x0.5x54

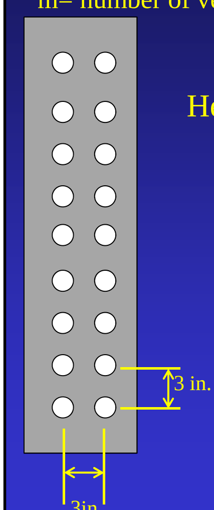
=474.0 kip

Web Corner Bolt Shear

Try:

n= number of bolts in one vertical row=15

m= number of vertical rows=2



Connection Height=14x3=42 in.

$$I_p = \frac{n\,m}{12}\left[s^2\left(n^2-1\right)+g^2\left(m^2-1\right)\right]$$

$$= \frac{15 \text{ x } 2}{12}\left[3^2\left(13^2-1\right)+3^2\left(2^2-1\right)\right]$$

$$= 5{,}108 \text{ in}^2$$

$$V = \frac{V_{uw}}{nm} + \frac{\left(e\,V_{uw}+M_{uw}\right)}{I_p}\left[\frac{(m-1)}{2}g\right]$$

$$= \frac{283.8}{30} + \frac{\left(3.25 \text{ x } 283.8 + 4{,}847\right)}{5{,}108}\left[\frac{(2-1)}{2}3\right] = 11.2 \text{ kip}$$

$$H = \frac{H_{uw}}{nm} + \frac{\left(e\,V_{uw}+M_{uw}\right)}{I_p}\left[\frac{(n-1)}{2}s\right]$$

$$= \frac{474}{30} + \frac{\left(922 + 4{,}847\right)}{5{,}882}\left[\frac{(15-1)}{2}3\right] = 39.5 \text{ kip}$$

$$\text{Maximum Bolt Shear} = \sqrt{V^2+H^2} = \sqrt{11.2^2+39.5^2}$$

$$= 41.1 \text{ kip} < 2\text{x}21.9 = 43.8 \text{ kip (A325N)}$$

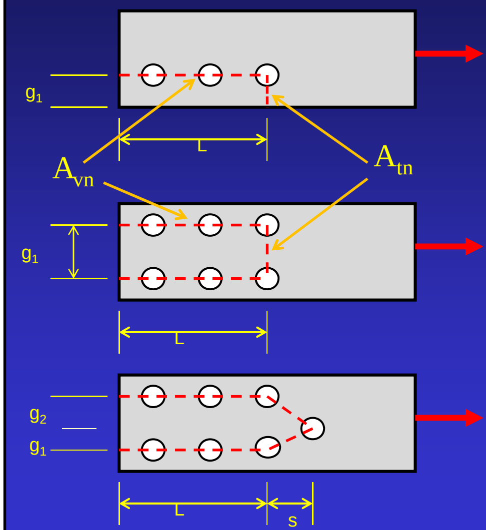
Contributions of Various Forces Upon Web Bolt Force

Force	V kip	H kip
V_{uw}	9.46	0
e x V_{uw}	0.27	3.8
M_{uw}	1.4	19.9
H_{uw}	0	15.8
Total	11.13	39.5

Additional Checks

- Bearing on Web Connection
- Block Shear
- Net Section
- Slip During Construction and Fatigue Loading

Block Shear Modes



Φ_{bs}=0.80

$$R_r = \varphi_{bs} R_p \left(0.58\, F_u A_{vn} + U_{bs} F_u A_{tn}\right)$$
$$\leq \varphi_{bs} R_p \left(0.58\, F_y A_{vg} + U_{bs} F_u A_{tn}\right)$$

R_p = 1 drilled holes

= 0.9 punched holes (cross frames)

U_{bs} = 1 axial force

= 0.5 bending

Good and Bad

1. Too Many Web Bolts ~72 bolts
2. End Distance on Flange Splice Plate?

Good and Bad

Two Rows in Web,
Higher Shear in Outer Girder (Curved Bridge)

Questions?