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Session Description

Session 4: Design Examples

July 9, 2018

This session will present a design example for vertically-curved members. In addition to demonstration the design of the member, the example will address the design of the connection. Then, the session will present a design example for horizontally-curved members. In addition to demonstration the design of the member, the example will address the design of the connection.

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Learning Objectives

- Describes the steps in designing a vertically-curved member.
- List the limit states of the vertically-curved member connection.
- Describes the steps in designing a horizontally-curved member.
- List the limit states of the horizontally-curved member connection.

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There's always a solution in steel.

Design of Curved Members

Session 4: Design Examples

July 9, 2018

Presented by
Bo Dowswell, P.E., Ph.D.
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Birmingham, AL

structural STEEL

There's always a solution in steel.

Design Examples

Session Description

structural STEEL

Session Description

- Design examples
  - Example 1: vertically-curved member
  - Example 2: horizontally-curved member

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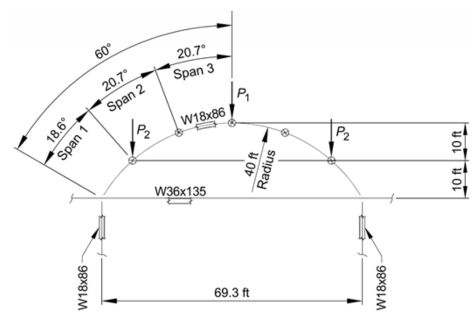
Design Examples

Example 1: Vertically-Curved Member

structural STEEL

Example 1



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There's always a solution in steel.

Design Examples

Example 1

Problem Statement

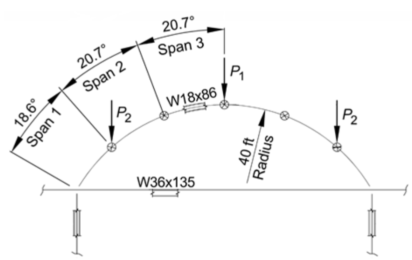
structural STEEL

Problem Statement

- Verify that the arch is adequate for the imposed loading
- Use LRFD

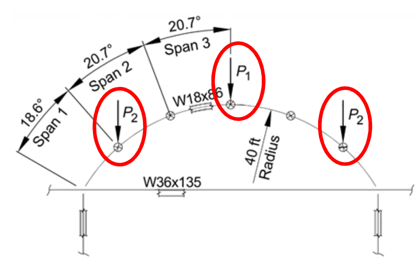
Problem Statement

- Curved member
  - W18×86
  - ASTM A992
  - Bent the hard way
  - Circular curve



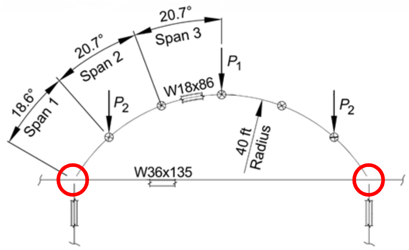
Problem Statement

- The factored (LRFD) loads are
  - P_{1u} = 120 kips
  - P_{2u} = 75 kips



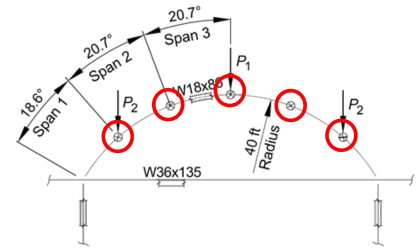
Problem Statement

- Supports
  - Translation fixed in all directions
  - Rotation free in all directions



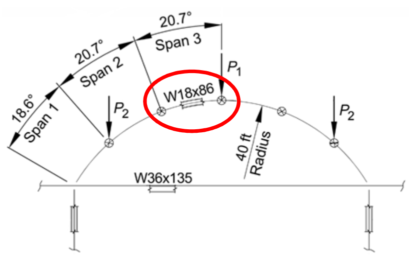
Problem Statement

- Braces
  - Prevent out-of-plane translation
  - Prevent torsional rotation



Problem Statement

- Assume Span 3 is critical



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Design Examples

Example 1

Properties

structural STEEL

Properties

- Material properties of ASTM A992 (AISC *Manual* Table 2-4)

F_y = 50 ksi
F_u = 65 ksi

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Properties

- Dimensions of W18×86 (AISC *Manual* Table 1-1)

d = 18.4 in. t_w = 0.480 in.
b_f = 11.1 in. t_f = 0.770 in.
h_o = 17.6 in.

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Properties

- Section properties of W18×86 (AISC *Manual* Table 1-1)

I_x = 1,530 in.^{4} S_x = 166 in.^{3}
r_x = 7.77 in. Z_x = 186 in.^{3}
I_y = 175 in.^{4} r_y = 2.63 in.
J = 4.10 in.^{4} C_w = 13,600 in.^{6}
r_{ts} = 3.05 in.

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Design Examples

Example 1

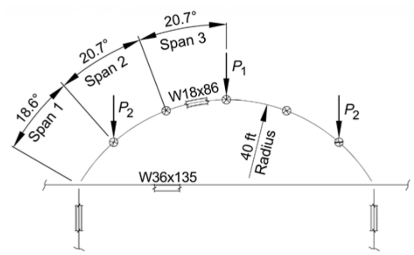
Arch Geometry

structural STEEL

Arch Geometry

- Centroidal radius

$$R = (40\text{ ft})(12\text{ in./ft})$$
$$= 480\text{ in.}$$



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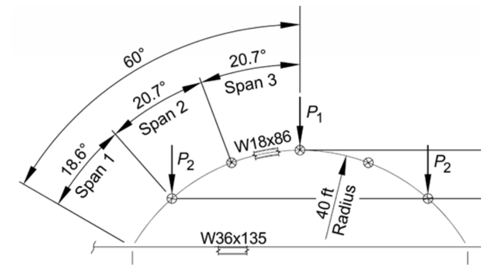
Arch Geometry

- Arch angle

$$\theta = (120^\circ)\left(\frac{\pi\text{ rad}}{180^\circ}\right)$$
$$= (2\pi/3)\text{ rad}$$



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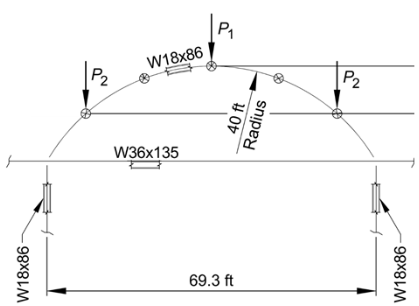
Arch Geometry

- Span length (chord)

$$L_s = (69.3\text{ ft})(12\text{ in./ft})$$
$$= 832\text{ in.}$$



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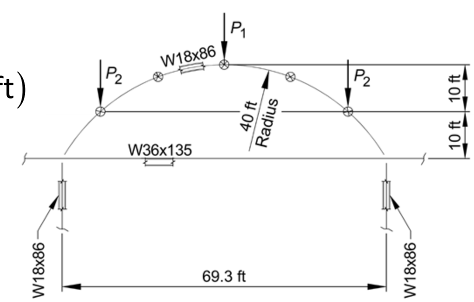
Arch Geometry

- Rise

$$H = (20\text{ ft})(12\text{ in./ft})$$
$$= 240\text{ in.}$$



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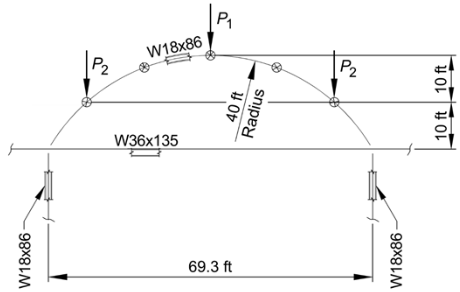
Arch Geometry

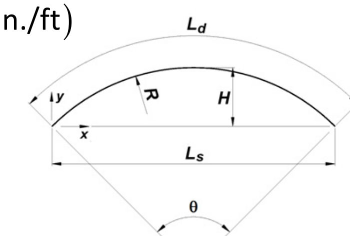
$$\frac{H}{L_s} = \frac{240\text{ in.}}{832\text{ in.}}$$
$$= 0.288$$



Arch Geometry

- Developed arc length

$$L_d = (40\text{ ft})\left[(2\pi/3)\text{rad}\right](12\text{ in./ft})$$
$$= 1{,}010\text{ in.}$$



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Design Examples

Example 1

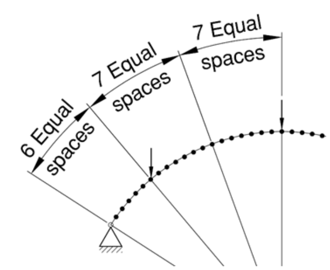
Structural Analysis

structural STEEL

Structural Analysis

Finite Element Model

- Segmented spans
  - Straight beam elements
  - ≈ 3° arc between nodes
- First-order analysis

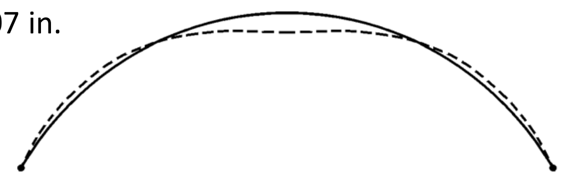


Structural Analysis

Deflection

- Maximum @ apex

Δ_1 = 1.07 in.



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Structural Analysis

Deflection

- If Δ_1 is less than $H/40$, a first-order finite element analysis is sufficiently accurate

$$\frac{H}{40} = \frac{240\text{ in.}}{40} = 6.00\text{ in.} \qquad 1.07\text{ in.} < 6.00\text{ in.} \quad \textbf{o.k.}$$

Ref: King, C. and Brown, D. (2001), *Design of Curved Steel*, The Steel Construction Institute

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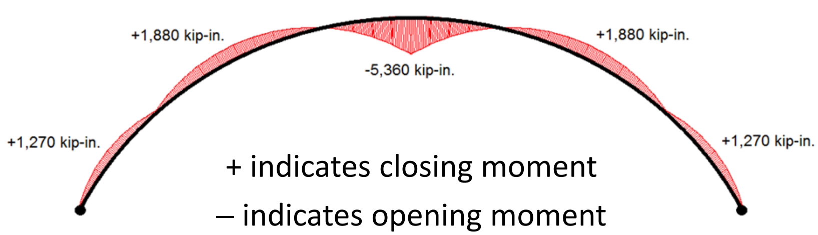
Arch Geometry

In-Plane Moment



+ indicates closing moment

– indicates opening moment

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Structural Analysis

Member Loads (kip, in.)			
Location	Axial	Moment	Shear
	P_u	M_{ux}	V_u
Supports	182	0	25.8
Apex	118	–5,360	57.5

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Structural Analysis

1st–Order Member Loads (kip, in.)			
Location	Axial	Moment	Shear
	P_u	M_{ux}	V_u
Max./Min. @ Span 3	131	+1,380 –5,360	57.5

There's always a solution in steel.

Design Examples

Example 1

Local Buckling

structural STEEL

Local Buckling

- Calculations are the same as for a straight member
- Axial: $\lambda_f < \lambda_{rf}$ and $\lambda_w < \lambda_{rw} \rightarrow$ the W18×86 is non-slender
- Flexure: $\lambda_f < \lambda_{pf}$ and $\lambda_w < \lambda_{pw} \rightarrow$ the W18×86 is compact

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Design Examples

Example 1

Shear Strength

structural STEEL

Shear Strength

- AISC *Manual* Table 6-2

$$\phi_v V_n = 265 \text{ kips} > V_u = 57.5 \text{ kips} \quad \textbf{o.k.}$$

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There's always a solution in steel.

Design Examples

Example 1

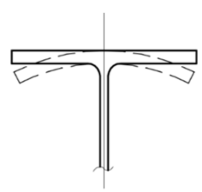
Local Flange Bending

structural STEEL

Local Flange Bending

Simplified Method

$$k_f = \frac{9.20}{8.80 + \dfrac{b_f^2}{R t_f}} \le 1.0$$



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Local Flange Bending

Simplified Method

$$k_f = \frac{9.20}{8.80 + \dfrac{(11.1 \text{ in.})^2}{(480 \text{ in.})(0.770 \text{ in.})}} \le 1.00$$

$$= 1.01 > 1.00$$

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Local Flange Bending

Simplified Method

- k_f = 1.00
- Flange bending will not occur
- The effective flexural properties are equal to the straight-member properties

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Design Examples

Example 1

In-Plane Strength

structural STEEL

In-Plane Strength

- Snap-through buckling is not critical because:
  - The supports are rigid
  - H/L_s = 0.288 > 0.2

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In-Plane Strength

- AISC *Specification* Section E3
- Radius of gyration about the axis of curvature: $r_i = r_x$ = 7.77 in.

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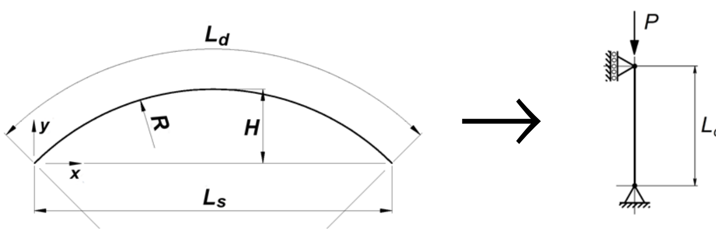
In-Plane Strength

- Unbraced length: L_d = 1,010 in.



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In-Plane Strength

- For a circular arch with pinned end conditions and H/L_s = 0.288, K_i = 0.55

In-plane effective length factor, K_i			
Arch Form	**End Conditions**	**H/L_s**	**K_i**
Circular	Pinned	$0.1 \le H/L_s \le 0.3$	0.55
		$0.3 \le H/L_s \le 0.5$	0.60
	Fixed	All	0.40

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In-Plane Strength

$$\frac{L_c}{r} = \frac{K_i L_d}{r_i}$$
$$= \frac{(0.55)(1{,}010\text{ in.})}{7.77\text{ in.}}$$
$$= 71.5$$

AISC *Manual* Table 4-14: $\phi_c F_{cr}$ = 31.0 ksi

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In-Plane Strength

- The available strength is

$$\phi_c P_{ni} = (31.0\text{ ksi})(25.3\text{ in.}^2)$$
$$= 784\text{ kips}$$

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Design Examples

Example 1

Out-of-Plane Strength

There's always a solution in steel.

Out-of-Plane Strength

- AISC *Specification* Section E3
- Moment of inertia perpendicular to the axis of curvature: $I_o = I_y = 175$ in.^{4}
- Radius of gyration perpendicular to the axis of curvature: $r_o = r_y = 2.63$ in.

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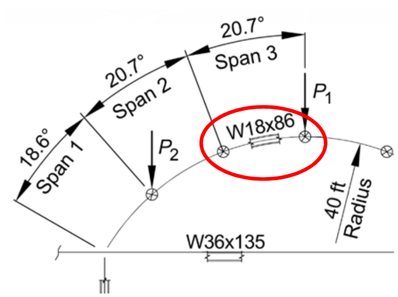
Out-of-Plane Strength

Span 3

$$\theta_b = (20.7^\circ)\left(\frac{\pi \text{ rad}}{180^\circ}\right)$$
$$= 0.361 \text{ rad}$$



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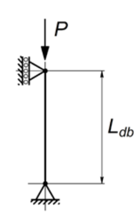
Out-of-Plane Strength

$$L_{db} = R\theta_b$$
$$= (480 \text{ in.})(0.361 \text{ rad})$$
$$= 173 \text{ in.}$$



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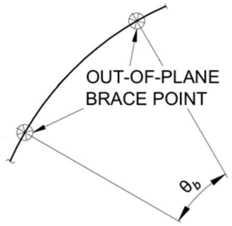
Out-of-Plane Strength

- Effective length factor, K_o: Circular doubly-symmetric segments

$$K_o = \frac{\sqrt{1+\frac{1}{C_o}\left(\frac{\theta_b}{\pi}\right)^2}}{1-\left(\frac{\theta_b}{\pi}\right)^2}$$



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Out-of-Plane Strength

$$C_o = \frac{1}{I_o}\left[\frac{GJ}{E}+C_w\left(\frac{\pi}{L_{db}}\right)^2\right]$$

$$= \frac{1}{175 \text{ in.}^4}\left[\frac{(11{,}200 \text{ ksi})(4.10 \text{ in.}^4)}{29{,}000 \text{ ksi}}+(13{,}600 \text{ in.}^6)\left(\frac{\pi}{173 \text{ in.}}\right)^2\right]$$

$$= 0.0347$$

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Out-of-Plane Strength

$$K_o = \frac{\sqrt{1+\frac{1}{C_o}\left(\frac{\theta_b}{\pi}\right)^2}}{1-\left(\frac{\theta_b}{\pi}\right)^2} = \frac{\sqrt{1+\frac{1}{0.0347}\left(\frac{0.361 \text{ rad}}{\pi}\right)^2}}{1-\left(\frac{0.361 \text{ rad}}{\pi}\right)^2} = 1.19$$

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Out-of-Plane Strength

$$\frac{L_c}{r} = \frac{K_o L_{db}}{r_o}$$

$$= \frac{(1.19)(173 \text{ in.})}{2.63 \text{ in.}}$$

$$= 78.3$$

AISC *Manual* Table 4-14: $\phi_c F_{cr}$ = 28.7 ksi

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Out-of-Plane Strength

- The available strength is

$$\phi_c P_{no} = (28.7\text{ ksi})(25.3\text{ in.}^2)$$
$$= 726\text{ kips}$$

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Design Examples

There's always a solution in steel.

Example 1

Second-Order Effects

structural STEEL

Second-Order Effects

- Amplified first-order analysis
- Second-order moment: $M_{ux2} = B_i M_{ux}$

$$B_i = \frac{1}{1 - \alpha\dfrac{P_u}{P_{ei}}}$$

P_{ei} = elastic critical load for in-plane buckling

α = 1.00 (LRFD)

REF: Session 2 Slide 87

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Second-Order Effects

$$F_e = \frac{\pi^2 E}{\left(\dfrac{L_c}{r}\right)^2}$$
$$= \frac{\pi^2(29{,}000\text{ ksi})}{(71.5)^2}$$
$$= 56.0\text{ ksi}$$

$$P_{ei} = F_e A_g$$
$$= (56.0\text{ ksi})(25.3\text{ in.}^2)$$
$$= 1{,}420\text{ kips}$$

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Second-Order Effects

$$B_i = \frac{1}{1-\alpha\frac{P_u}{P_{ei}}} = \frac{1}{1-(1.0)\left(\frac{182\text{kips}}{1{,}420\text{ kips}}\right)} = 1.15$$

Second-Order Effects

2nd–Order Moments (kip-in.)	
Location	M_{ux2}
Max./Min. @ Span 3	+1,590 –6,160

There's always a solution in steel.

Design Examples

Example 1

Flexural Strength

structural STEEL

Flexural Strength

- AISC *Specification* Section F2
- Moment of inertia perpendicular to the axis of curvature: $I_o = I_y = 175$ in.^{4}

Flexural Strength

$$M_p = F_y Z_x$$
$$= (50\text{ ksi})(186\text{ in.}^3)$$
$$= 9{,}300\text{ kip-in.}$$

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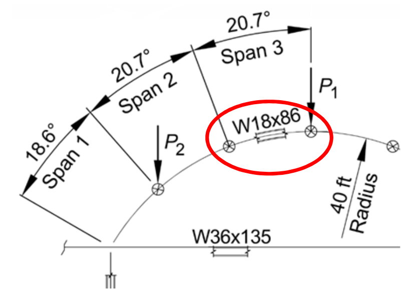
Flexural Strength

Span 3

$$L_{db} = 173\text{ in.}$$



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Flexural Strength

M_{es} = elastic lateral-torsional buckling moment of the equivalent straight member subjected to uniform moment with a length equal to L_{db}

$$M_{es} = \frac{\pi}{L_{db}}\sqrt{EI_oGJ + \left(\frac{\pi E}{L_{db}}\right)^2 I_oC_w} = 17{,}200\text{ kip-in.}$$

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Flexural Strength

$$C_y = EI_o$$
$$= (29{,}000\text{ ksi})(175\text{ in.}^4)$$
$$= 5{,}080{,}000\text{ kip-in.}^2$$

REF: Session 2 Slide 80

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Flexural Strength

$$C_z = GJ + \frac{\pi^2 E C_w}{L_{db}^2}$$

$$= (11{,}200 \text{ ksi})(4.10 \text{ in.}^4) + \frac{\pi^2 (29{,}000 \text{ ksi})(13{,}600 \text{ in.}^6)}{(173 \text{ in.})^2}$$

$$= 176{,}000 \text{ kip-in.}^2$$

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Flexural Strength

$$C_a = \frac{C_y + C_z}{2RM_{es}}$$

$$= \frac{5{,}080{,}000 \text{ kip-in.}^2 + 176{,}000 \text{ kip-in.}^2}{(2)(480 \text{ in.})(17{,}200 \text{ kip-in.})}$$

$$= 0.318$$

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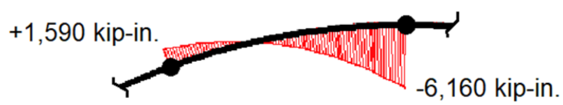
Flexural Strength

- Span 3



$$C_{bs} = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} = 2.38$$

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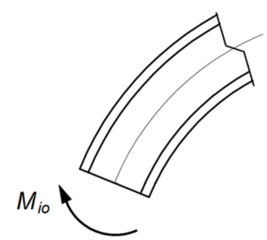
Flexural Strength

Opening moments $\rightarrow$ negative root

$$C_{bi} = C_{bs}\left(\sqrt{1 + C_a^2 - \frac{C_y C_z}{R^2 M_{es}^2}} - C_a\right)$$

$$= 1.75$$



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Flexural Strength

- AISC *Specification* Section F2
  - $L_b = L_{db}$ = 173 in. = 14.4 ft
  - $C_b = C_{bi}$ = 1.75
- AISC *Manual* Table 3-6
  - L_p = 9.29 ft
  - L_r = 28.6 ft

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Flexural Strength

- $L_p < L_b < L_r \rightarrow$ Use AISC *Spec.* Eq. F2-2

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$= 14{,}700 \text{ kip-in.} > 9{,}300 \text{ kip-in.}$$

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Flexural Strength

- The available strength is

$$\phi_b M_n = 0.90(9{,}300 \text{ kip-in.})$$
$$= 8{,}370 \text{ kip-in.}$$

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There's always a solution in steel.

Design Examples

Example 1

Combined Loading

structural STEEL

Combined Loading

- AISC *Specification* Section H1
- In-plane buckling
- Out-of-plane buckling

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Combined Loading

In-Plane Buckling

- The largest flexural load ratio is at the apex
- The largest axial load ratio is at the supports

$$\frac{P_u}{\phi_c P_{ni}} = \frac{182 \text{ kips}}{784 \text{ kips}} = 0.232$$

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Combined Loading

0.232 > 0.2 → AISC *Specification* Equation H1-1a

$$\frac{P_u}{\phi_c P_{ni}} + \frac{8}{9}\left(\frac{M_u}{\phi_b M_n}\right) \leq 1.0$$

$$0.232 + \left(\frac{8}{9}\right)\left(\frac{6{,}160 \text{ kip-in.}}{8{,}370 \text{ kip-in.}}\right) \leq 1.0$$

$$0.886 < 1.0 \quad \textbf{o.k.}$$

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Combined Loading

Out-of-Plane Buckling (Span 3)

- Largest flexural load in the span
- Largest axial load in the span

$$\frac{P_u}{\phi_c P_{no}} = \frac{131 \text{ kips}}{726 \text{ kips}} = 0.180$$

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Combined Loading

0.180 < 0.2 → AISC *Specification* Equation H1-1b

$$\frac{P_u}{2\phi_c P_{no}} + \frac{M_u}{\phi_b M_n} \le 1.0$$

$$\frac{0.180}{2} + \frac{6{,}160 \text{ kip-in.}}{8{,}370 \text{ kip-in.}} \le 1.0$$

$$0.826 < 1.0 \quad \textbf{o.k.}$$

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Design Examples

There's always a solution in steel.

Example 1

End Connections

structural STEEL

End Connections

- Support reactions from FE model
  - Vertical: R_{uy} = 140 kips
  - Horizontal: R_{ux} = 117 kips

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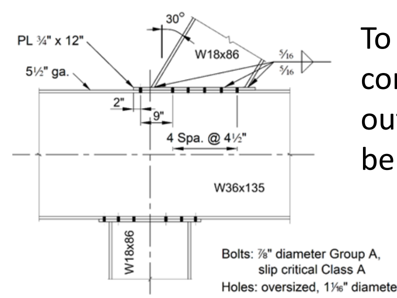
End Connections



To satisfy the boundary conditions assumed in design, out-of-plane translation must be restrained.

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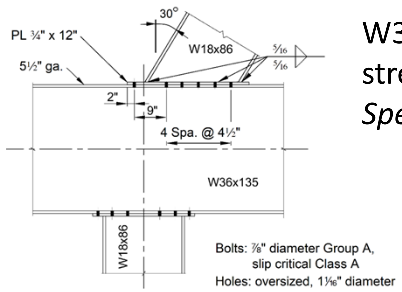
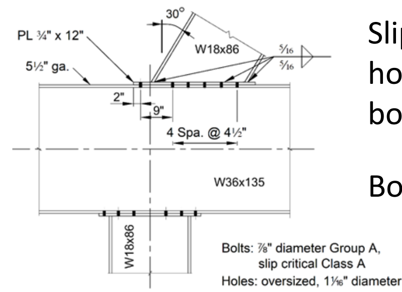
End Connections



W36 web local compression strength > 140 kips (AISC *Specification* Section J10)

89

End Connections

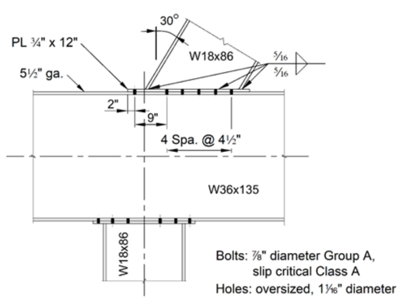


Slip critical joints increase horizontal rigidity by limiting bolt slip

Bolt slip load > 117 kips

90

End Connections



AISC COSP chord length tolerance: ± 1/8 in.

Slip critical bolts allow the use of oversized holes for easier field fit-up

REF: Session 2 Slide 23

91

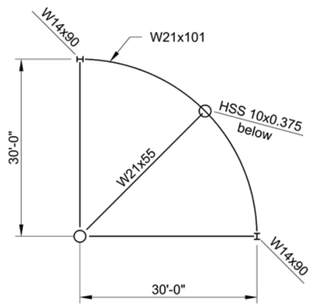
There's always a solution in steel.

Design Examples

Example 2: Horizontally-Curved Member

structural STEEL

Example 2



93

Design Examples

There's always a solution in steel.

Example 2

Problem Statement

structural STEEL

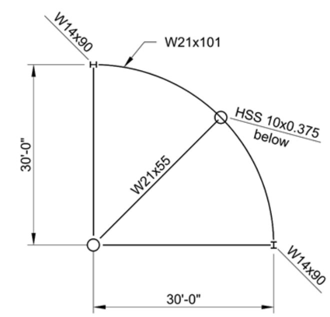
Problem Statement

- Verify that the horizontally-curved beam is adequate for the imposed loading
- Use LRFD

95

Problem Statement

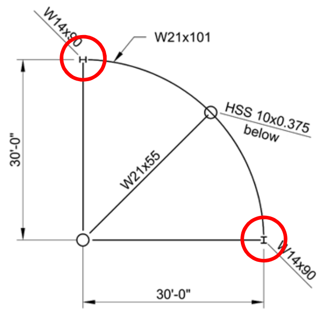
- Curved member
  - W21×101
  - ASTM A992
  - Bent the easy way
  - Circular curve



96

Problem Statement

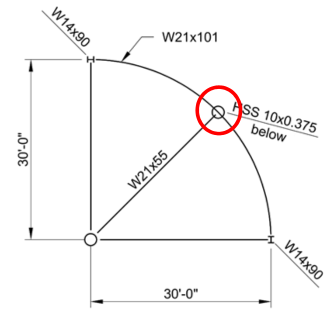
- End connections
  - Torsional rotation is restrained
  - No warping restraint
  - No flexural restraint



97

Problem Statement

- Connections @ the HSS10
  - Continuous for flexure
  - Torsion is restrained by the W21 beam
  - Continuous for warping



98

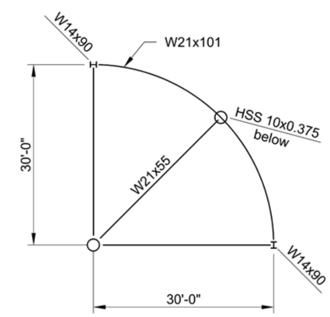
Problem Statement

- The factored uniformly distributed load along the member circumference including the beam self weight is w_u = 0.750 kip/ft

99

Problem Statement

- Assume the critical condition is for patch loading (one span loaded)



100

Design Examples

Example 2

Properties

Properties

- Material properties of ASTM A992 (AISC *Manual* Table 2-4)

  F_y = 50 ksi
  F_u = 65 ksi

Properties

- Dimensions of W21×101 (AISC *Manual* Table 1-1)

  d = 21.4 in. t_w = 0.500 in.
  b_f = 12.3 in. t_f = 0.800 in.
  h_o = 20.6 in.

Properties

- Section properties of W21×101 (AISC *Manual* Table 1-1)

  I_x = 2,420 in.^{4} S_x = 227 in.^{3}
  Z_x = 253 in.^{3}
  J = 5.21 in.^{4} C_w= 26,200 in.^{6}
  r_{ts} = 3.35 in.

There's always a solution in steel.

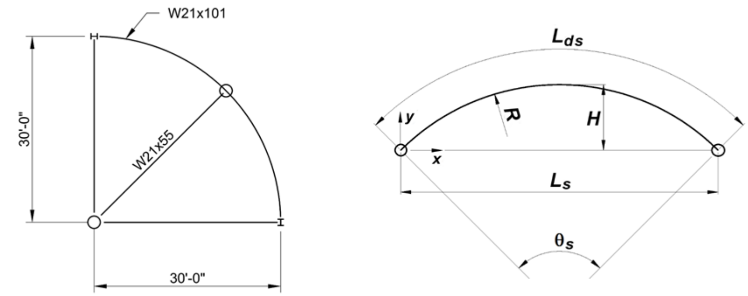
Design Examples

Example 2

Beam Geometry

structural STEEL

Beam Geometry

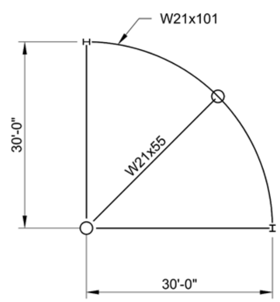


106

Beam Geometry

- Centroidal radius

$$R = (30\text{ ft})(12\text{ in./ft})$$
$$= 360\text{ in.}$$

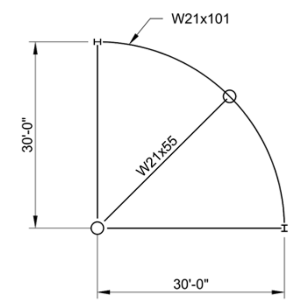


107

Beam Geometry

- Span angle

$$\theta_s = (45^\circ)\left(\frac{\pi\text{ rad}}{180^\circ}\right)$$
$$= \pi / 4\text{ rad}$$

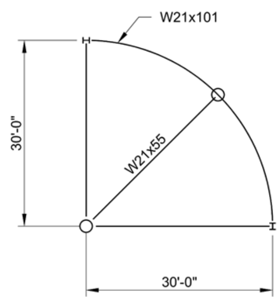


108

Beam Geometry

- Developed span length

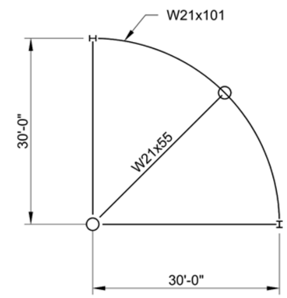
$$L_{ds} = (30\text{ ft})(\pi/4\text{ rad})(12\text{ in./ft})$$
$$= 283\text{ in.}$$



Beam Geometry

- Angle between torsional restraints

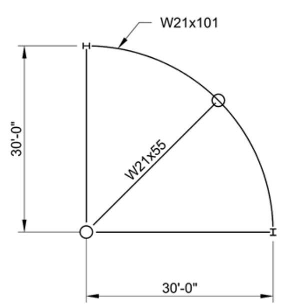
$$\theta_b = (45^\circ)\left(\frac{\pi\text{ rad}}{180^\circ}\right)$$
$$= \pi/4\text{ rad}$$



Beam Geometry

- Developed length between braces

$$L_{db} = (30\text{ ft})(\pi/4\text{ rad})$$
$$= 23.6\text{ ft}$$
$$= 283\text{ in.}$$



There's always a solution in steel.

Design Examples

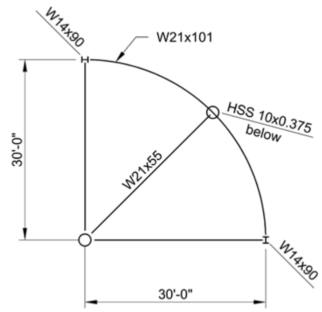
Example 2

Structural Analysis

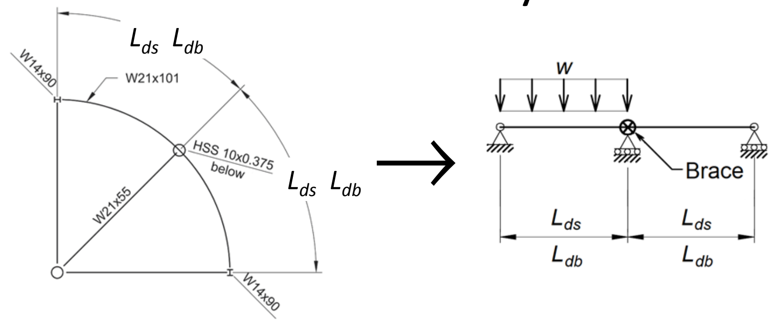
structural STEEL

Structural Analysis

- At the ends
  - No warping restraint
  - No flexural restraint
- At the HSS10
  - Continuous for flexure
  - Continuous for warping



Structural Analysis



There's always a solution in steel.

Design Examples

Example 2

Structural Analysis

Flexural Loads

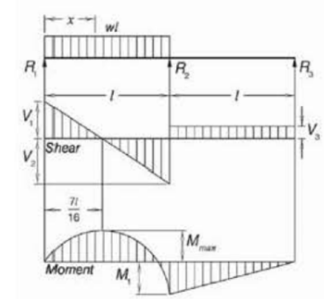
structural STEEL

Structural Analysis

- AISC *Manual* Table 3-23 Case 29
- Max. Moment

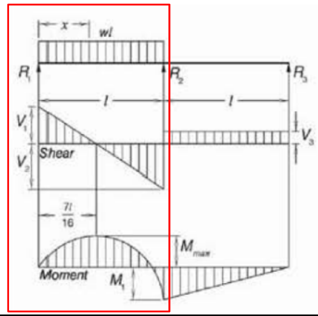
  M_{ux} = +480 kip-in.
- Max. Shear

  V_u = 9.95 kips



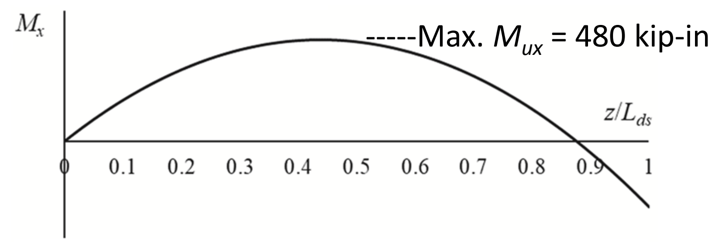
Structural Analysis

- Isolate the loaded span



Structural Analysis

- Flexural moment at the loaded span



There's always a solution in steel.

Design Examples

Example 2

Structural Analysis

Torsional Loads

Structural Analysis

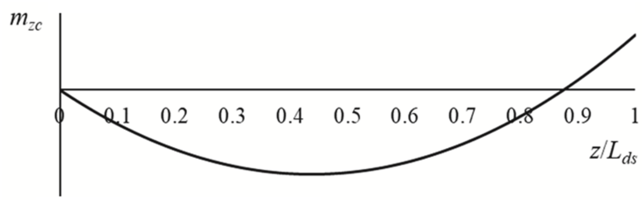
- M/R method
- Torsional moment per unit length:

$$m_{zc} = \frac{M_x}{R}$$

REF: Session 3 Slide 44

Structural Analysis

- Distributed torsion



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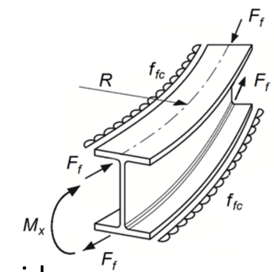
Structural Analysis

- Isolated Flange method

$$f_{fc} = \frac{m_{zc}}{h_o}$$



h_o = distance between flange centroids

REF: Session 3 Slide 82

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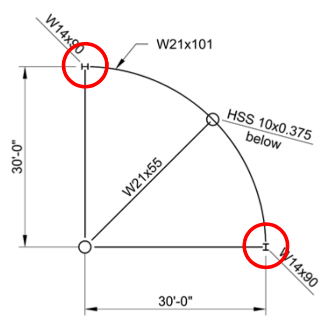
Structural Analysis

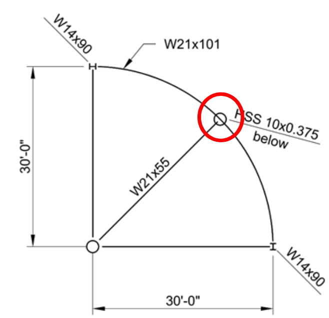
- Warping is not restrained at the end connections → use pinned ends at the isolated flange



123

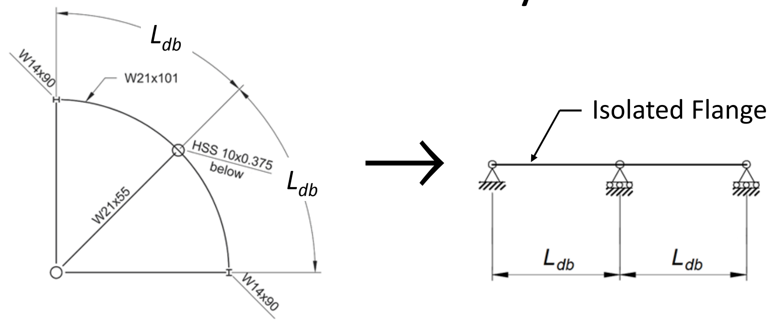
Structural Analysis

- Warping is continuous at the HSS10 → the isolated flange is continuous for flexure



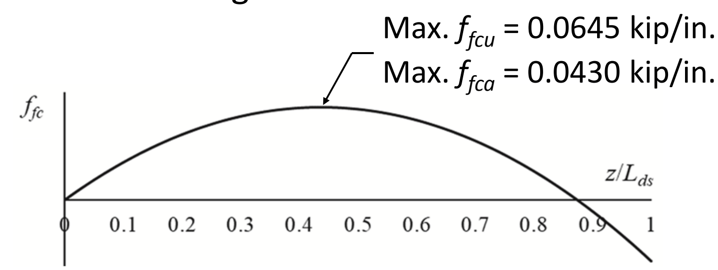
124

Structural Analysis



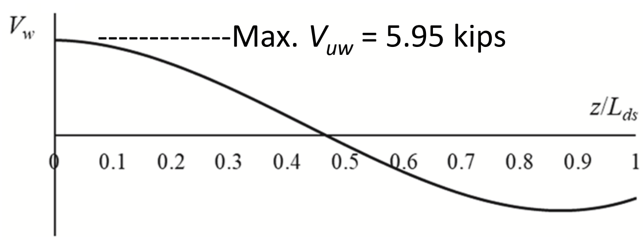
Structural Analysis

- Distributed flange force



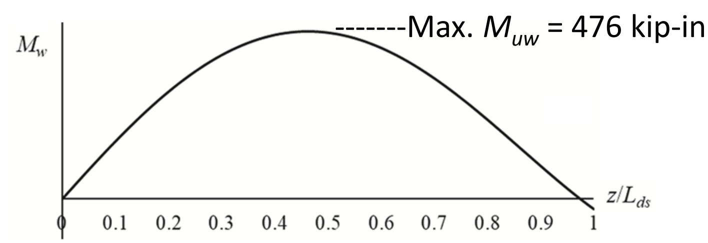
Structural Analysis

- Horizontal shear



Structural Analysis

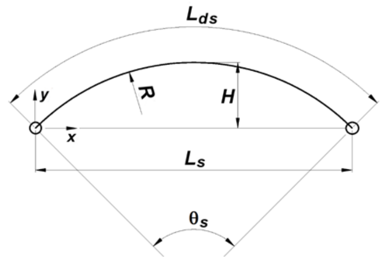
- Warping moment



Structural Analysis

Corrected Moments

$$C = 1 - \frac{\theta_s}{30} + \frac{\theta_s^2}{6.2}$$
$$= 1 - \frac{\pi/4}{30} + \frac{(\pi/4)^2}{6.2}$$
$$= 1.07$$



REF: Session 3 Slide 47

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Structural Analysis

Corrected Moments

- Flexure:

$$M_{uxc} = CM_{ux} = (1.07)(480 \text{ kip-in.}) = 514 \text{ kip-in.}$$

- Warping:

$$M_{uwc} = CM_{uw} = (1.07)(476 \text{ kip-in.}) = 509 \text{ kip-in.}$$

130

There's always a solution in steel.

Design Examples

Example 2

Shear Strength

structural
STEEL

Shear Strength

- AISC *Manual* Table 6-2

$$\phi_v V_n = 321 \text{ kips} > V_u = 9.95 \text{ kips} \quad \textbf{o.k.}$$

132

Design Examples

There's always a solution in steel.

Example 2

Local Buckling

Local Buckling

- Calculations are the same as for a straight member
- Flexure: $\lambda_f < \lambda_{pf}$ and $\lambda_w < \lambda_{pw} \rightarrow$ the W21×101 is compact

134

Design Examples

There's always a solution in steel.

Example 2

Flexural Strength

Flexural Strength

- Design as a straight beam
- AISC *Specification* Chapter F

136

Flexural Strength

Use C_{bs} = 1.0

$$C_{bo} = C_{bs}\left[1-\left(\frac{\theta_b}{\pi}\right)^2\right]^2$$

$$= (1.0)\left[1-\left(\frac{\pi/4}{\pi}\right)^2\right]^2 = 0.879$$

REF: Session 3 Slide 68

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Flexural Strength

- AISC *Specification* Section F2
  - $L_b = L_{db}$ = 23.6 ft
  - $C_b = C_{bo}$ = 0.879
- AISC *Manual* Table 3-6
  - L_p = 10.2 ft
  - L_r = 30.1 ft

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Flexural Strength

$$M_p = F_y Z_x$$

$$= (50 \text{ ksi})(253 \text{ in.}^3)$$

$$= 12{,}700 \text{ kip-in.}$$

139

Flexural Strength

- $L_p < L_b < L_r \rightarrow$ Use AISC *Spec.* Eq. F2-2

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$

$$= 8{,}350 \text{ kip-in.} < 12{,}700 \text{ kip-in.}$$

140

Flexural Strength

- The available strength is

$$\phi_b M_n = 0.90(8{,}350 \text{ kip-in.}) \\ = 7{,}520 \text{ kip-in.}$$

141

There's always a solution in steel.

Design Examples

Example 1

Second-Order Effects

structural STEEL

Second-Order Effects

- Amplified First-Order Analysis
- Second-order warping moment: $M_{uw} = B_o M_{uwc}$

$$B_o = \frac{0.85}{1 - \alpha \dfrac{M_{uxc}}{M_{eo}}} \geq 1.0$$

M_{eo} = elastic lateral-torsional buckling moment

α = 1.00 (LRFD)

REF: Session 3 Slide 95

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Second-Order Effects

$$F_{cr} = \frac{C_{bo}\pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \\ = 44.9 \text{ ksi}$$

144

Second-Order Effects

$$M_{eo} = F_{cr} S_x$$
$$= (44.9 \text{ ksi})(227 \text{ in.}^3)$$
$$= 10{,}200 \text{ kip-in.}$$

145

Second-Order Effects

$$B_o = \frac{0.85}{1-(1.0)\left(\dfrac{514 \text{ kip-in.}}{10{,}200 \text{ kip-in.}}\right)} \geq 1.0$$
$$= 0.895 < 1.0$$

$\rightarrow B_o = 1.0$

146

Second-Order Effects

$$M_{uw} = (1.0)(509 \text{ kip-in.})$$
$$= 509 \text{ kip-in.}$$

147

There's always a solution in steel.

Design Examples

Example 2

Warping Strength

structural STEEL

Warping Strength

- The isolated flange plastic modulus is

$$Z_f = \frac{t_f b_f^2}{4} = \frac{(0.800\text{ in.})(12.3\text{ in.})^2}{4} = 30.3\text{ in.}^3$$

REF: Session 3 Slide 87

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Warping Strength

- The nominal flexural strength of the isolated flange is

$$M_{nw} = F_y Z_f = (50\text{ ksi})(30.3\text{ in.}^3) = 1{,}520\text{ kip-in.}$$

150

Warping Strength

- The available flexural strength of the isolated flange is

$$M_{cw} = \phi_b M_{nw} = 0.90(1{,}520\text{ kip-in.}) = 1{,}370\text{ kip-in.}$$

151

There's always a solution in steel.

Design Examples

Example 2

Combined Loading

structural STEEL

Combined Loading

- Flexural moment + flange warping moment

$$\frac{M_{uxc}}{\phi_b M_n} + \frac{8}{9}\frac{M_{uw}}{\phi M_{nw}} \leq 1.0$$

$$\frac{514\text{ kip-in.}}{7{,}520\text{ kip-in.}} + \left(\frac{8}{9}\right)\left(\frac{509\text{ kip-in.}}{1{,}370\text{ kip-in.}}\right) \leq 1.0$$

$$0.399 < 1.00 \quad \textbf{o.k.}$$

REF: Session 3 Slide 100

There's always a solution in steel.

Design Examples

Example 1

Serviceability

structural STEEL

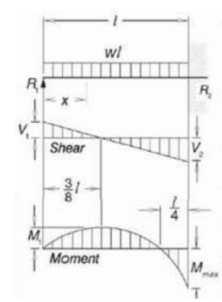
Serviceability

- The torsional rotation can be estimated using the horizontal deflection of the isolated flange
- Maximum second-order distributed flange force under service loads: f_{fc} = 0.0430 kip/in.

Serviceability

- AISC *Manual* Table 3-23 Case 12

$$\Delta_{max} = \frac{f_{fc} L_{ds}^4}{185 E I_f}$$



Serviceability

- The isolated flange moment of inertia is

$$I_f = \frac{t_f b_f^3}{12} = \frac{(0.800\text{ in.})(12.3\text{ in.})^3}{12}$$
$$= 124\text{ in.}^4$$

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Serviceability

$$\Delta_{max} = \frac{f_{fc} L_{ds}^4}{185 E I_f} = \frac{(0.0430\text{ kip/in.})(283\text{ in.})^4}{(185)(29{,}000\text{ ksi})(124\text{ in.}^4)}$$
$$= 0.415\text{ in.}$$

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Serviceability

- The 1st-order torsional rotation is

$$\theta_1 = \tan^{-1}\left(\frac{2\Delta_{max}}{h_o}\right) = \tan^{-1}\left[\frac{(2)(0.415\text{ in.})}{20.6\text{ in.}}\right]$$
$$= 2.31^\circ$$

h_o = distance between flange centroids

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Serviceability

- The 2nd-order torsional rotation is

$$\theta_2 = B_o \theta_1$$
$$= (1.00)(2.31^\circ)$$
$$= 2.31^\circ$$

160

There's always a solution in steel.

Design Examples

Example 2

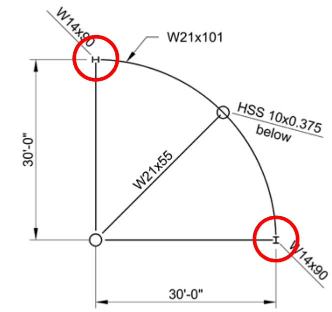
Connections

structural STEEL

Problem Statement

End Connections

- Torsional rotation is restrained
- No warping restraint
- No flexural restraint



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Connections

- Support reactions
  - Vertical reaction: R_u = 7.74 kips
  - Horizontal shear at flanges: V_{uw} = 5.95 kips

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Connections

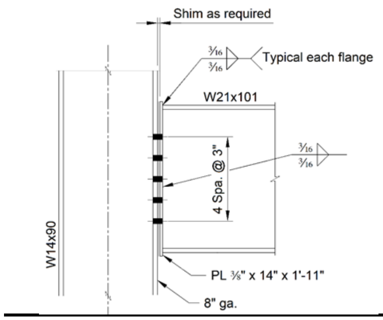
- Torsional moment

$$M_{uz} = V_{uw} h_o$$
$$= (5.95\text{ kips})(20.6\text{ in.})$$
$$= 123\text{ kip-in.}$$

h_o = distance between flange centroids

164

Connections

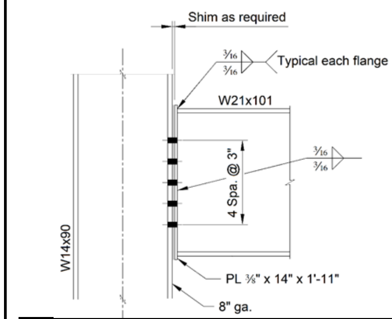


End-plate connections are efficient in transferring torsion

Flange welds transfer V_{uw}

165

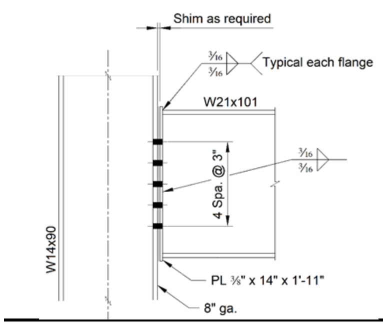
Connections



A wide column gage provides efficient torsional resistance

166

Connections



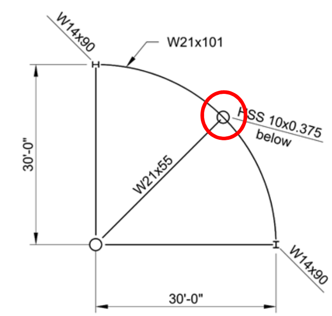
Shims are used to ensure proper fit-up

167

Problem Statement

Connections @ the HSS10

- Torsion is restrained by the W21 beam



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Connections

- Support reactions
  - Vertical reaction @ W21x55: R_u = 30 kips
  - Torsion @ W21x101: M_{uz} = 80.5 kip-in.

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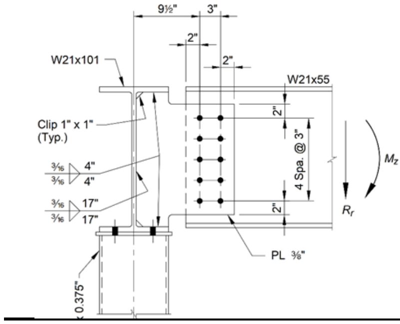
Connections



The torsional reaction from the curved beam is resisted by the single-plate connection

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There's always a solution in steel.

Question time

structural STEEL

Individual Webinar Registrants

CEU/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!

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

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

8-Session Registrants

CEU/PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz: Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG

Quiz and Attendance records: Posted Tuesday mornings. www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

•EEU – must take all quizzes and final to receive EEU
•CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHs.
•REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

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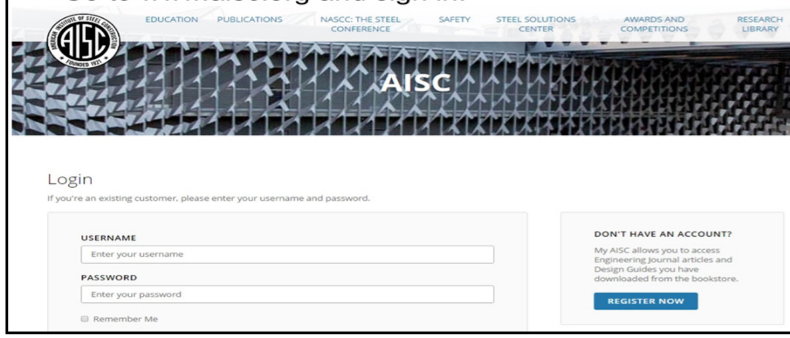
Night School Resources for 8-session package Registrants

Go to www.aisc.org and sign in.



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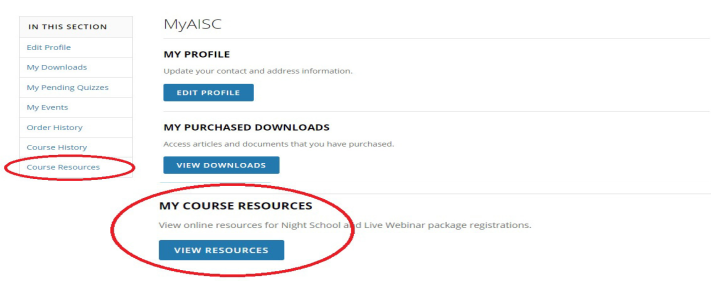

Night School Resources for 8-session package Registrants

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending
NS13 - Final Exam	4/10/2017 7:00:00 PM			Available 04/12/2017 5pm EST	

Night School Resources for 8-session package Registrants

- Weekly "quiz and recording" email.
- Weekly updates of the master Quiz and Attendance record found at www.aisc.org/nightschool. Scroll down to Quiz and Attendance records.
  - Updated on Tuesday mornings.

Night School Resources for 8-session package Registrants

- Webinar connection information:
  - Found in your registration confirmation/receipt.
  - Reminder email sent out Monday mornings.
- Link to handouts also found here.

There's always a solution in steel.

Thank You

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

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