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Course Description

Design of Reinforcement for Steel Members Part II
Presented by Bo Dowswell, Ph.D., P.E.

February 13, 2014

This presentation is a continuation of the AISC live webinar, *Design of Reinforcement for Steel Members*. Several topics will be discussed which affect the strength and serviceability of reinforced steel members. Deflection of beams with pre-load will be discussed briefly, and further information will be provided on the design of built-up columns. Other topics include: weld design, including the calculation of weld strength for built-up columns; how intermittent welds affect the section properties of a built-up member; local buckling, including the effect of stitch welds on the local buckling of plates. An example problem will be used to illustrate some of the design principles for a composite beam with reinforcement at the tension flange.

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Learning Objectives

- To learn and understand the AISC Specification strength and serviceability requirements for designing reinforced structural steel members.
- To gain familiarity with the deflection of beams with pre-load when safely reinforcing steel members for structural steel buildings.
- To learn and understand material and code requirements for welding of built-up structural steel beams and columns.
- To learn and understand the design principles for reinforcing composite beams through the presentation of a design example.

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DESIGN OF REINFORCEMENT FOR STEEL MEMBERS- PART 2

Bo Dowswell, P.E., Ph.D.
Principal
ARC International, LLC

structural STEEL

Part 1 Summary

- Reinforced Beam & Column Design
  - Reinforcement types
  - Pre-load
  - Partial-length reinforcement
- Course video and presentation slides are available on the AISC Website: www.aisc.org/content.aspx?id=37256

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Course Description

- Beam Deflection
- Section Properties of Built-Up Members
- Weld Design
- Built-Up Columns: Additional Topics
- Flexural Plate Buckling (Plate Buckling Between Intermittent Welds)
- Composite Beam Example

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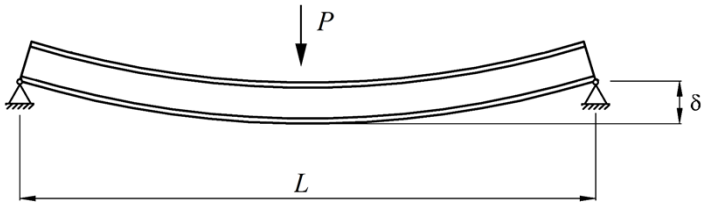
BEAM DEFLECTION

Beam Deflection

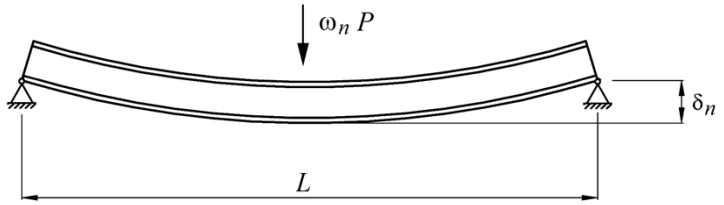
Deflection of Beams with Pre-Load

- Structural analysis models are built with reinforced member properties
- Deflection must be adjusted to account for pre-load.



Beam Deflection

Pre-load deflection: deflection before reinforcement is added.



ω_n = proportion of load applied before the member is reinforced

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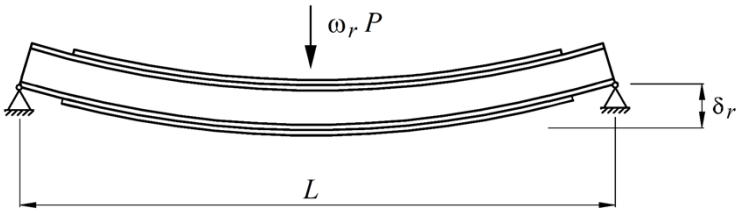
Beam Deflection

Deflection of reinforced member: only loads applied after reinforcement is added.



ω_r = proportion of load applied after the member is reinforced

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Beam Deflection

Total Deflection: $\delta = \delta_n + \delta_r$

δ_n = deflection of non-reinforced member using loads applied before the member is reinforced

δ_r = deflection of reinforced member using loads applied after the member is reinforced

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Beam Deflection

The adjusted deflection can be calculated using the value from a finite element model

$$\delta = \delta_m \left[\omega_r + \omega_n \left(\frac{I_r}{I_n} \right) \right]$$

δ_m = total deflection from the finite element model using the reinforced member properties

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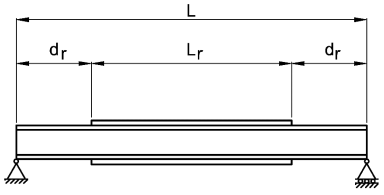
Beam Deflection

I_n = moment of inertia of non-reinforced member

I_r = moment of inertia of reinforced member



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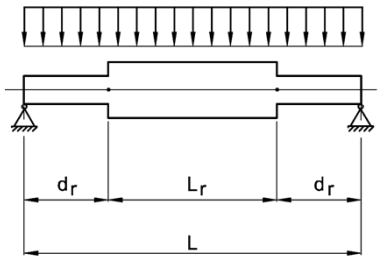
Beam Deflection

Deflection of Beams with Partial-Length Reinforcement



Finite Element Model

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SECTION PROPERTIES OF BUILT-UP MEMBERS

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AISC *Specification*

AISC *Specification* Section F4

$$F_{cr} = \frac{C_b \pi^2 E}{\left(L_b / r_t\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t}\right)^2}$$

Equations F4-5 and F4-8 require the calculation of J

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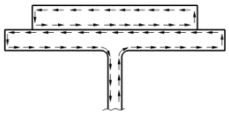
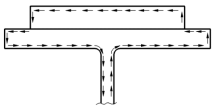
Torsion Constant

The torsion constant, J, can be calculated based on individual element behavior or integral element behavior



individual behavior



integral behavior

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Torsion Constant

Individual Element Behavior (1 Plate)

$$J_s = J_w + J_p \qquad J_p = \frac{1}{3} b_p t_p^3$$

J_w = torsion constant of the W shape
J_p = torsion constant of the plate
t_p = plate thickness
b_p = plate width

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Torsion Constant

Individual Element Behavior (2 Plates)

$$J_s = J_w + J_{p1} + J_{p2}$$

J_{p1} = torsion constant of Plate 1
J_{p2} = torsion constant of Plate 2

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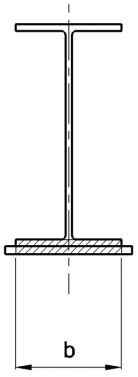
Torsion Constant

Integral Element Behavior (1 Plate)

$$J_i = J_w + J_p + bt_p t_f \left(t_p + t_f \right)$$



b = minimum of b_f and b_p
t_f = flange thickness
b_f = flange width

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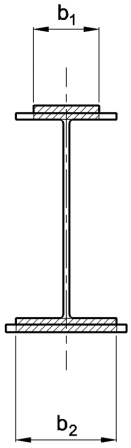
Torsion Constant

Integral Element Behavior (2 Plates)

$$J_i = J_w + J_{p1} + J_{p2} + b_1 t_{p1} t_f \left(t_{p1} + t_f \right) + b_2 t_{p2} t_f \left(t_{p2} + t_f \right)$$



b_1 = minimum of b_f and b_{p1}
b_2 = minimum of b_f and b_{p2}
t_{p1} = thickness of Plate 1
t_{p2} = thickness of Plate 2

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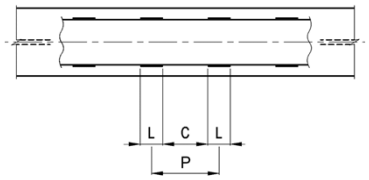
Torsion Constant

Effective torsion constant for members with intermittent welds, J_{eff}

$$J_s < J_{eff} < J_i$$



J_{eff} = effective torsion constant
J_s = torsion constant for independent behavior
J_i = torsion constant for integral behavior

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Torsion Constant

- Generally, commercial software packages provide J_i
- J_s can be used as a conservative estimate of J_{eff}, but this can be extremely conservative
- See Chang and Johnston (1952) for a more accurate solution

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WELD DESIGN

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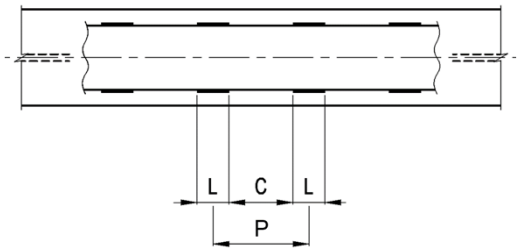
Intermittent Welds

Intermittent Welds

- Lower cost
- Less weld shrinkage distortion
- Greater corrosion potential



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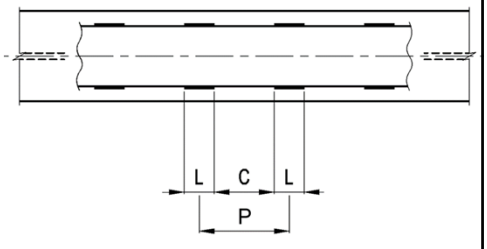
Intermittent Welds

AISC *Specification* Section J2.2b

- $L \geq 4w$
- $L \geq 1.5$ in.



L = segment length
w = weld size

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Intermittent Welds

Maximum weld spacing for intermittent welds in AISC *Specification* Sections D.4 and J3.5

1. Ensure close fit-up over entire faying surface
2. Prevent corrosion between connected elements

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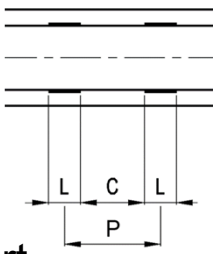
Intermittent Welds

AISC *Specification* Sections D.4 and J3.5

(a) $P_{max} = 24t \leq 12$ in.

(b) Unpainted Weathering Steel:

$P_{max} = 14t \leq 7$ in.



P_{max} = weld spacing

t = thickness of the thinner part

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Welding to Loaded Members

Members Welded Under Load

Elevated temperatures near the weld cause a reduction in material properties of the base material

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Welding to Loaded Members

General Recommendations

- Use small diameter electrodes and low welding current
- Use stringer beads only (not weave beads)
- Specify intermittent welding in short lengths

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Welding to Loaded Members

General Recommendations (continued)

- Where appropriate, specify a maximum interpass temperature of 300 °F
- Allow time for welds to cool between passes
- Use temperature crayons or other suitable means to monitor the temperature of the base metal near the weld

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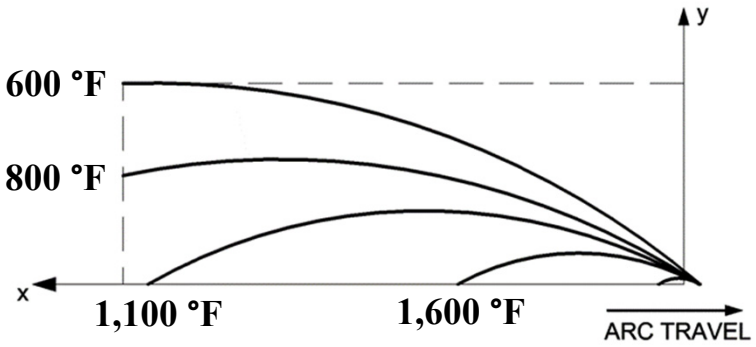
Welding to Loaded Members

Concept of Inactive Areas



Standard Weld Temperature Field

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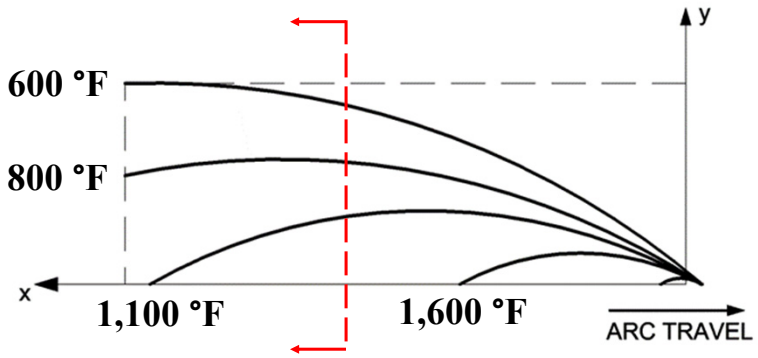
Welding to Loaded Members

Section Normal to Arc Travel



Standard Weld Temperature Field

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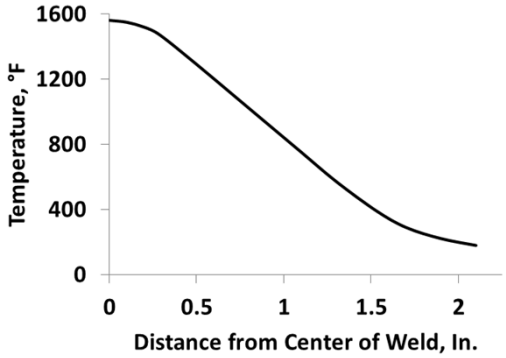
Welding to Loaded Members

Example Temperature Curve for **Low Heat Input Weld**

Section Through Weld Normal to Arc Travel



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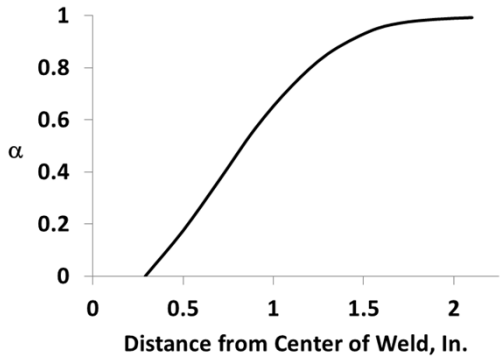
Welding to Loaded Members

Example Yield Strength Curve for **Low Heat Input Weld**

Section Through Weld Normal to Arc Travel

$$F_y' = \alpha F_y$$



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Welding to Loaded Members

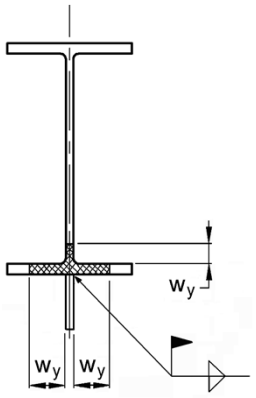
Inactive Area Normal to Arc Travel

Conservative Values:

$w_y = 1$ in. for SMAW

$w_y = 2$ in. for FCAW

See Huenersen et al. (1990) for a more rigorous solution



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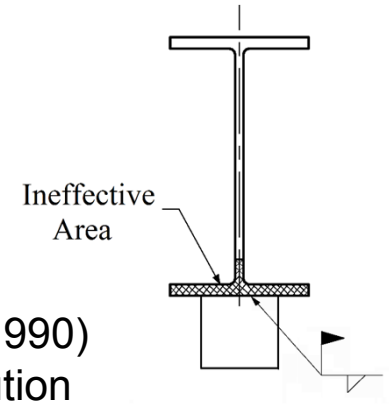
Welding to Loaded Members

Inactive Area Parallel to Arc Travel

Conservative Estimate:

Full flange width plus the portion of the web defined by w_y



See Huenersen et al. (1990) for a more rigorous solution

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BUILT-UP COLUMNS: ADDITIONAL TOPICS

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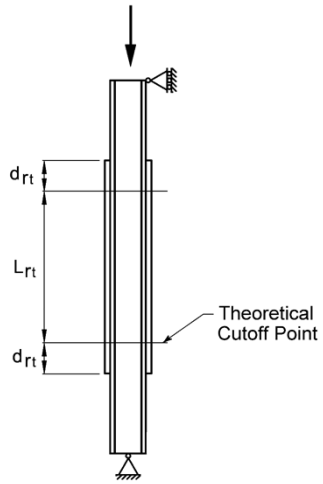
Weld Design for Columns

How can we design the reinforcement welds for columns?

What are the loads?



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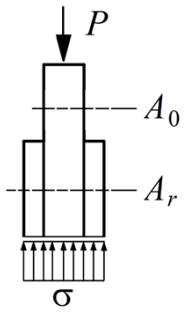
Weld Design for Columns

The load in the cover plate must be transferred to the non-reinforced section at the theoretical cutoff point.

Welds must develop the cover plate's portion of the required axial load in the column



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Weld Design for Columns

The load in each plate, based on equal axial stress over the cross sectional area, is

$$F_p = P\frac{A_p}{A_r}$$

A_p = area of the reinforcing plate
A_r = total area of the reinforced section

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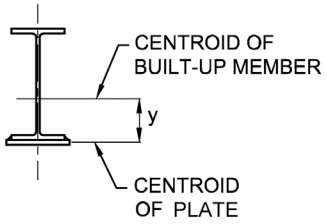
Weld Design for Columns

Shear Flow

$$v = \frac{V_r Q}{I_{rx}} \qquad Q = A_p y$$



A_p = area of the reinforcing plate
I_{rx} = strong-axis moment of inertia of the built-up member
V_r = required shear force
v = shear force per unit length

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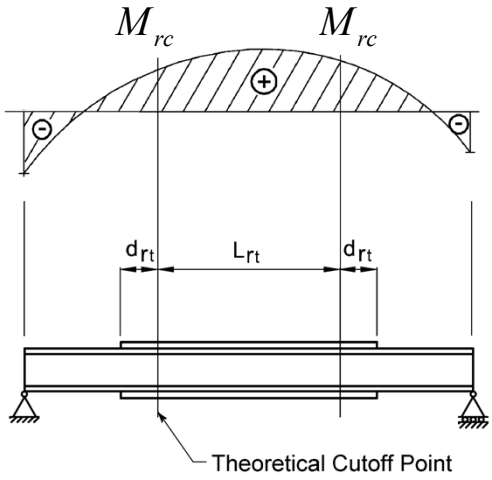
Weld Design for Columns

Anchor Force

$$F_A = \frac{M_{rc} Q}{I_{rx}}$$



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Weld Design for Columns

How can we determine V_r and M_{rc} ?

Apply a uniform notional load to develop a bending moment equal to that caused by the initial out-of-straightness. For $\delta_0 = L/1{,}000$

$$w_n = \frac{P_r}{125L}$$

P_r = required axial load

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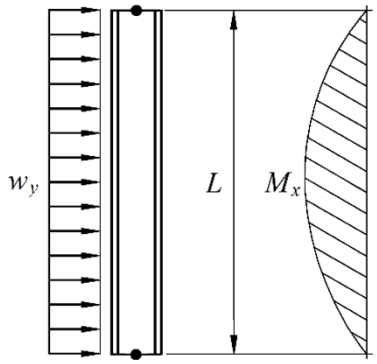
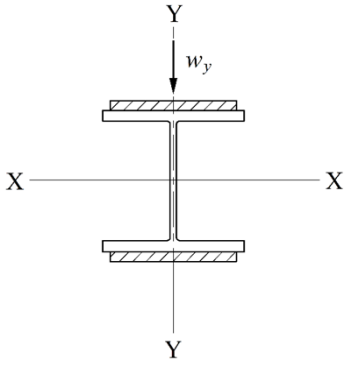
Weld Design for Columns

Apply w_n in the proper direction





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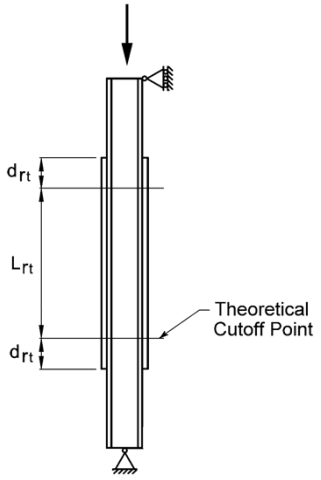
Weld Design for Columns

The first-order moment, M_c, is calculated at the theoretical cutoff point.

If the column is subjected to axial load only, the maximum shear, V_r, is at the theoretical cutoff point.



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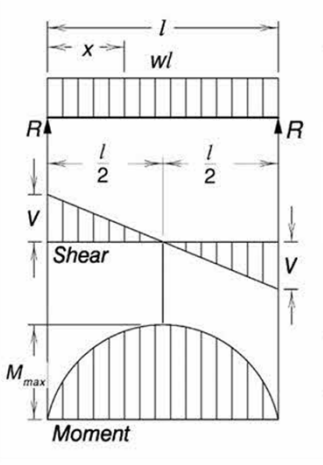
Weld Design for Columns

AISC *Manual* Table 3-23, Case 1.

$$M_x = \frac{wx}{2}(l - x)$$

$$V_x = w\left(\frac{l}{2} - x\right)$$



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Weld Design for Columns

The required moment, M_{rc}, can be determined with a second order analysis or calculated using multiplier, B_1 from AISC *Specification* Appendix 8, Section 8.2.

$$M_{rc} = B_1 M_c \qquad B_1 = \frac{C_m}{1 - \alpha \frac{P_r}{P_{e1}}}$$

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Weld Design for Columns

P_{el} = elastic lateral buckling load of the column

P_r = required axial strength

$C_m = 1.0$

α = 1.00 for LRFD
= 1.60 for ASD

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Weld Design for Columns

The total load in the plate at the theoretical cutoff point is the anchor force for the moment plus the plate's portion of the axial load.

$$F_r = F_p + F_A$$

$$F_p = P\frac{A_p}{A_r} \qquad F_A = \frac{M_{rc}Q}{I_{rx}}$$

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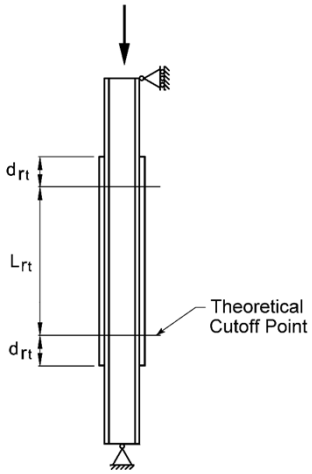
Weld Design for Columns



F_r must be developed over distance, d_{rt}

The shear force per unit length, v, must me developed between the theoretical cutoff points

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FLEXURAL PLATE BUCKLING

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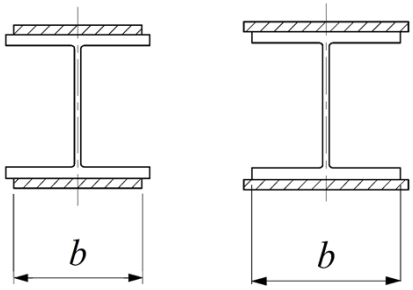
Column Local Buckling

Column Flange Plates

Specification Table B4.1a, Case 7



$$\lambda = \frac{b}{t}$$

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}}$$

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Beam Local Buckling

Beam Compression Flange Plates

Specification Table B4.1b, Case 18

$$\lambda = \frac{b}{t}$$

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}}$$

$$\lambda_p = 1.12\sqrt{\frac{E}{F_y}}$$

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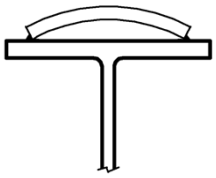
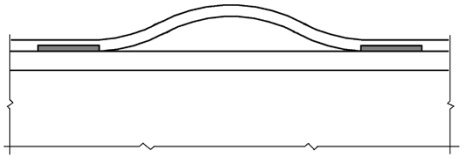
Flexural Plate Buckling

Flexural Buckling Between Intermittent Welds



Local Buckling



Flexural Buckling

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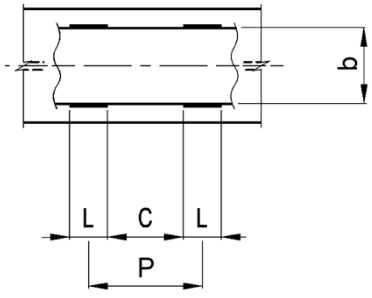
Flexural Plate Buckling

AISC *Specification* Section E6.2

Non-Staggered:

$$P_{max} = 0.75t\sqrt{\frac{E}{F_y}} \le 12 \text{ in.}$$



t = plate thickness

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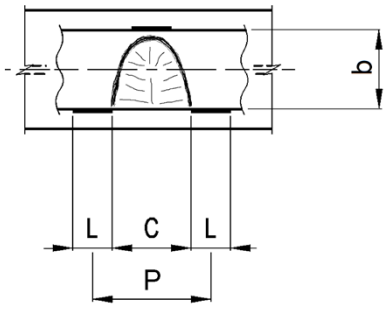
Flexural Plate Buckling

The buckled shape of the plate changes when staggered welds are used



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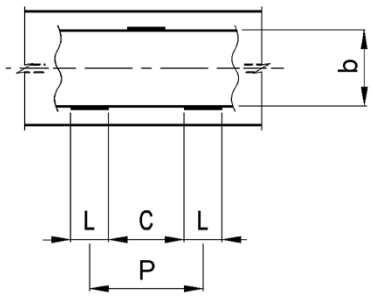
Flexural Plate Buckling

AISC *Specification* Section E6.2

Staggered:

$$P_{max} = 1.12t\sqrt{\frac{E}{F_y}} \le 18 \text{ in.}$$



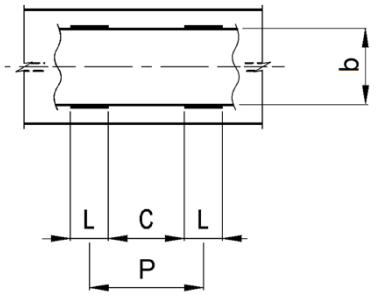
50% increase in P_{max} when staggered

62

Flexural Plate Buckling

Norris et al. (1951) found critical C/b ratios, where local buckling will be the controlling limit state If $C/b < (C/b)_{cr}$.

C = clear distance between fasteners
b = plate width

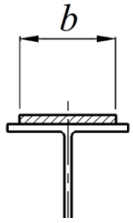


63

Flexural Plate Buckling

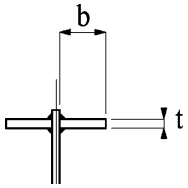
For Stiffened Elements,

$$\left(\frac{C}{b}\right)_{cr} = 0.50$$



For Unstiffened Elements,

$$\left(\frac{C}{b}\right)_{cr} = 1.4$$



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Flexural Plate Buckling

Beam Compression Flange Plates

$\left(\frac{C}{b}\right)_{cr} = 0.50$ For Stiffened Elements

$\lambda_r = \frac{b}{t} = 1.40\sqrt{\frac{E}{F_y}}$ *Specification* Table B4.1b, Case 18

$C_r = 0.5b_r = 0.5t\lambda_r = 0.70t\sqrt{\frac{E}{F_y}}$

65

Flexural Plate Buckling

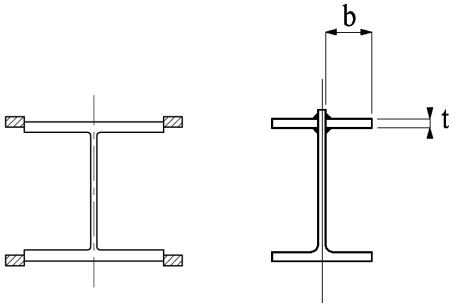
Beam Compression Flange Plates

$\lambda_p = \frac{b}{t} = 1.12\sqrt{\frac{E}{F_y}}$ *Specification* Table B4.1b, Case 18

$C_p = 0.5b_p = 0.5t\lambda_p = 0.56t\sqrt{\frac{E}{F_y}}$

66

Flexural Plate Buckling



For Unstiffened Elements

$$\left(\frac{C}{b}\right)_{cr} = 1.4$$

$C_r = 1.4t\lambda_r$

$C_p = 1.4t\lambda_p$

Select λ_r and λ_p from AISC *Specification* Table B4.1a or B4.1b.

67

COMPOSITE BEAM EXAMPLE

68

Composite Beam Example

An existing composite beam must carry an additional live load of 40 psf. This example shows calculations for the required reinforcement using LRFD.

For the existing beam calculations, see Example I.1 in the AISC Design Examples, Version 14.1.

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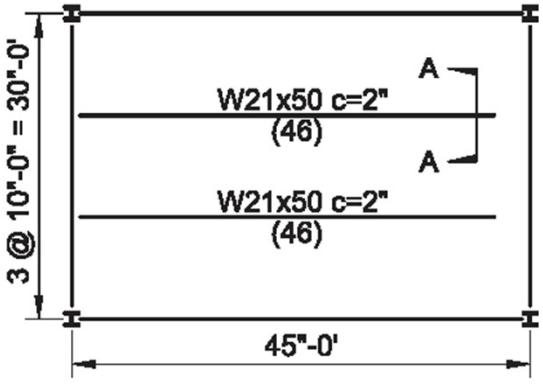
Composite Beam Example



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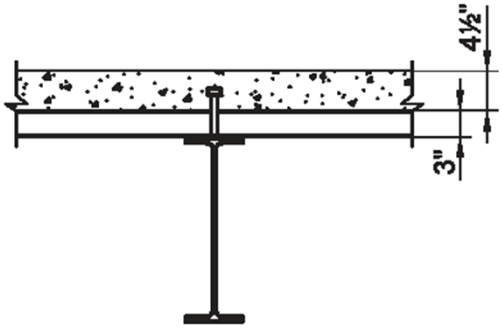
Composite Beam Example



Section A-A

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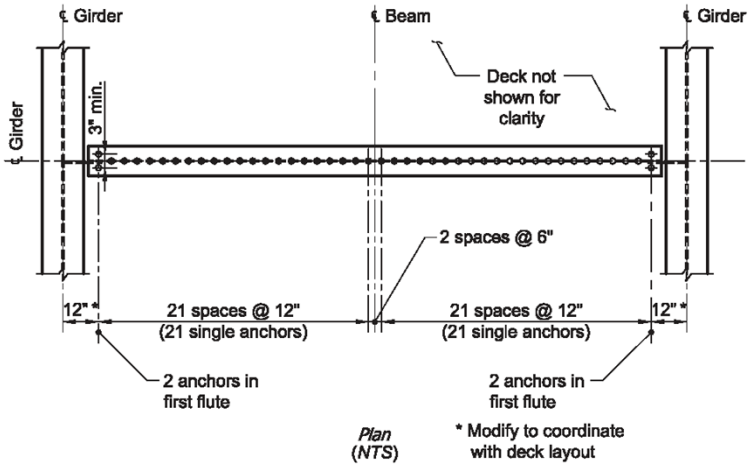
Composite Beam Example



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Composite Beam Example

Given:

The live load increased from 100 psf to 140 psf.

The existing live load will be removed before the beam is reinforced.

The existing beam is A992.

73

Composite Beam Example

For the existing beam:

The available flexural strength,

$$\phi_b M_n = 769 \text{ kip-ft}$$

The lower bound moment of inertia,

$$I_{LB} = 2{,}520 \text{ in.}^4$$

74

Composite Beam Example

The concrete is normal weight with a specified compressive strength, $f_c' = 4$ ksi.

There are 46 ¾-in.-diameter headed stud anchors symmetrically placed about the midspan.

The existing dead load is 0.900 kip/ft

75

Composite Beam Example

Solution:

Existing Beam Material Properties

A992

$F_y = 50$ ksi

$F_u = 65$ ksi

76

Composite Beam Example

Applied Loads

Estimated weight of reinforcement:

20 lbs/ft = 0.02 kip/ft

Dead load:

w_D = 0.900 kip/ft + 0.02 kip/ft = 0.920 kip/ft

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Composite Beam Example

Live load:

w_L = (10.0 ft)(140 psf)(0.001 kip/lb)
= 1.40 kips/ft

Required load:

w_u = (1.2)(0.920 kip/ft) + (1.6)(1.40 kip/ft)
= 3.34 kips/ft

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Composite Beam Example

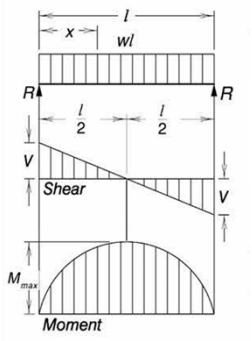
Required Flexural Strength

AISC *Manual* Table 3-23, Case 1.

$$M_u = \frac{w_u L^2}{8}$$

$$= \frac{(3.34 \text{ kip/ft})(45.0 \text{ ft})^2}{8}$$

$$= 845 \text{ kip-ft}$$



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Composite Beam Example

Preliminary Analysis Conclusions

For the existing beam, $\phi_b M_n = 769$ kip-ft

769 kip-ft < 845 kip-ft
(Utilization Ratio = 1.10)

Live load deflection > $L/360$ (18% Over)

The beam must be reinforced

80

Composite Beam Example

Reinforcement Selection Issues

- Flexural reinforcement is most efficient when located farthest from the neutral axis
- Partial-length reinforcement is more economical than full-length reinforcement

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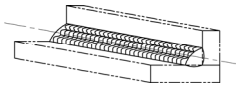
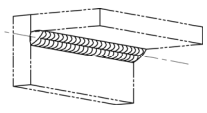
Composite Beam Example

Reinforcement Selection Issues (continued)

- Productivity for horizontal welding is about 4 times that of overhead welding



Horizontal



Overhead

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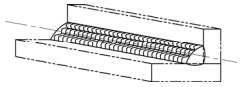
Composite Beam Example

Reinforcement Selection Issues (continued)

- An adequate shelf dimension must be provided for fillet welds



- For this example, the bottom of the beam must remain clear of obstructions

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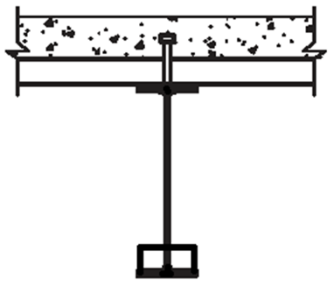
Composite Beam Example

Two L 2 ½ × 2 ½ × 3/8 will be added to the top of the bottom flange. The material will be A36 ($F_y = 36$ ksi, $F_u = 58$ ksi).



L 2 ½ × 2 ½ × 3/8 :

$A_a = 1.73$ in.^{2}
$I_{xa} = 0.972$ in.^{4}
$\overline{y} = 0.758$ in.

84

Composite Beam Example

The total weight of the reinforcement is
(2)(5.90 lb/ft) = 11.8 lb/ft = 0.0118 kip/ft

The revised load is

$w_u = 1.2(0.912 \text{ kip/ft}) + 1.6(1.40 \text{ kip/ft})$
$= 3.33 \text{ kips/ft}$

The revised moment, $M_u = 844$ kip-ft.

85

Composite Beam Example

Available Flexural Strength

AISC *Specification* Section I3.2a: Plastic stress distribution shall be used when

$$\frac{h}{t_w} \le 3.76\sqrt{\frac{E}{F_y}}$$

The stabilizing effect of the reinforcing angles will be neglected.

86

Composite Beam Example

From AISC *Manual* Table 1-1, h/t_w for a W21×50 is 49.4.

$$3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 90.6$$

49.4 < 90.6; therefore, the plastic stress distribution will be used.

87

Composite Beam Example

Concrete Strength

$$A_c = (120 \text{ in.})(4.5 \text{ in.})$$
$$= 540 \text{ in.}^2$$

$$C = 0.85 f_c' A_c$$
$$= (0.85)(4 \text{ ksi})(540 \text{ in.}^2)$$
$$= 1{,}840 \text{ kips}$$

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Composite Beam Example

Steel Strength

$T = \Sigma A_s F_y$

$= (50 \text{ ksi})(14.7 \text{ in.}^2) + (2)\ (36 \text{ ksi})(1.73 \text{ in.}^2)$

$= 860 \text{ kips}$

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Composite Beam Example

Steel Anchor Strength

AISC *Manual* Table 3-21

Assume all anchors are placed in the weak position

1 anchor per rib: $Q_n = 17.2$ kips/anchor

2 anchors per rib: $Q_n = 14.6$ kips/anchor

90

Composite Beam Example

$\Sigma Q_n = (2)(14.6 \text{ kips/anchor})$
$+ (21)(17.2 \text{ kips/anchor})$
$= 390 \text{ kips}$

The compression load in the concrete is

$C = \text{MIN}(1{,}840, 860, 390) = 390 \text{ kips}$

Shear transfer for composite action is controlled by the anchor strength.

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Composite Beam Example

AISC *Specification* Section I3.1a

The effective width of the concrete slab is the minimum of

(1) One eighth of the beam span

$$b_{e1} = (45 \text{ ft.})(2 \text{ sides})/8 = 11.3 \text{ ft}$$

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Composite Beam Example

(2) One-half the distance to the centerline of the adjacent beam

$$b_{e2} = (10 \text{ ft.})(2 \text{ sides})/2 = 10.0 \text{ ft}$$

(3) Distance to the Edge of the slab

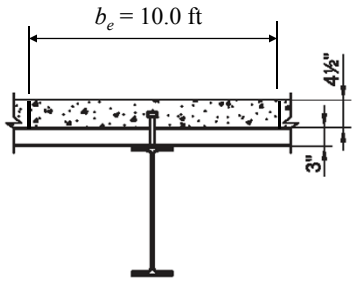
Not Applicable

93

Composite Beam Example

The effective width of slab is

$b_e = b_{e2} = 10.0 \text{ ft} = 120 \text{ in.}$



Section A-A

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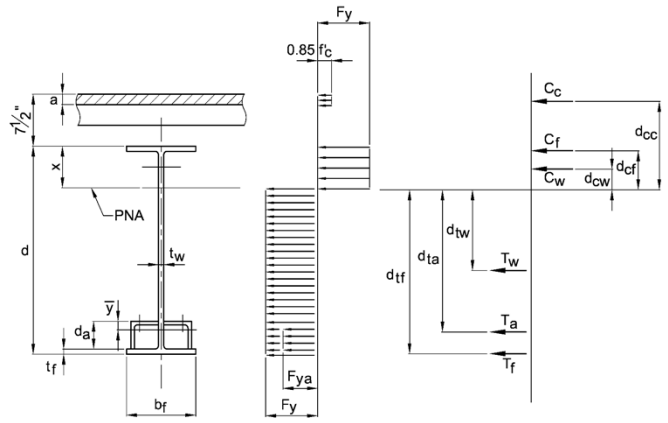
Composite Beam Example

Plastic Analysis of Cross Section



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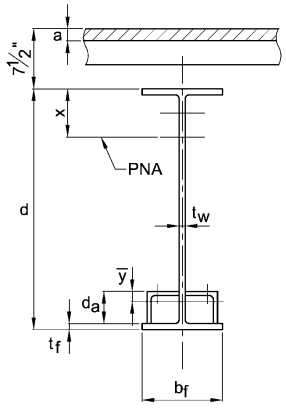
Composite Beam Example

Compression Block Depth



$$a = \frac{C}{0.85 f_c' b}$$

$$= \frac{390 \text{ kips}}{(0.85)(4 \text{ ksi})(120 \text{ in.})}$$

$$= 0.956 \text{ in.}$$

96

Composite Beam Example

$C_c = 390$ kips

$C_f = (50 \text{ ksi})(0.535 \text{ in.})(6.53 \text{ in.}) = 210$ kips

$T_f = (50 \text{ ksi})(0.535 \text{ in.})(6.53 \text{ in.}) = 210$ kips

$T_a = (2 \text{ angles})(36 \text{ ksi})(1.73 \text{ in.}^2) = 125$ kips

$C_w = (50 \text{ ksi})(0.380 \text{ in.})(x - 0.535 \text{ in.})$

$T_w = (50 \text{ ksi})(0.380 \text{ in.})(20.8 \text{ in.} - 0.535 \text{ in.} - x)$

97

Composite Beam Example

The plastic neutral axis (PNA) is located by force equilibrium

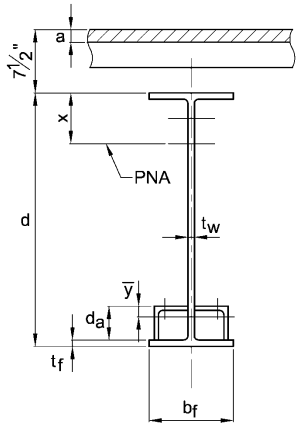
$\Sigma C = \Sigma T$

$C_c + C_f + C_w = T_f + T_a + T_w$

Solve for $x = 3.44$ in.

Also: $C_w = 55.3$ kips

$T_w = 320$ kips



98

Composite Beam Example

d_{cc} = 3.44 in. + 7.50 in. – 0.956 in./2 = 10.5 in.

d_{cf} = 3.44 in. – 0.535 in./2 = 3.17 in.

d_{cw} = (3.44 in. – 0.535 in.)/2 = 1.45 in.

d_{tf} = 20.8 in. – 3.44 in. – 0.535 in./2 = 17.1 in.

99

Composite Beam Example

d_{ta} = 20.8 in. – 3.44 in. – 0.535 in. – 2.50
+ 0.758 in.
= 15.1 in.

d_{tw} = (20.8 in. – 3.44 in. – 0.535 in.)/2 = 8.41 in.

100

Composite Beam Example

M_n = (390 kips)(10.5 in.)
+ (210 kips)(3.17 in.)
+ (55.3 kips)(1.45 in.)
+ (320 kips)(8.41 in.)
+ (125 kips)(15.1 in.)
+ (210 kips)(17.1 in.)
= 13,010 kip-in.
= 1,080 kip-ft.

101

Composite Beam Example

$\phi_b M_n$ = (0.90)(1,080 kip-ft)
= 972 kip-ft

M_u = 844 kip-ft. < 972 kip-ft **o.k.**

102

Composite Beam Example

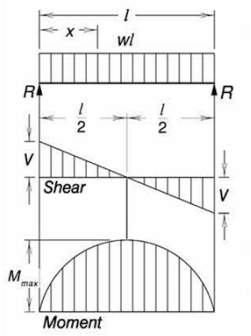
Required Shear Strength

AISC *Manual* Table 3-23, Case 1.

$$V_u = \frac{w_u L}{2}$$

$$= \frac{(3.33 \text{ kips/ft})(45.0 \text{ ft})}{2}$$

$$= 74.9 \text{ kips}$$



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Composite Beam Example

Available Shear Strength

AISC *Specification* Section I4.2

The available shear strength is based on the properties of the steel section alone

AISC *Manual* Table 3-2

$\phi_v V_n = 237 \text{ kips} > 74.9$ **o.k.**

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Composite Beam Example

Location of the Theoretical Cutoff Point

Only partial-length reinforcement will be provided. The location of the theoretical cutoff point must be determined so the flexural strength of the non-reinforced segments will be adequate.

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Composite Beam Example

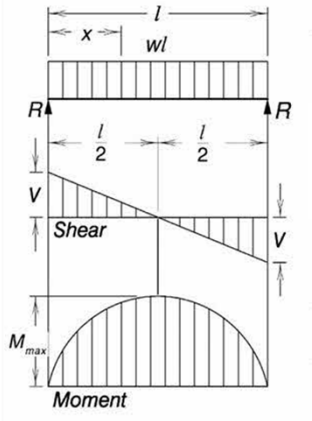
AISC *Manual* Table 3-23, Case 1.

For the existing beam,

$\phi_b M_n = 769$ kip-ft.

$l = 45$ ft

$w_u = 3.33$ kips/ft



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Composite Beam Example

$$M_x = \frac{wx}{2}(l - x)$$

$$769 \text{ kip-ft} = \frac{(3.33 \text{ kips/ft})x}{2}(45.0 \text{ ft} - x)$$

Solve for $x = 15.5$ ft

Use a preliminary distance from the end of the beam to the theoretical cutoff point of 14 ft.

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Composite Beam Example

Live Load Deflection

Live load deflection will be limited to:

1. For 100% of the design live load:
   $\Delta_a = L/360 = (45 \text{ ft})(12 \text{ in./ft})/360 = 1.50$ in.

2. For 50% of the design live load:
   $\Delta_a = 1.0$ in.

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Composite Beam Example

Deflection will be calculated using I_{eff}

AISC *Specification* Commentary Section I3.2

$$I_{eff} = 0.75 I_{equiv}$$

AISC *Specification* Commentary Equation C-I3-4

$$I_{equiv} = I_s + \left(I_{tr} - I_s\right)\sqrt{\frac{\Sigma Q_n}{C_f}}$$

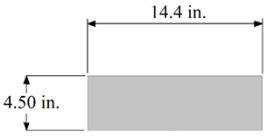
Composite Beam Example

Transformed Moment of Inertia

$\eta = E_s/E_c$

$= (29{,}000 \text{ ksi})/(3{,}490 \text{ ksi})$

$= 8.31$



$b_{tr} = (10 \text{ ft})(12 \text{ in./ft})/8.31$

$= 14.4 \text{ in.}$

$I_{tr} = 5{,}500 \text{ in.}^4$

Composite Beam Example

The moment of inertia of the structural steel section, including the reinforcing angles, $I_s = 1{,}170 \text{ in.}^4$

111

Composite Beam Example

$\Sigma Q_n = 390$ kips

$T = \Sigma A_s F_y = 860$ kips

$C = 0.85 f_c' A_c = 1{,}840$ kips

C_f = compression force in slab for fully composite beam
= minimum of T and C
= 860 kips

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Composite Beam Example

AISC *Specification* Commentary Equation C-I3-4

$$I_{equiv} = I_s + \left(I_{tr} - I_s\right)\sqrt{\frac{\Sigma Q_n}{C_f}}$$

$$= 1{,}170 \text{ in.}^4 + \left(5{,}500 \text{ in.}^4 - 1{,}170 \text{ in.}^4\right)\sqrt{\frac{390 \text{ kips}}{860 \text{ kips}}}$$

$$= 4{,}090 \text{ in.}^4$$

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Composite Beam Example

AISC *Specification* Commentary Section I3.2

$$I_{eff} = 0.75 I_{equiv}$$

$$= (0.75)(4{,}090 \text{ in.}^4)$$

$$= 3{,}070 \text{ in.}^4$$

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Composite Beam Example

For 100% live load

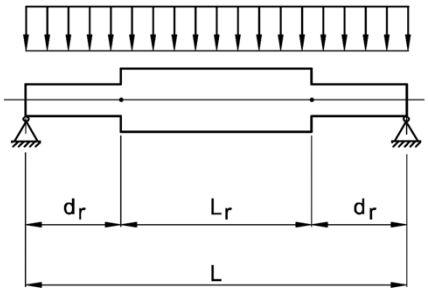
Using a three-member stepped beam finite element model with:

$w_L = 1.40$ kips/ft

$L_r = 18.3$ ft

$d_r = 13.3$ ft

$E = 29{,}000$ ksi



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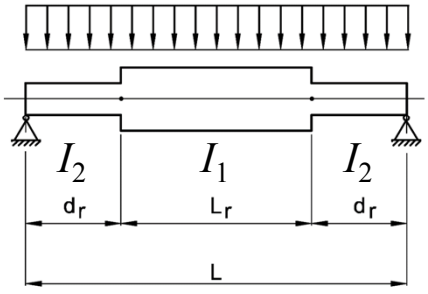
Composite Beam Example

$I_1 = 3{,}070$ in.^{4}

$I_2 = 2{,}520$ in.^{4}



$\Delta_c = 1.58$ in. > 1.50 **n.g.**

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Composite Beam Example

Options to Limit Deflection

- Calculate I using a more rigorous approach
- Increase I_1 by changing the angle size
- Increase the length of reinforcement, L_r

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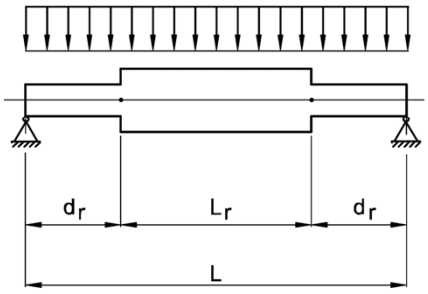
Composite Beam Example

Using the three-member stepped beam finite element model with:

$L_r = 39$ ft

$d_r = 3$ ft



$\Delta_c = 1.48$ in. < 1.50 **o.k.**

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Composite Beam Example

For 50% live load

Using the three-member stepped beam finite element model with:

$w_L = 0.70$ kips/ft

$L_r = 39$ ft

$d_r = 3$ ft

$\Delta_c = 0.73$ in. < 1.0 **o.k.**

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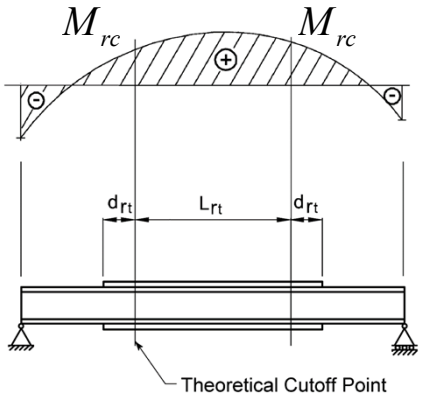
Composite Beam Example

Welds at the End of the Reinforcement



120

Composite Beam Example

The anchor force is

$$F_A = \frac{M_{rc} Q}{I_{rx}}$$

However, this equation is valid only in the elastic range, where

$$M_r \le \phi M_y \qquad M_y = \phi F_y S_x$$

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Composite Beam Example

Due to the approximate nature of the moment of inertia calculations, the weld will be designed to develop the available strength of the angles.

$$\phi_b P_n = (0.9)(36 \text{ ksi})(1.73 \text{ in.}^2) = 56.0 \text{ kips}$$

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Composite Beam Example

Use two 3/16 in. fillet welds extending 8 in. beyond the theoretical cutoff point. For concentrically loaded weld groups, AISC *Manual* Equation 8-2a is applicable.

$$\phi_w R_n = 1.39Dl$$
$$= (1.39)(3)[(2 \text{ welds})(8 \text{ in.})]$$
$$= 66.7 \text{ kips} > 56.0 \text{ kips} \quad \textbf{o.k.}$$

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Composite Beam Example

The beam web bust be thick enough to transmit the load from the angles from the beam. AISC *Manual* Equation 9-3 is applicable.

$$t_{min} = \frac{6.19D}{F_u} = \frac{(6.19)(3)}{65 \text{ ksi}} = 0.286 \text{ in.}$$

0.286 in. < 0.380 in. **o.k.**

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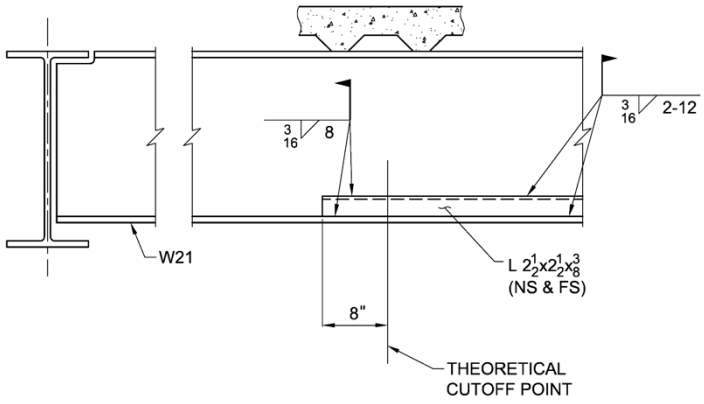
Composite Beam Example

Final Weld Design



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Questions?

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