Basic Steel Design -- Session 8:
Composite Flexural Members

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Smarter.
Stronger.
Steel.

AISC Night School

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Session Description

22.8 Composite Flexural Members
March 31, 2020

This session examines the principles of composite action. The lecture will review elastic vs. plastic behavior and the influence of steel deck on composite members. The session will discuss the design of composite beams including determining steel-headed stud anchor strength and placement, deflection, and economy in shape selection. The session will conclude with a review of the encased and filled composite members.

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Learning Objectives:

- Describe the principles of composite action used in the design of buildings.
- List the AISC Specification requirements for the design of composite beam members.
- Describe the design requirements for steel-headed stud anchors and placement, deflection and economy of shape selection in structural steel building design.
- Describe the AISC Specification requirements for the design of encased and filled composite members.

Basic Steel Design: A review of the principles of steel design according to ANSI/AISC 360-16

Night School 22

Lesson 8

Composite Flexural Members

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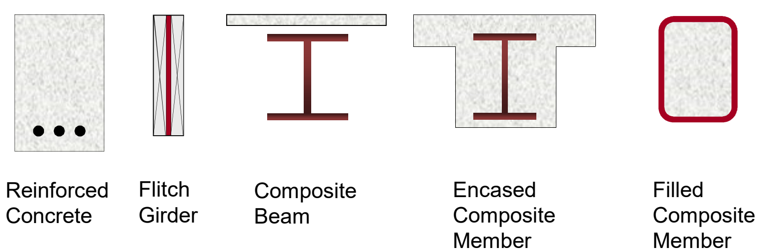
Lesson 8 – Composite Flexural Members

- Composite flexural members
  - Types treated
  - Material limitations
  - Stress distributions
  - Plastic neutral axis locations
  - Strength determination
  - Steel anchors
  - Member design

8.9

Composite Flexural Members

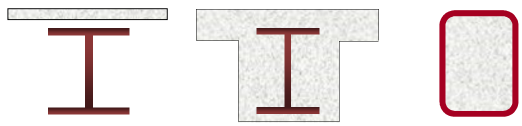
- What are composite members?
  - Combination of dissimilar materials into a single member so that they work together.



8.10

Composite Flexural Members

- These are the composite flexural members that are addressed in the AISC Specification.



Composite Beam

Encased Composite Member

Filled Composite Member

8.11

Composite Flexural Members

I1.3 Material Limitations

- Concrete
  - Normal weight, $3 \text{ ksi} \leq f_c' \leq 10 \text{ ksi}$
  - Light weight, $3 \text{ ksi} \leq f_c' \leq 6 \text{ ksi}$
- Reinforcing steel

$$F_y \leq 80 \text{ ksi}$$

- Steel shapes

$$F_y \leq 75 \text{ ksi}$$

8.12

Composite Flexural Members

I1.2. Nominal strength of composite sections

- Strain Compatibility Method
  - Linear distribution of strain
  - Concrete compressive strain 0.003 in./in.
  - Stress-strain relationships from tests
- Plastic Stress Distribution Method
  - Steel has reached F_y
  - Concrete has reached $0.85f'_c$

    $0.95f'_c$ for filled round HSS

8.13

Composite Flexural Members

I1.2. Nominal strength of composite sections

- Elastic Stress Distribution Method
  - Superposition of elastic stresses
  - Yielding of steel
  - Crushing of concrete
- Effective Stress-Strain Method
  - Strain compatibility
  - Stress-strain relationship
  - Local buckling, yielding, interaction, and concrete confinement

8.14

Composite Flexural Members

- Positive moment strength
  - For compact web W-shapes
    - limit strength of yielding (plastic moment)
    - use plastic stress distribution.
  - For noncompact shapes
    - Limit state of yielding (yield moment)
    - Use elastic stress distribution

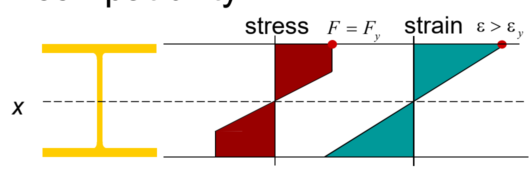
All W-shapes for $F_y \leq 70$ ksi

$\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

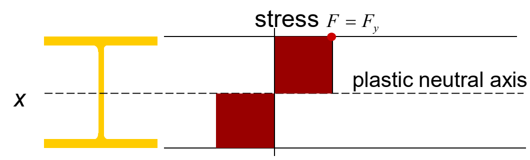
8.15

Composite Flexural Members

- Strength Determination (Lesson 4-steel only)
  - Strain compatibility



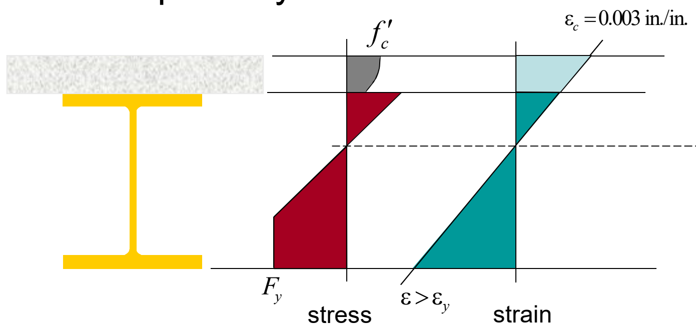
  - Plastic stress distribution



8.16

Composite Flexural Members

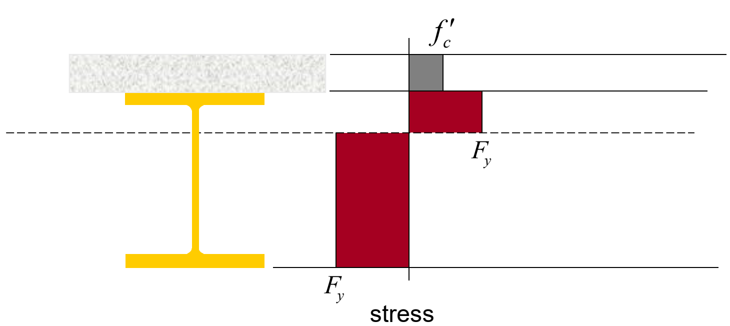
- Strength Determination (composite)
  - Strain compatibility



8.17

Composite Flexural Members

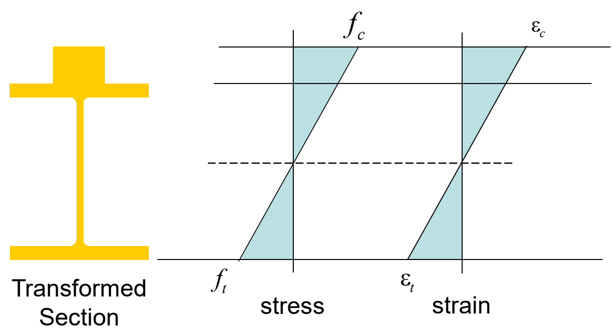
- Strength Determination (composite)
  - Plastic stress distribution



8.18

Composite Flexural Members

- Strength Determination (composite)
  - Elastic



8.19

+M Plastic Stress Distribution

- All rolled W-shapes have compact webs.
- Therefore,
  - Use plastic stress distribution
  - Determine plastic neutral axis location
  - PNA based on force that can be transferred to the concrete

8.20

+M Plastic Stress Distribution

- The concrete force will be the smallest of:

$$V' = 0.85 f'_c A_c \quad \text{(all concrete in compression)}$$

$$V' = A_s F_y \quad \text{(full steel shape in tension)}$$

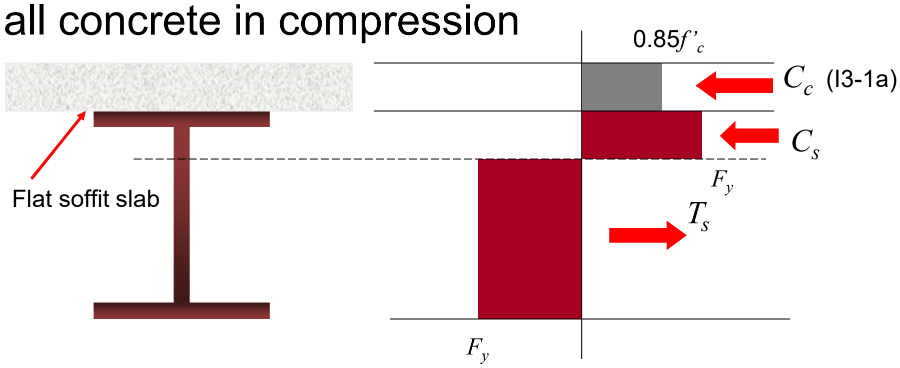
$$V' = \sum Q_n \quad \text{(maximum that can transfer)}$$

(Eqs. I3-1a, I3-1b, and I3-1c)

8.21

+M Plastic Stress Distribution

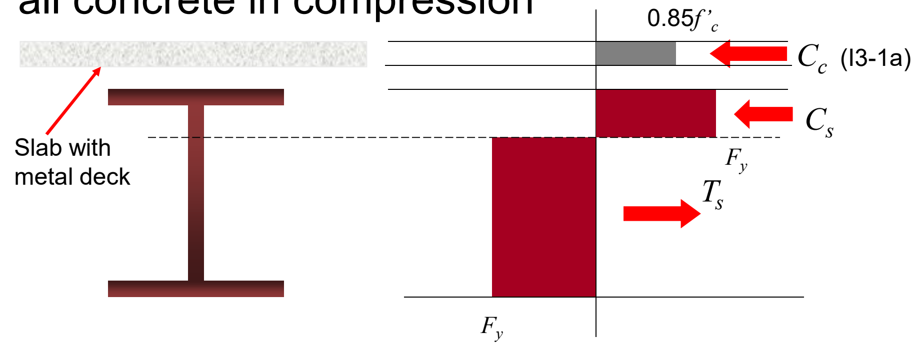
Plastic Neutral Axis is in steel web,
all concrete in compression



8.22

+M Plastic Stress Distribution

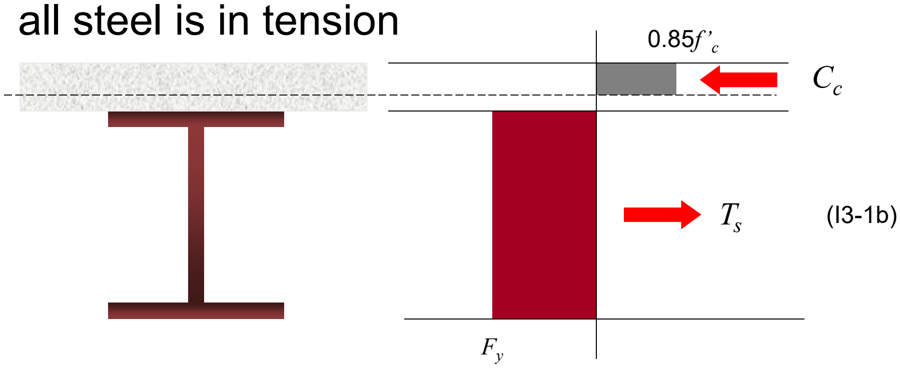
Plastic Neutral Axis is in steel web,
all concrete in compression



8.23

+M Plastic Stress Distribution

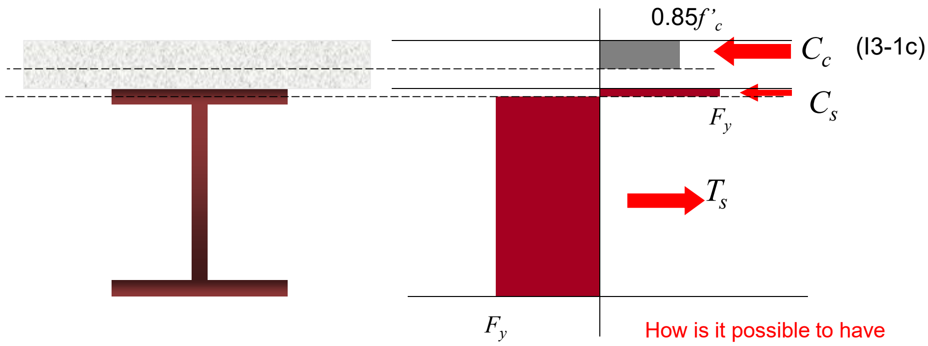
Plastic Neutral Axis is in concrete,
all steel is in tension



8.24

+M Plastic Stress Distribution

Plastic Neutral Axis is in concrete and steel



How is it possible to have two plastic neutral axis locations?

8.25

+M Plastic Stress Distribution

- Once the PNA location has been determined, all element forces may be determined
  - Concrete in compression is stressed to 0.85f'_c
  - Concrete in tension is ignored
  - Steel in compression is stressed to F_y
  - Steel in tension is stressed to F_y

8.26

Composite Beams

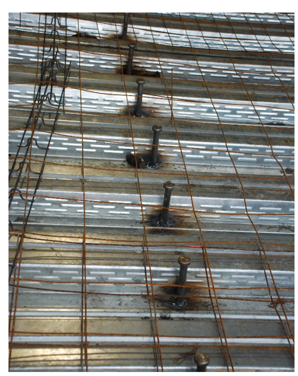
I3. Flexure

- Three types of composite members subject to flexure are addressed:
  - composite beams with steel anchors consisting of steel headed stud anchors or steel channel anchors
  - encased composite members
  - filled composite members

8.27

Composite Beams

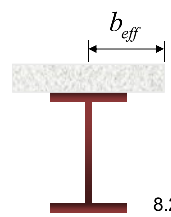
- Steel headed stud anchors (shear studs)



8.28

Composite Beams

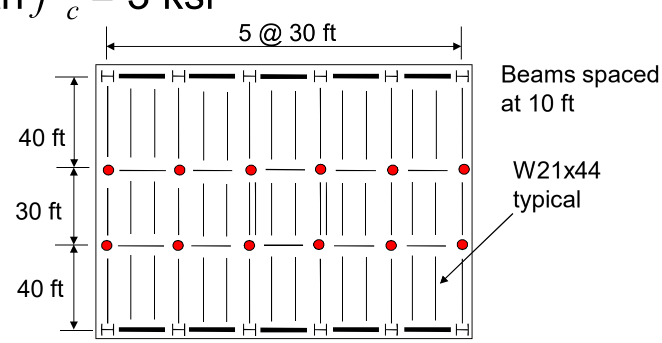
- How much of the concrete works with the steel?
  - The effective width of concrete flange on each side of beam shall not exceed
    - One-eighth of the beam span, *L*
    - One-half the distance to the adjacent beam, *s*
    - The distance to the edge of the slab



8.29

Example 1

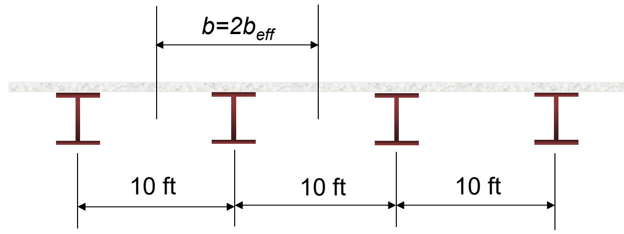
- Determine the strength of a W21x44 A992 beam with a 4.5 in. thick flat soffit concrete slab with f'_c = 3 ksi



8.30

Example 1

- Determine the effective width of the flat soffit concrete flange.



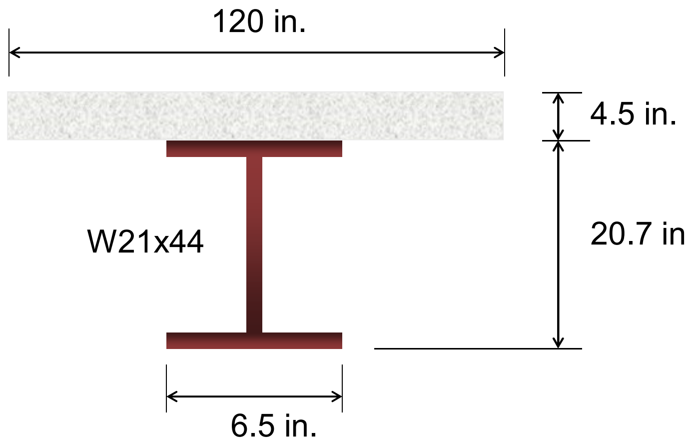
$$b = 2\left(\frac{L}{8}\right) = \frac{L}{4} = \frac{40}{4} = 10 \text{ ft}$$ I3.1a(a)

$$b = 2\left(\frac{s}{2}\right) = s = 10 \text{ ft}$$ I3.1a(b)

8.31

Example 1

- Using this cross section.



8.32

Example 1

- Determine location of PNA

$$C_c = 0.85(3)(120)(4.5) = 1380 \text{ kips}$$
$$T_s = 13.0(50.0) = 650 \text{ kips} \;\star$$
$$C_q = \text{ full composite}$$ (our assumption for this problem)

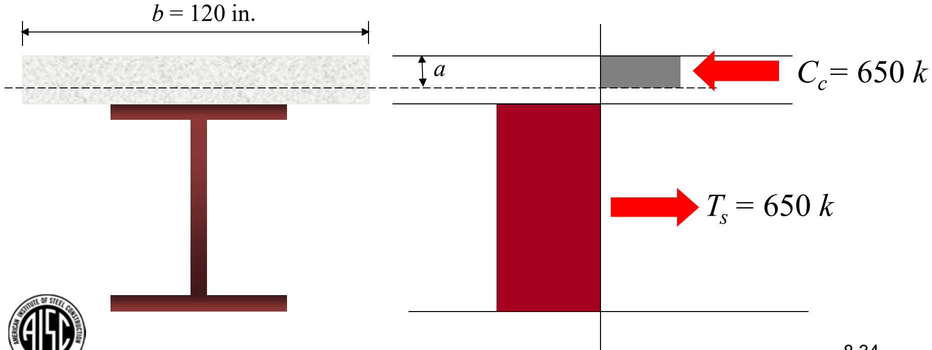
- Thus, the steel controls so the PNA is in the concrete

8.33

Example 1

- Plastic Stress Distribution

$$C_c = T_s = 650 \text{ kips}$$



8.34

Example 1

- Determine effective concrete depth from

$$T = C = 0.85 f_c' ab$$

$$a = \frac{650}{0.85(3)(120)} = 2.12 \text{ in.}$$

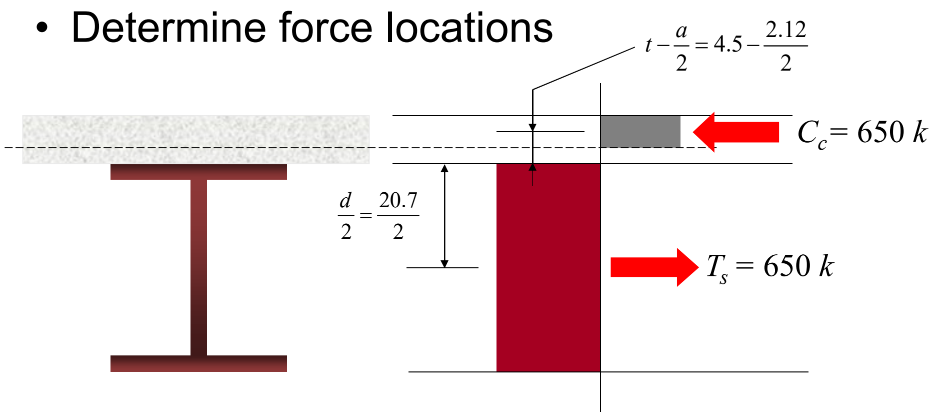
- Nominal moment = plastic moment

$$M_n = M_{pc} = C_c \times \text{ arm} + T_s \times \text{ arm}$$

8.35

Example 1

- Determine force locations



8.36

Example 1

- Plastic Moment Strength: take moments about the top of the steel

$$M_{pc} = 650\left(\frac{20.7}{2} + 4.5 - \frac{2.12}{2}\right) = 8960 \text{ in.-kips}$$

$$M_n = \left(\frac{8960}{12}\right) = 747 \text{ ft-kips}$$

8.37

Example 1

- Allowable Strength (ASD)

$$\frac{M_n}{\Omega_b} = \frac{747}{1.67} = 447 \text{ ft-kips}$$

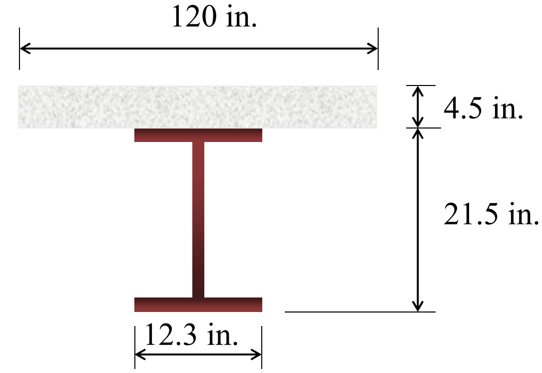
- Design Strength (LRFD)

$$\phi_b M_n = 0.9\left(747\right) = 672 \text{ ft-kips}$$

8.38

Example 2

- Consider the same slab in combination with a larger wide flange. Try a W21x111, A992



8.39

Example 2

- Determine location of PNA.

$$C_c = 0.85(3)(120)(4.5) = 1380 \text{ kips}$$

$$T_s = 32.6(50.0) = 1630 \text{ kips}$$

$$C_q = \text{ full composite}$$

- Thus, the concrete controls so the PNA is in the steel

8.40

Example 2

- With the PNA in the steel, determine its location in the web or the flange

$$T_f = 12.3 \times 0.875(50) = 540$$

$$T_w = 1630 - 2(540) = 550$$

Since

$$C_c > T_f + T_w$$

This means that we need more tension.

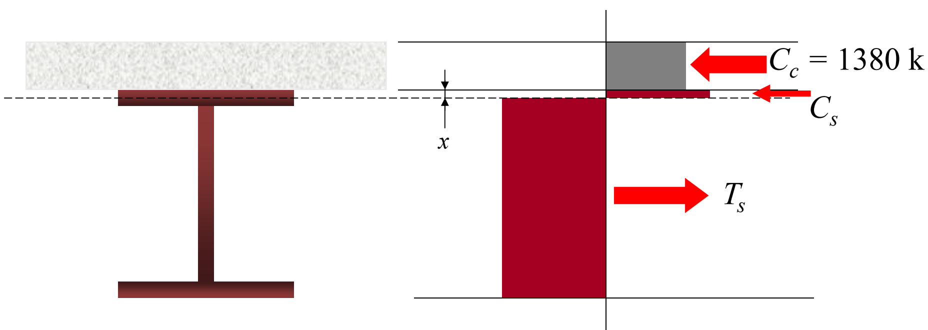
the PNA is in the flange

There are several ways to approach this calculation.

8.41

Example 2

Plastic Stress Distribution



8.42

Example 2

Plastic Neutral Axis Location

$$C = T$$

$$C_c + C_{\text{steel}} = T_{\text{full shape}} - C_{\text{steel}}$$

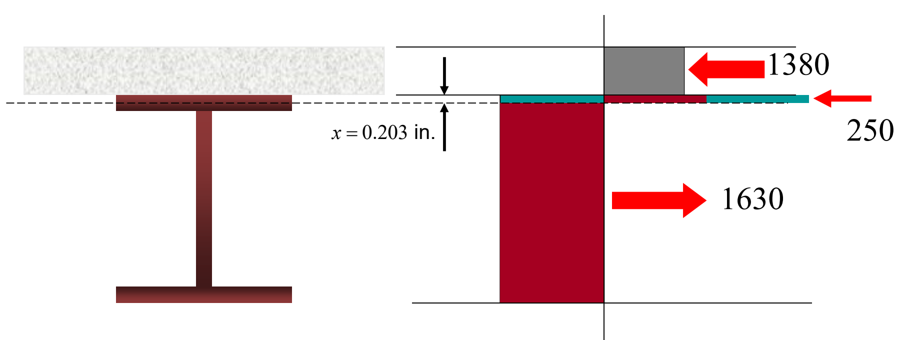
$$1380 + 12.3(x)(50) = 1630 - 12.3(x)(50)$$

$$x = 0.203 \text{ in.}$$

8.43

Example 2

$$C_f = 12.3(0.203)(50) = 125 \text{ kips}$$



We are going to use a math "trick" to simplify our calculations

8.44

Example 2

Plastic Moment Strength: take moments about the top of the steel

$$M_{pc} = 1630\left(\frac{21.5}{2}\right) + 1380\left(\frac{4.5}{2}\right) - 2\left(125\left(\frac{0.203}{2}\right)\right)$$

$$= 20,600 \text{ in.-kips}$$

$$M_n = M_{pc} = \frac{20,600}{12} = 1720 \text{ ft-kips}$$

8.45

Example 2

- Allowable Strength (ASD)

$$\frac{M_n}{\Omega_b} = \frac{1720}{1.67} = 1030 \text{ ft-kips}$$

- Design Strength (LRFD)

$$\phi_b M_n = 0.9(1720) = 1550 \text{ ft-kips}$$

8.46

Partial Composite Action

- Any time the shear studs control, we can not use the full strength of the concrete or of the steel shape.
  - This is referred to as partial composite action.
  - This requires slip between the steel and concrete. (two pna locations)
- If the strength is controlled by the steel shape or the concrete slab
  - It is referred to as full composite action.

8.47

Partial Composite Action

Load transfer between steel beam and concrete slab

- The effect of ductility (slip capacity) of the shear connection at the interface of the concrete slab and steel beam shall be considered.

8.48

Partial Composite Action

- Commentary guidelines
  - If one of these are satisfied, shear connectors are not subject to failure due to insufficient ductility.
    1. Beam span not exceeding 30 ft
    2. Beams with at least 50% composite action
    3. Beams with minimum ¾ in. stud at 12 in. o/c

8.49

Example 3

Determine the available strength of the composite beam from Example 1 if the shear studs are only capable of transferring

$\sum Q_n = 400$ kips (partial composite action)

$$C_c = 0.85(3)(120)(4.5) = 1380 \text{ kips}$$

$$T_s = 13.0(50.0) = 650 \text{ kips}$$

$$C_q = 400 \text{ kips}$$ ☆

$$\% \text{ Composite Action} = \frac{400}{650} \times 100 = 61.5\%$$

8.50

Example 3

Determine the effective depth of concrete from

$$C_c = 0.85 f_c' ab$$

$$a = \frac{400}{0.85(3)(120)} = 1.31 \text{ in.}$$

8.51

Example 3

Area of steel in compression

There can only be 400 kips in tension

$$A_{s-c} = \frac{650 - 400}{2(50)} = 2.5 \text{ in.}^2$$

$$x = \frac{2.5}{b_f} = \frac{2.5}{6.5} = 0.385 < t_f = 0.45 \text{ in.}$$

Therefore the PNA is in the flange

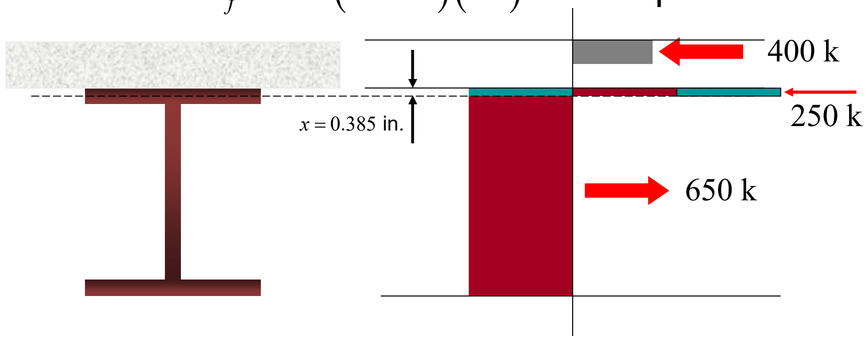
This is a trial and error calculation. If it had come out larger than t_f, it would not be the correct calculation.

8.52

Example 3

Plastic stress distribution

$$C_f = 6.5(0.385)(50) = 125 \text{ kips}$$



Again, we are going to use a math "trick" to simplify our calculations

8.53

Example 3

Plastic moment strength: take moments about the top of the steel

$$M_{pc} = 650\left(\frac{20.7}{2}\right) + 400\left(4.5 - \frac{1.31}{2}\right) - 250\left(\frac{0.385}{2}\right)$$

$$= 8,220 \text{ in.-kips}$$

$$M_n = M_{pc} = \frac{8,220}{12} = 685 \text{ ft-kips}$$

8.54

Example 3

Allowable strength (ASD)

$$\frac{M_n}{\Omega_b} = \frac{685}{1.67} = 410 \text{ ft-kips}$$

Design strength (LRFD)

$$\phi_b M_n = 0.9(685) = 617 \text{ ft-kips}$$

38% reduction in stud strength leads to an 8% reduction in moment strength

8.55

Composite Beams

- I3.1b. Strength during construction
  - With shoring (supported from below)
    - Shoring carries construction loads

  - Without shoring
    - Steel must support all load applied before concrete attains 75% of its specified strength
    - Strength determined according to Chapter F

8.56

Composite Beams

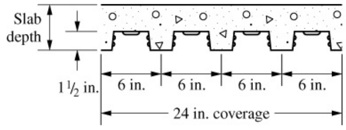
- I3.2c. Composite beams with formed steel deck
  - Ribs perpendicular to steel beam
    - Ignore concrete below top of steel deck in calculating area of concrete

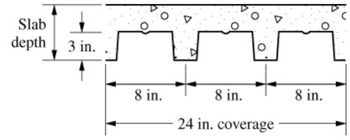
  - Ribs parallel to steel beam
    - Concrete below top of steel deck is included in calculating area of concrete

8.57

Composite Beams

- I3.2c. Composite beams with formed steel deck
  - Limitations
    - Rib height ≤ 3.0 in.
    - Average rib width ≥ 2.0 in.
    - Must use maximum ¾ in. steel headed stud anchors (I8.1)
    - Anchors must extend not less than 1.5 in. above top of deck
    - Slab above deck min 2.0 in.
    - Minimum concreter cover on studs ½ in.
    - Steel deck anchored at a spacing not to exceed 18 in.





8.58

Composite Beams

- I3.3. Encased Composite Members

The nominal flexural strength, M_n, shall be determined using one of the following methods

- Superposition of elastic stresses
- Plastic stress distribution on steel section alone
- Plastic stress distribution or strain compatibility method on composite section. Steel anchors are required.

$\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

8.59

Composite Beams

- I3.4. Filled Composite Members
  - For compact sections, use the plastic stress distribution, M_p.
  - For noncompact sections interpolate between M_p and M_y corresponding to first yield of the compression flange
  - For slender sections limit the compression flange stress to the local buckling stress.

8.60

I8. Steel Anchors

- Two types of anchors
  - Steel Headed Stud Anchors
  - Steel Channel Anchors
- Three applications with different strengths
  - in a solid slab (examples we have looked at)
  - in a slab on metal deck (will address soon)
  - in a composite component (encased or filled)

8.61

I8. Steel Anchors

- I8.2. Steel Anchors in Composite Beams
  - Length of installed stud anchor must be at least 4 times the diameter.
    - Normally use ¾ in. studs, thus minimum length is 3 in.
  - Nominal strength of a single stud anchor in a solid slab or slab with decking

$$Q_n = 0.5 A_{sa}\sqrt{f_c' E_c} \le R_g R_p A_{sa} F_u \quad \text{(I8-1)}$$

8.62

Shear Studs

Nominal stud strength

$$Q_n = 0.5 A_{sa} \sqrt{f'_c E_c} \le R_g R_p A_{sa} F_u \qquad \text{(I8-1)}$$

User Note: The table below presents values for R_g and R_p for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	R_g	R_p
No decking	1.0	0.75

Table 3-21
Shear Stud Anchor
F_u = 65 ksi
Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n, kips

Q_n

Deck Condition	Stud Anchor Diameter, in.	Normal Weight Concrete w_c = 145 pcf, f'_c = 3 ksi	Normal Weight Concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 4 ksi
No Deck	3/8	5.26	5.38	4.28	5.31
	1/2	9.35	9.57	7.60	9.43
	5/8	14.6	15.0	11.9	14.7
	3/4	21.0	21.5	17.1	21.2
	7/8	28.6	29.3	23.3	28.9
	1	37.4	38.3	30.4	37.7

8.63

Example 4

- Select studs for the beam of Example 1
- If ΣQ_n = 650 kips

$$\#\ \text{studs} = \frac{650}{21} = 31 \text{ for half of beam}$$

From point of maximum moment to support.

- Beam span = 40 ft
  - 62 studs over 40 ft means spacing of approximately 7.5 in.

8.64

Example 4

- Select studs for the beam of Example 2
- If ΣQ_n = 1380 kips

$$\#\ \text{studs} = \frac{1380}{21} = 66 \text{ for half of beam}$$

From point of maximum moment to support.

- Beam span = 40 ft
  - 132 studs over 40 ft means spacing of approximately 3.5 in.

8.65

Composite Beams with Metal Deck

- Area taken by the deck can carry no compressive force.
- Direction of deck with respect to composite beam matters.
- Strength of shear studs must be adjusted (reduced) for existence of deck.

8.66

User Note: The table below presents values for R_g and R_p for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	R_g	R_p
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85[a]	0.75
Decking oriented perpendicular to the steel shape Number of steel headed stud anchors occupying the same decking rib:		
1	1.0	0.6[b]
2	0.85	0.6[b]
3 or more	0.7	0.6[b]

h_r = nominal rib height, in. (mm)
w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)
[a] For a single steel headed stud anchor
[b] This value may be increased to 0.75 when $e_{mid\text{-}ht} \geq 2$ in. (50 mm).

- Factors influencing stud strength
  - Rib width/height
  - Number of studs per rib
  - Strong or weak stud location

$$Q_n = 0.5 A_{sa} \sqrt{f'_c E_c} \leq R_g R_p A_{sa} F_u \qquad \text{(I8-1)}$$

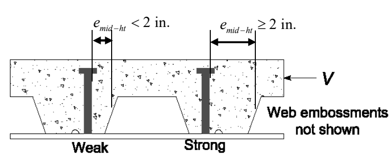
8.67

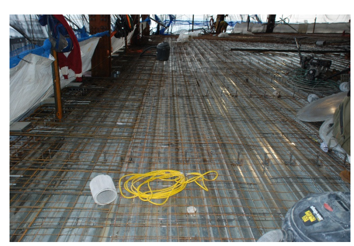
Table 3-21
F_u = 65 ksi
Shear Stud Anchor Q_n
Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n, kips

Deck Condition			Stud Anchor Diameter, in.	Normal Weight Concrete w_c = 145 pcf, f'_c = 3 ksi	Normal Weight Concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 4 ksi
No Deck			3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
			7/8	28.6	29.3	23.3	28.9
			1	37.4	38.3	30.4	37.7
Deck Parallel	$\frac{w_r}{h_r} \geq 1.5$		3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
	$\frac{w_r}{h_r} < 1.5$		3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
Deck Perpendicular	Weak studs per rib (R_p = 0.60)	1	3/8	4.31	4.31	4.28	4.31
			1/2	7.66	7.66	7.60	7.66
			5/8	12.0	12.0	11.9	12.0
			3/4	17.2	17.2	17.1	17.2
		2	3/8	3.66	3.66	3.66	3.66
			1/2	6.51	6.51	6.51	6.51
			5/8	10.2	10.2	10.2	10.2
			3/4	14.6	14.6	14.6	14.6
		3	3/8	3.02	3.02	3.02	3.02
			1/2	5.36	5.36	5.36	5.36
			5/8	8.38	8.38	8.38	8.38
			3/4	12.1	12.1	12.1	12.1
	Strong studs per rib (R_p = 0.75)	1	3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
		2	3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
		3	3/8	3.77	3.77	3.77	3.77
			1/2	6.70	6.70	6.70	6.70
			5/8	10.5	10.5	10.5	10.5
			3/4	15.1	15.1	15.1	15.1

Notes: Tabulated values are applicable only to concrete made with ASTM C33 aggregates for normal weight concrete and ASTM C330 aggregates for lightweight concrete.
After-weld steel headed stud anchor lengths assumed to be ≥ Deck height + 1.5 in.

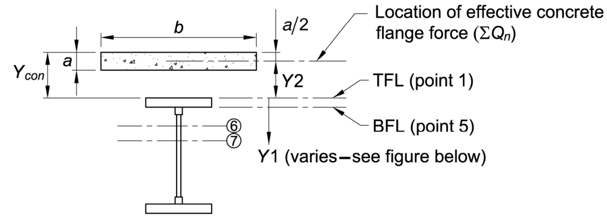
Recommendation:
Always assume studs are located in weak position





8.68

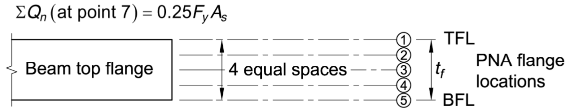
Composite Beam Design Aids



$Y1 =$ Distance from top of steel flange to any of the seven tabulated PNA locations

$$\Sigma Q_n\,(\text{at point } 6) = \frac{\Sigma Q_n\,(\text{at point } 5) + \Sigma Q_n\,(\text{at point } 7)}{2}$$

$$\Sigma Q_n\,(\text{at point } 7) = 0.25 F_y A_s$$



(c)

Fig. 3-3. Strength design models for composite beams.

Manual page 3-14

8.69

Composite Beam Design Aids

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

W18–W16

Full composite action

Shape	$Y2^b$, in. 4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×35	330	496	343	516	356	535	369	554	382	574	394	593	407	612
	317	477	329	494	340	511	351	528	362	545	374	562	385	578
	304	457	314	472	323	486	333	501	343	515	352	530	362	544
	291	437	299	449	307	461	315	473	323	485	331	497	339	510
	277	416	283	426	290	435	296	445	303	455	309	465	316	474
	259	390	264	397	269	404	274	411	279	419	283	426	288	433
	235	353	238	358	241	363	244	367	248	372	251	377	254	382
W16×45	400	601	416	626	433	651	450	676	466	701	483	726	499	751
	380	571	394	592	408	613	422	634	436	655	450	677	464	698

Partial composite action

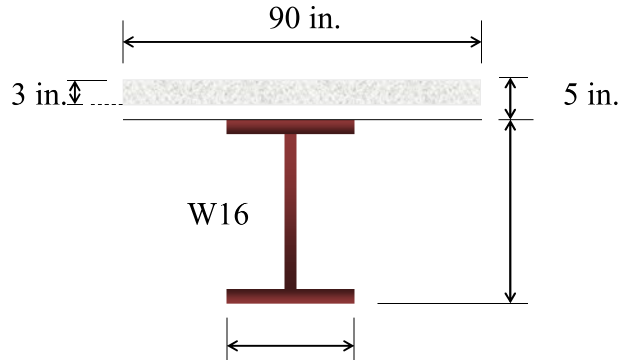
8.70

Example 5 (ASD)

- Design using Table 3-19, select an A992 W-shape for the following conditions:
  - 30 ft x 30 ft panel
  - beams spaced 10 ft on center
  - effective slab width = 90 in. (1/4 span)
  - 5 in. total slab on 2 in. deck
  - f'_c = 3 ksi, normal weight concrete
  - F_y = 50 ksi
  - w_{LL} = 80 psf, w_{DL} = 80 psf

8.71

Example 5 (ASD)



8.72

Example 5 (ASD)

- design load

$$w_a = 80 + 80 = 160 \text{ psf}$$

- Required moment strength (ASD)

$$M_a = \frac{0.160(10)(30)^2}{8} = 180 \text{ ft-kips}$$

8.73

Example 5 (ASD)

- Try a W16 composite beam

  Assume *a* = 1.0 in. (depth of concrete stress block)

- Therefore, location of compression force from top of steel

$$Y2 = 5.0 - \frac{1.0}{2} = 4.5 \text{ in.}$$

- Enter beam selection Table 3-19

8.74

Example 5 (ASD)

Table 3-19 (continued)
Composite W-Shape
Available Strength in Flexu
kip-ft

W16–W14

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$[d]	2		2.5	
	kip-ft			in.	kip	ASD	LRFD	ASD	LRF
	ASD	LRFD							
W16×26	110	166	TFL	0	384	189	284	198	29
			2	0.0863	337	184	276	192	28
			3	0.173	289	179	269	186	28
			4	0.259	242	174	261	180	27
			BFL	0.345	194	168	253	173	26
			6	2.05	145	161	241	164	24
			7	4.01	96.0	148	223	151	22
W14×38	153	231	TFL	0	560	253	380	267	40
			2	0.129	473	244	367	256	38

Table 3-19 (co
Composite W
F_y = 50 ksi
Available Strength
kip-ft

Shape	$Y2$ 4		4.5		5		5
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD
W16×26	227	341	237	356	246	370	256
	218	327	226	340	234	352	243
	208	312	215	323	222	334	229
	198	297	204	306	210	315	216
	188	282	192	289	197	296	202
	175	263	179	268	182	274	186
	158	237	160	241	163	244	165
W14×38	309	464	323	485	337	506	351
	291	438	303	455	315	473	327

For $Y2$ = 4.5 select $Y1$= 2.05

Thus, a partial composite beam

8.75

Example 5 (ASD)

- For this PNA location

$$\sum Q_n = 145 \text{ kips}$$

- Determine actual depth of concrete working

$$a = \frac{145}{0.85(3)(90)} = 0.632 \text{ in.} < \text{ assumed } a = 1.0 \text{ in.}$$

8.76

Example 5 (ASD)

- Determine actual *Y*2

$$Y2 = 5.0 - \frac{0.632}{2} = 4.68 \text{ in.} > 4.5 \text{ in. assumed}$$

- Check table for actual strength

See next slide

$$Y2 = 4.5 \quad \frac{M_n}{\Omega_b} = 179 \text{ ft-kips}$$

$$Y2 = 5.0 \quad \frac{M_n}{\Omega_b} = 182 \text{ ft-kips}$$

8.77

Example 5 (ASD)

Table 3-19 (continued)
Composite W-Shape
Available Strength in Flexu
kip-ft

W16–W14

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$[d]	2		2.5	
	kip-ft			in.	kip				
	ASD	LRFD				ASD	LRFD	ASD	LRF
W16×26	110	166	TFL	0	384	189	284	198	29
			2	0.0863	337	184	276	192	28
			3	0.173	289	179	269	186	28
			4	0.259	242	174	261	180	27
			BFL	0.345	194	168	253	173	26
			6	2.05	145	161	241	164	24
			7	4.01	96.0	148	223	151	22
W14×38	153	231	TFL	0	560	253	380	267	40
			2	0.129	473	244	367	256	38

Table 3-19 (co
Composite W
Available Strength
kip-ft

F_y = 50 ksi

Shape	Y2						
	4		4.5		5		5
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD
W16×26	227	341	237	356	246	370	256
	218	327	226	340	234	352	243
	208	312	215	323	222	334	229
	198	297	204	306	210	315	216
	188	282	192	289	197	296	202
	175	263	179	268	182	274	186
	158	237	160	241	163	244	165
W14×38	309	464	323	485	337	506	351
	291	438	303	455	315	473	327

Interpolate for *Y*2 = 4.68 in.

$$\frac{M_n}{\Omega_b} = 180 \text{ ft-kips}$$

8.78

Example 5 (ASD)

- Check construction strength for bare steel

$$w_a = \text{concrete+beam+construction live load}$$

$$w_a = 0.54 + 0.03 + 0.20 = 0.77 \text{ k/ft}$$

$$M_{a-const} = \frac{0.77(30)^2}{8} = 86.6 \text{ ft-kips}$$

- From Table 3-19

$$\frac{M_n}{\Omega_b} = 110 \text{ ft-kips} > 86.6 \text{ ft-kips}$$

Therefore OK for construction strength

8.79

Example 5 (ASD)

- Check construction dead load deflection

$$\Delta = \frac{5(0.57)(30)^4 1728}{384(29000)(301)} = 1.19 \text{ in.} > \frac{L}{360} = 1.0 \text{ in.}$$

- Thus, if this is our limit, the beam will deflect too much.

8.80

Example 5 (ASD)

- What should be done about deflection?
  - Camber (beam is pre-deformed up)
  - Shore (supports under beam during concrete placement)
  - Use a larger section (in this example likely the best solution since we are only using a small part of composite action)

$$\text{\% Composite Action} = \frac{145}{384} \times 100 = 37.8\%$$

8.81

Example 5 (ASD)

Check live load deflection using lower bound Moment of Inertia

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}
W16–W14

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W16×26 (301)	TFL	0	384	674	712	753	796	840	887	935	985	1040	1090	1150
	2	0.0863	337	649	686	724	763	805	849	894	941	990	1040	1090
	3	0.173	289	621	654	689	726	764	804	846	889	934	980	1030
	4	0.259	242	589	619	651	683	718	754	791	830	871	912	956
	BFL	0.345	194	551	577	604	633	663	694	727	760	795	832	869
	6	2.05	145	505	527	549	572	597	622	649	676	705	734	765
	7	4.01	96.0	450	466	482	499	517	535	555	575	596	617	640
W14×38 (385)	TFL	0	560	844	896	951	1010	1070	1130	1200	1270	1340	1410	1490
	2	0.129	473	805	853	903	956	1010	1070	1130	1190	1260	1330	1400

8.82

Example 5 (ASD)

- Using I_{LB}= 622 in.^{4}

$$\Delta = \frac{5(0.8)(30)^4 1728}{384(29000)(622)} = 0.81 \text{ in.}$$

- Limiting deflection

$$\Delta = \frac{30(12)}{360} = 1.0 \text{ in.} > 0.81 \text{ in.}$$

8.83

Example 5 (ASD)

- Select shear studs
  - Use ¾ in. shear studs
  - Assume one stud per rib
  - Assume studs in weak position

$$\Sigma Q_n = 145 \text{ kips per half span}$$

$$Q_n = 17.2 \text{ kips (from Table 3-21)}$$

$$N = \frac{145}{17.2} = 9$$

  - Therefore 18 studs are required for the beam span

Table 3-21
F_u = 65 ksi
Shear Stud Anchor
Nominal Horizontal Shear Strength
for One Steel Headed Stud Anchor, Q_n, kips
Q_n

Deck Condition			Stud Anchor Diameter, in.	Normal Weight Concrete w_c = 145 pcf		Lightweight Concrete w_c = 110 pcf	
				f'_c = 3 ksi	f'_c = 4 ksi	f'_c = 3 ksi	f'_c = 4 ksi
No Deck			3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
			7/8	28.6	29.3	23.3	28.9
			1	37.4	38.3	30.4	37.7
Deck Parallel	$\frac{w_r}{h_r} \geq 1.5$		3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
	$\frac{w_r}{h_r} < 1.5$		3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
Deck Perpendicular	Weak studs per rib (R_p = 0.60)	1	3/8	4.31	4.31	4.28	4.31
			1/2	7.66	7.66	7.60	7.66
			5/8	12.0	12.0	11.9	12.0
			3/4	17.2	17.2	17.1	17.2
		2	3/8	3.66	3.66	3.66	3.66
			1/2	6.51	6.51	6.51	6.51
			5/8	10.2	10.2	10.2	10.2
			3/4	14.6	14.6	14.6	14.6
		3	3/8	3.02	3.02	3.02	3.02
			1/2	5.36	5.36	5.36	5.36
			5/8	8.38	8.38	8.38	8.38
			3/4	12.1	12.1	12.1	12.1
	Strong studs per rib (R_p = 0.75)	1	3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
		2	3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
		3	3/8	3.77	3.77	3.77	3.77
			1/2	6.70	6.70	6.70	6.70
			5/8	10.5	10.5	10.5	10.5
			3/4	15.1	15.1	15.1	15.1

Notes: Tabulated values are applicable only to concrete made with ASTM C33 aggregates for normal weight concrete and ASTM C330 aggregates for lightweight concrete.
After-weld steel headed stud anchor lengths assumed to be ≥ Deck height + 1.5 in.

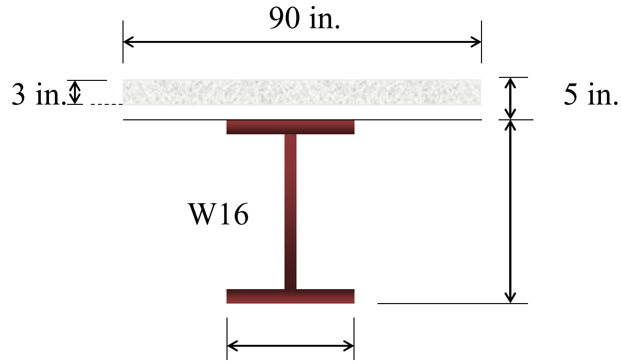
8.84

Example 5 (LRFD)

- Design using Table 3-19, select an A992 W-shape for the following conditions:
  - 30 ft x 30 ft panel
  - beams spaced 10 ft on center
  - effective slab width = 90 in. (1/4 span)
  - 5 in. total slab on 2 in. deck
  - f'_c = 3 ksi, normal weight concrete
  - F_y = 50 ksi
  - w_{LL} = 80 psf, w_{DL} = 80 psf

8.85

Example 5 (LRFD)



8.86

Example 5 (LRFD)

- design load

$$w_u = 1.2(80) + 1.6(80) = 224 \text{ psf}$$

- Required moment strength (LRFD)

$$M_u = \frac{0.224(10)(30)^2}{8} = 252 \text{ ft-kips}$$

8.87

Example 5 (LRFD)

- Try a W16 composite beam
  Assume *a* = 1.0 in.
- Therefore, location of compression force from top of steel

$$Y2 = 5.0 - \frac{1.0}{2} = 4.5 \text{ in.}$$

- Enter beam selection Table 3-19

8.88

Example 5 (LRFD)

W16–W14

Table 3-19 (continued)
Composite W-Shape
Available Strength in Flexu
kip-ft

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	$\sum Q_n$[d]				
	kip-ft					2		2.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRF
W16×26	110	166	TFL	0	384	189	284	198	29
			2	0.0863	337	184	276	192	28
			3	0.173	289	179	269	186	28
			4	0.259	242	174	261	180	27
			BFL	0.345	194	168	253	173	26
			6	2.05	145	161	241	164	24
			7	4.01	96.0	148	223	151	22
W14×38	153	231	TFL	0	560	253	380	267	40
			2	0.129	473	244	367	256	38

F_y = 50 ksi

Table 3-19 (co
Composite W
Available Strength
kip-ft

Shape	$Y2$						
	4		4.5		5		5
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD
W16×26	227	341	237	356	246	370	256
	218	327	226	340	234	352	243
	208	312	215	323	222	334	229
	198	297	204	306	210	315	216
	188	282	192	289	197	296	202
	175	263	179	268	182	274	186
	158	237	160	241	163	244	165
W14×38	309	464	323	485	337	506	351
	291	438	303	455	315	473	327

For $Y2$ = 4.5 select $Y1$= 2.05

Thus, a partial composite beam

8.89

Example 5 (LRFD)

- For this PNA location,

$$\sum Q_n = 145 \text{ kips}$$

- Determine actual depth of concrete working

$$a = \frac{145}{0.85(3)(90)} = 0.632 \text{ in.} < \text{ assumed } a = 1.0 \text{ in.}$$

8.90

Example 5 (LRFD)

- Determine actual Y2

$$Y2 = 5.0 - \frac{0.632}{2} = 4.68 \text{ in.} > 4.5 \text{ in. assumed}$$

- Check table for capacity

See next slide

$$Y2 = 4.5 \quad \phi_b M_n = 268 \text{ ft-kips}$$
$$Y2 = 5.0 \quad \phi_b M_n = 274 \text{ ft-kips}$$

8.91

Example 5 (LRFD)

Table 3-19 (continued)
Composite W-Shape
Available Strength in Flexu
kip-ft

W16–W14

Shape	M_p/Ω_b kip-ft ASD	$\phi_b M_p$ kip-ft LRFD	PNA[c]	Y1[a] in.	ΣQ_n[d] kip	2 ASD	2 LRFD	2.5 ASD	2.5 LRF
W16×26	110	166	TFL	0	384	189	284	198	29
			2	0.0863	337	184	276	192	28
			3	0.173	289	179	269	186	28
			4	0.259	242	174	261	180	27
			BFL	0.345	194	168	253	173	26
			6	2.05	145	161	241	164	24
			7	4.01	96.0	148	223	151	22
W14×38	153	231	TFL	0	560	253	380	267	40
			2	0.129	473	244	367	256	38

Table 3-19 (co
Composite W
Available Strength
ip-ft

F_y = 50 ksi

Shape	Y2 4 ASD	4 LRFD	4.5 ASD	4.5 LRFD	5 ASD	5 LRFD	5 ASD
W16×26	227	341	237	356	246	370	256
	218	327	226	340	234	352	243
	208	312	215	323	222	334	229
	198	297	204	306	210	315	216
	188	282	192	289	197	296	202
	175	263	179	268	182	274	186
	158	237	160	241	163	244	165
W14×38	309	464	323	485	337	506	351
	291	438	303	455	315	473	327

Interpolate for $Y2$ = 4.68 in. $\phi M_n = 271$ ft-kips

8.92

Example 5 (LRFD)

- Check construction strength bare steel

$$w_u = \text{concrete+steel+construction live load}$$

$$w_u = 1.2(0.54) + 1.2(0.03) + 1.6(0.20) = 1.0 \text{ k/ft}$$

$$M_{u-const} = \frac{1.0(30)^2}{8} = 113 \text{ ft-kips}$$

- From Table 3-19

$$\phi_b M_n = 166 \text{ ft-kips} > 113 \text{ ft-kips}$$

8.93

Example 5 (LRFD)

- Check construction dead load deflection

$$\Delta = \frac{5(0.57)(30)^4 1728}{384(29000)(301)} = 1.19 \text{ in.} > \frac{L}{360} = 1.0 \text{ in.}$$

- Thus, if this is our limit, the beam will deflect too much.

8.94

Example 5 (LRFD)

- What should be done about deflection?
  - Camber (beam is pre-deformed up)
  - Shore (supports under beam during concrete placement)
  - Use a larger section (in this example likely the best solution since we are only using a small part of composite action)

$$\text{\% Composite Action} = \frac{145}{384} \times 100 = 37.8\%$$

8.95

Example 5 (LRFD)

Check live load deflection using lower bound Moment of Inertia

Table 3-20 (continued)

Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB} W16–W14

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W16×26 (301)	TFL	0	384	674	712	753	796	840	887	935	985	1040	1090	1150
	2	0.0863	337	649	686	724	763	805	849	894	941	990	1040	1090
	3	0.173	289	621	654	689	726	764	804	846	889	934	980	1030
	4	0.259	242	589	619	651	683	718	754	791	830	871	912	956
	BFL	0.345	194	551	577	604	633	663	694	727	760	795	832	869
	6	2.05	145	505	527	549	572	597	622	649	676	705	734	765
	7	4.01	96.0	450	466	482	499	517	535	555	575	596	617	640
W14×38 (385)	TFL	0	560	844	896	951	1010	1070	1130	1200	1270	1340	1410	1490
	2	0.129	473	805	853	903	956	1010	1070	1130	1190	1260	1330	1400

8.96

Example 5 (LRFD)

- Using I_{LB}= 622 in.^{4}

$$\Delta = \frac{5(0.8)(30)^4 1728}{384(29000)(622)} = 0.81 \text{ in.}$$

- Limiting deflection

$$\Delta = \frac{30(12)}{360} = 1.0 \text{ in.} > 0.81 \text{ in.}$$

8.97

Example 5 (LRFD)

- Select shear studs
  - Use ¾ in. shear studs
  - Assume one stud per rib
  - Assume studs in weak position

$$\sum Q_n = 145 \text{ kips per half span}$$

$$Q_n = 17.2 \text{ kips (from Table 3-21)}$$

$$N = \frac{145}{17.2} = 9$$

  - Therefore 18 studs are required for the beam span

Table 3-21

F_u = 65 ksi

Shear Stud Anchor Q_n

Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n, kips

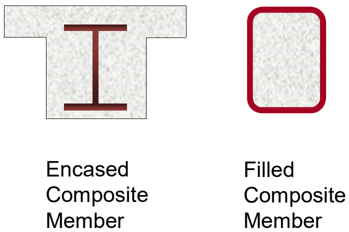
Deck Condition	Stud Anchor Diameter, in.	Normal Weight Concrete w_c = 145 pcf, f'_c = 3 ksi	Normal Weight Concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 4 ksi
No Deck	3/8	5.26	5.38	4.28	5.31
	1/2	9.35	9.57	7.60	9.43
	5/8	14.6	15.0	11.9	14.7
	3/4	21.0	21.5	17.1	21.2
	7/8	28.6	29.3	23.3	28.9
	1	37.4	38.3	30.4	37.7
Deck Parallel, $\frac{w_r}{h_r} \ge 1.5$	3/8	5.26	5.38	4.28	5.31
	1/2	9.35	9.57	7.60	9.43
	5/8	14.6	15.0	11.9	14.7
	3/4	21.0	21.5	17.1	21.2
Deck Parallel, $\frac{w_r}{h_r} < 1.5$	3/8	4.58	4.58	4.28	4.58
	1/2	8.14	8.14	7.60	8.14
	5/8	12.7	12.7	11.9	12.7
	3/4	18.3	18.3	17.1	18.3
Deck Perpendicular, Weak studs per rib (R_p = 0.60), 1	3/8	4.31	4.31	4.28	4.31
	1/2	7.66	7.66	7.60	7.66
	5/8	12.0	12.0	11.9	12.0
	3/4	17.2	17.2	17.1	17.2
Deck Perpendicular, Weak studs per rib (R_p = 0.60), 2	3/8	3.66	3.66	3.66	3.66
	1/2	6.51	6.51	6.51	6.51
	5/8	10.2	10.2	10.2	10.2
	3/4	14.6	14.6	14.6	14.6
Deck Perpendicular, Weak studs per rib (R_p = 0.60), 3	3/8	3.02	3.02	3.02	3.02
	1/2	5.36	5.36	5.36	5.36
	5/8	8.38	8.38	8.38	8.38
	3/4	12.1	12.1	12.1	12.1
Deck Perpendicular, Strong studs per rib (R_p = 0.75), 1	3/8	5.26	5.38	4.28	5.31
	1/2	9.35	9.57	7.60	9.43
	5/8	14.6	15.0	11.9	14.7
	3/4	21.0	21.5	17.1	21.2
Deck Perpendicular, Strong studs per rib (R_p = 0.75), 2	3/8	4.58	4.58	4.28	4.58
	1/2	8.14	8.14	7.60	8.14
	5/8	12.7	12.7	11.9	12.7
	3/4	18.3	18.3	17.1	18.3
Deck Perpendicular, Strong studs per rib (R_p = 0.75), 3	3/8	3.77	3.77	3.77	3.77
	1/2	6.70	6.70	6.70	6.70
	5/8	10.5	10.5	10.5	10.5
	3/4	15.1	15.1	15.1	15.1

Notes: Tabulated values are applicable only to concrete made with ASTM C33 aggregates for normal weight concrete and ASTM C330 aggregates for lightweight concrete.
After-weld steel headed stud anchor lengths assumed to be ≥ Deck height + 1.5 in.

8.98

Composite Components

- Other flexural members



8.99

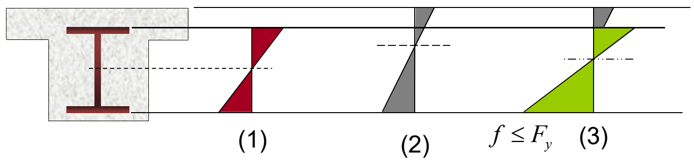
Encased Beam

- Three approaches for strength determination in Section I3.3
  - Superposition of elastic stresses
  - Plastic stress distribution on steel shape alone
  - Plastic stress distribution or strain-compatibility on composite section

8.100

Encased Beam

- Superposition of elastic stresses

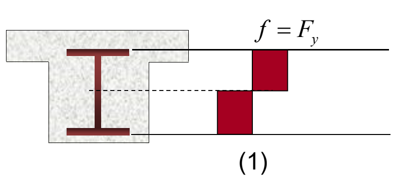


1. Construction load on steel shape alone
2. Additional dead and live load on transformed section
3. Superposition of stresses

8.101

Encased Beam

- Plastic stress distribution on steel shape

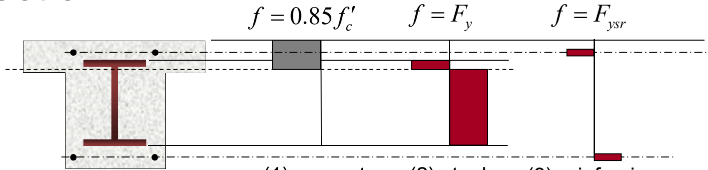


1. Strength M_n = M_p for the steel shape alone
   Lateral-torsional buckling is ignored
   Local buckling is ignored

8.102

Encased Beam

- Plastic stress distribution on composite section



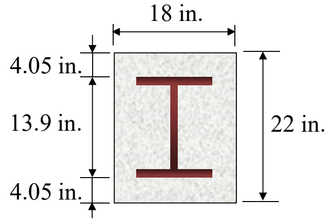
1. Concrete stressed to $0.85f'_c$ in compression, zero in tension
2. Steel stressed to F_y in compression and tension
3. Reinforcing, if provided, stressed to F_{ysr} in tension and compression

8.103

Example 6

- Determine the available strength of a W14x53 encased in 18 in. by 22 in. concrete.

$F_y = 50$ ksi
$f'_c = 5$ ksi
$d = 13.9$ in.
$b_f = 8.06$ in.
$t_w = 0.370$ in.
$t_f = 0.660$ in.
$A_s = 15.6$ in.^{2}

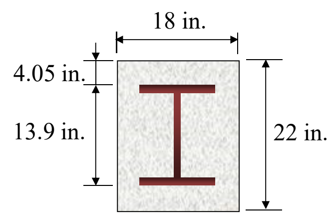


Ignoring required longitudinal reinforcing for our example.

8.104

Example 6

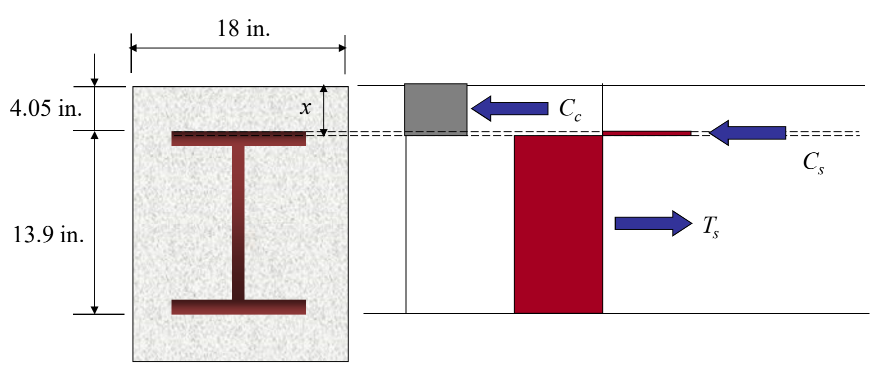
- Locate the plastic neutral axis – 3 choices
  - Above steel shape
  - In steel flange
  - In steel web



8.105

Example 6

- Assume pna is in steel flange as shown



8.106

Example 6

- Forces in steel and concrete

$$C_c = 0.85 f_c'(18x) = 0.85(5)(18x) = 76.5x$$

$$C_s = (50 - 0.85 f_c')(x - 4.05)(8.06) = 368.7x - 1493$$

$$T_s = F_y A_s - F_y(x - 4.05)8.06 = 2412 - 403x$$

- Determine x from T=C

$$T_s = C_c + C_s$$

$$x = 4.60 \text{ in.}$$

Distance to underside of steel flange is cover plus flange thickness.

$$4.05 + t_f = 4.05 + 0.660$$

$$= 4.71 \text{ in.}$$

8.107

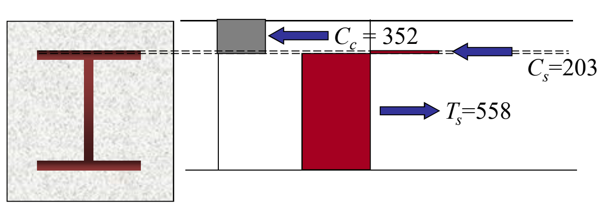
Example 6

- Corresponding forces

$$C_c = 76.5x = 76.5(4.60) = 352 \text{ kips}$$

$$C_s = 368.7x - 1493 = 368.7(4.60) - 1493 = 203 \text{ kips}$$

$$T_s = 2412 - 403x = 2412 - 403(4.60) = 558 \text{ kips}$$

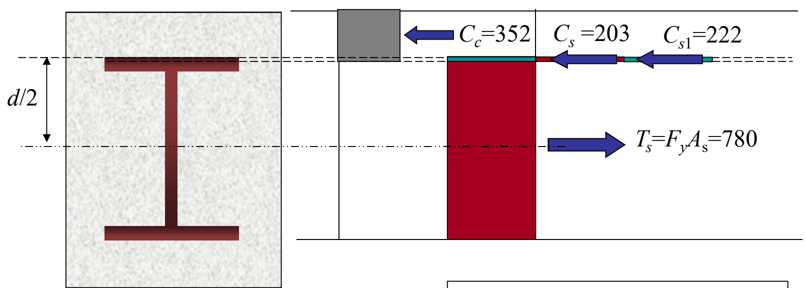


8.108

Example 6

- Take moments about mid-height

$C_{s1} = (4.60 - 4.05) b_f F_y = 0.55(8.06)(50) = 222$ kips



Remember the math "trick" to simplify taking moments?

8.109

Example 6

- Take moments about mid-height

$$M_n = 352\left(11.0 - \frac{4.60}{2}\right) + (203 + 222)\left(\frac{13.9}{2} - \frac{4.60 - 4.05}{2}\right)$$

$= 5898$ in.-kips $\rightarrow 492$ ft-kips

- For ASD $\frac{M_n}{\Omega} = \frac{492}{1.67} = 295$ ft-kips

- For LRFD $\phi M_n = 0.9(492) = 442$ ft-kips

8.110

Example 6

- Steel headed stud anchors are required in this composite component
- Section I8.3a, for ¾ in. studs with F_u = 65 ksi

$$Q_{nv} = F_u A_{sa} = 65(0.442) = 28.7 \text{ kips}$$

$$\phi_v = 0.65 \qquad \Omega_v = 2.31$$

8.111

Example 6

- Since ϕ and Ω are applied to the stud strength, stud selection is a bit different from composite beams.
- The nominal tensile force in the steel is

$$T_s = 558 \text{ kips}$$

- The available strength is

$$\phi T_s = 0.9(558) = 502 \text{ kips} \qquad \frac{T_s}{\Omega} = \frac{558}{1.67} = 334 \text{ kips}$$

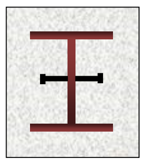
8.112

Example 6

- Required steel headed stud anchors

$$\#\text{ studs} = \frac{\phi T_s}{\phi_v Q_n} = \frac{502}{0.65(28.7)} = 27 \quad \#\text{ studs} = \frac{T_n/\Omega}{Q_n/\Omega_v} = \frac{334}{28.7/2.31} = 27$$

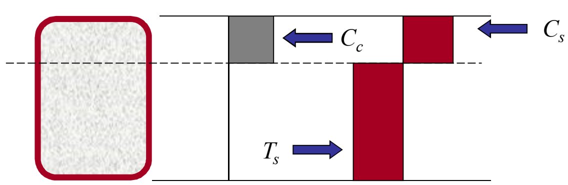
- Place 28 studs between beam mid-span and support, 14 on each side of web.



8.113

Filled Beam

- Section I3.4 provides for compact, noncompact and slender element filled composite members
  - For compact shapes, use the plastic stress distribution on the composite section



8.114

Filled Beam

- Section I3.4 provides for compact, noncompact and slender element filled composite members
  - For noncompact shapes, use a linear variation from plastic moment to yield moment
  - For slender shapes the compression flange stress is limited to the buckling stress and the tension flange stress is limited to the yield stress.

8.115

Summary

- Considered various stress distributions for composite beams
- Determined plastic neutral axis location and flexural strength of composite beams
- Studied steel headed stud anchor strength and factors influencing strength
- Designed a composite beam using Manual tables
- Addressed encased and filled composite flexural components

8.116

Course Conclusion

- We have studied tension members, compression members, and flexural members.
- We have investigated the interaction of compression and bending.
- We have addressed the requirements for stability analysis and design.
- And we have concluded with a treatment of composite beams.

8.117

Course Conclusion

- We have only touched on the basic principles of structural steel design
  - It was our intent to make this a useful refresher for those who have not designed in structural steel for some time
  - We hope that through this course your overall capabilities in structural steel design have been improved.

8.118

Thank You

American Institute of Steel Construction
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Chicago, IL 60601

8.119

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- Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

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8-Session Registrants

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

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- EEU – You must take all quizzes and the final exam to receive EEU.
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- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

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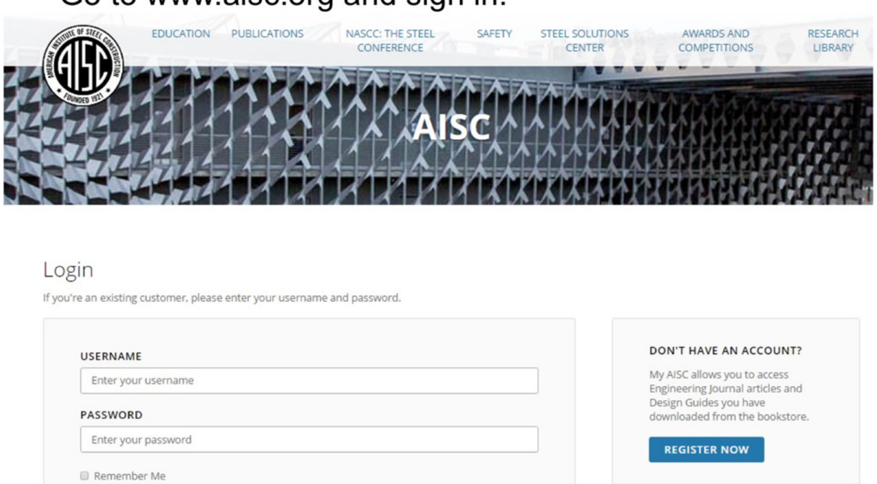

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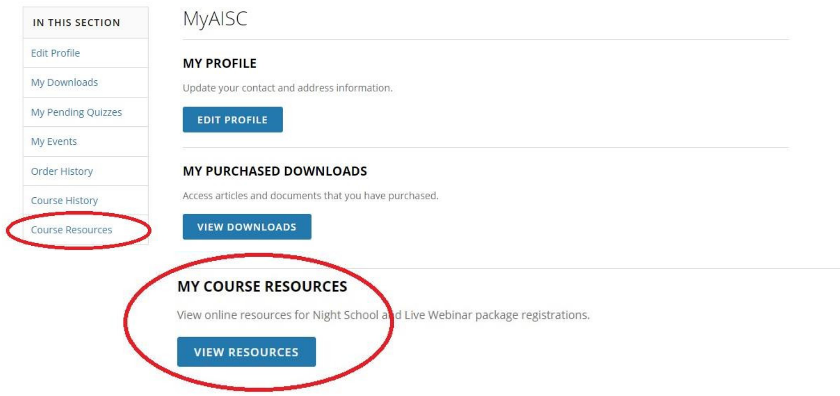

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