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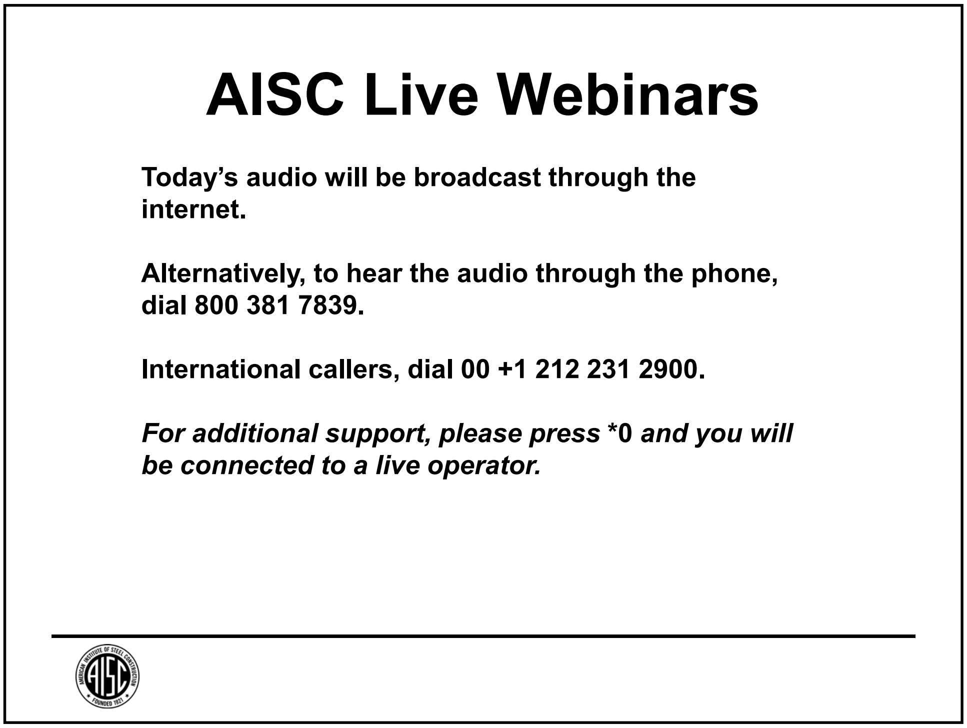

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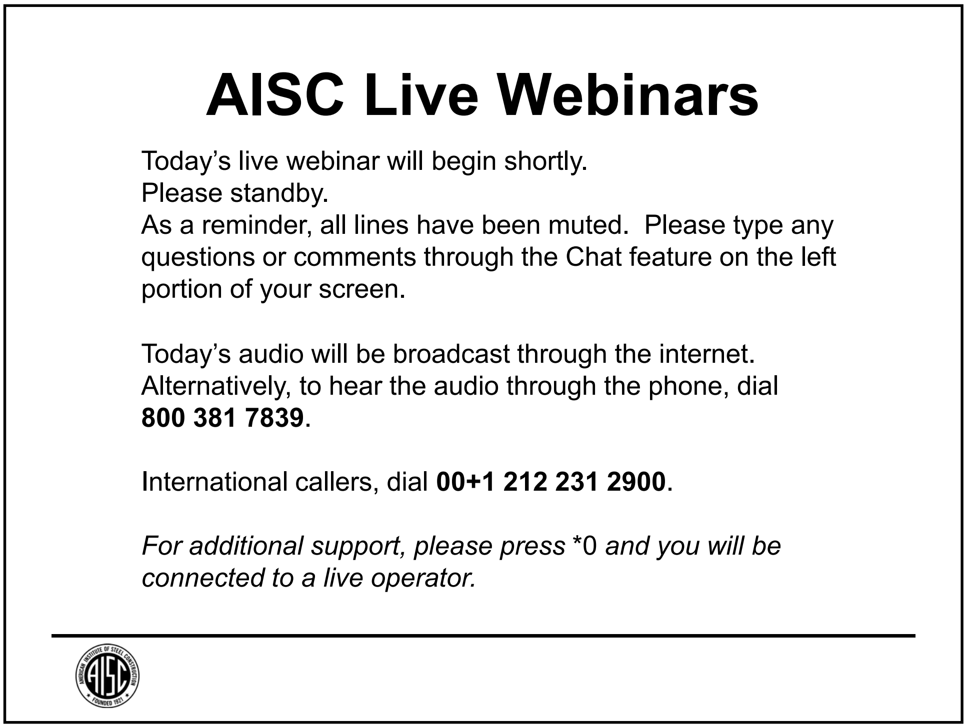

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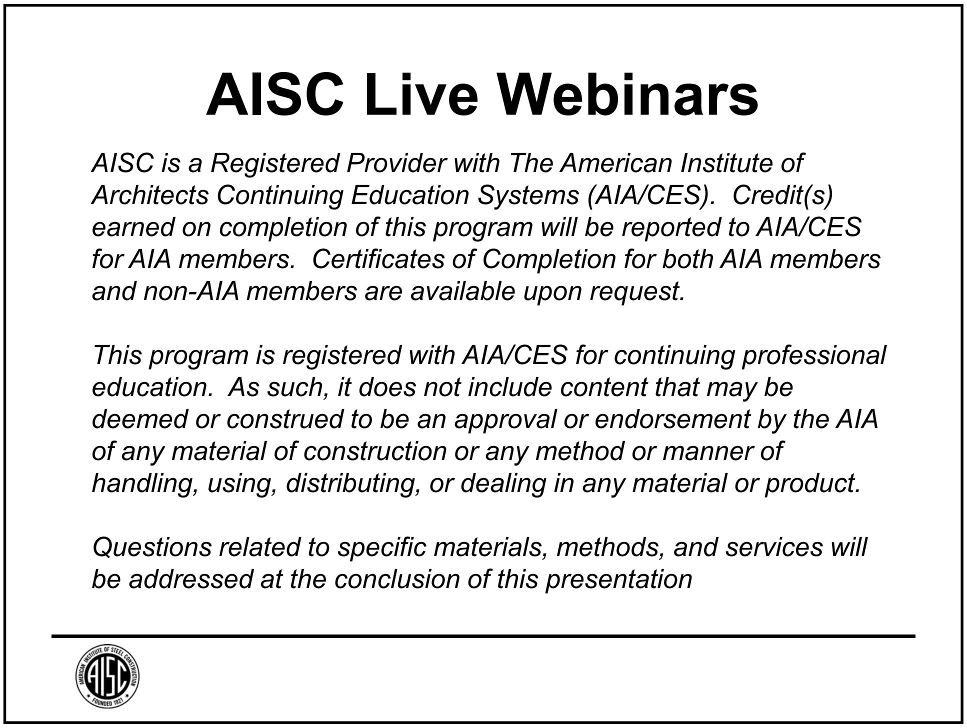

# AISC Live Webinars

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# AISC Live Webinars

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# Course Description

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

***November 14, 2013***

This seminar presents practical guidance for designing reinforced wide flange members, both beams and columns, using the 2010 AISC *Specification*. The existing design procedures for reinforced members are based on the allowable stress approach; however, the 2010 *Specification* is based on strength design. A strength approach is presented that is compatible with the 2010 AISC *Specification*. Considerations that affect the strength and stability of reinforced members, such as residual stresses and welding distortion will be discussed; however, the main focus of the seminar is the presentation of practical design information.

# Learning Objectives

- To learn and understand the strength design requirements for designing reinforced wide flange members.
- To gain familiarity with the design requirements in the 2010 AISC Specification for reinforcing steel members.
- To learn and understand the items that affects the strength and stability of reinforced steel members.
- Learn and understand design of reinforcing for steel members through the use of practical design information.

There's always a solution in steel.

# DESIGN OF REINFORCEMENT FOR STEEL MEMBERS

Bo Dowswell, P.E., Ph.D.

Principal

ARC International, LLC

# Course Description

- General Information
- Column Design
- Beam Design

# GENERAL INFORMATION

## Reasons for Reinforcement

- Change in Service Conditions
  - Building expansions
  - Buildings used for purposes not intended in the original design
  - Additional machinery
  - Higher capacity cranes
  - Etc.

## Reasons for Reinforcement

- Repairs
  - Failures due to accidents or overloads
  - Reduction in cross section (corrosion)
  - Fatigue cracks
  - Seismic, fire, blast or impact damage
  - Foundation settlement
  - Etc.

# Reinforcement Schemes

- Member Reinforcement: Cross section is enlarged to provide the properties required to resist the applied loads.
- Change the Load Path
- Composite Action, Prestressing. Etc.

# COLUMN DESIGN

## General Design Procedure

1. Determine the Reinforcement Type
2. Estimate the reinforcement size
3. Calculate the properties of the reinforced cross section using the estimated reinforcement size from Step 2
4. Determine the effect of Geometric Imperfections on flexural buckling

(continued)

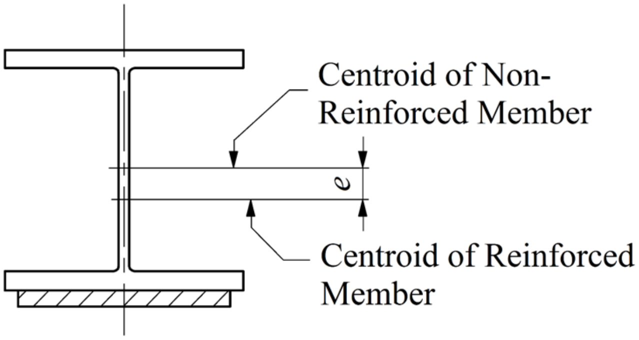
## General Design Procedure

5. Determine the effect of Pre-Load on flexural buckling
6. Calculate the effect of Partial-Length Reinforcement
7. Check strength and stability using the AISC *Specification*

# Reinforcement Type

Singly Symmetric Reinforcement

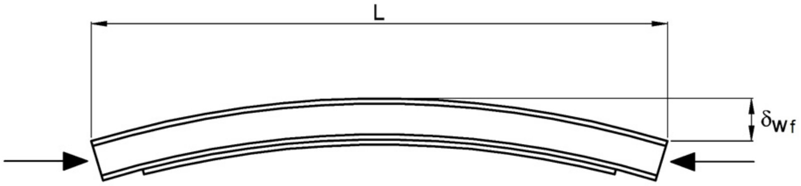
- Axial Eccentricity



# Reinforcement Type

Singly Symmetric Reinforcement

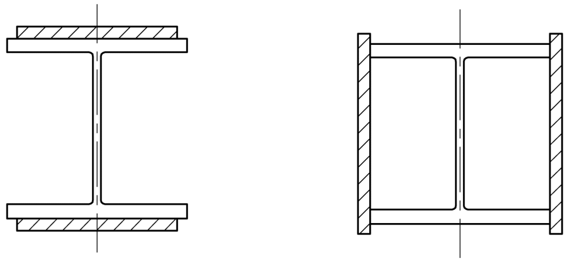
- Weld Shrinkage Distortion Potentially Significant



# Reinforcement Type

Doubly Symmetric Reinforcement

- No Axial Eccentricity
- Less Weld Shrinkage Distortion



# AISC *Specification*

The flexural buckling strength is calculated with AISC *Specification* Section E3.

$$P_n = F_{cr} A_g$$

When $F_y / F_e \le 2.25$ $\quad F_{cr} = \left[ 0.658^{\left(\frac{F_y}{F_e}\right)} \right] F_y$

When $F_y / F_e > 2.25$ $\quad F_{cr} = 0.877 F_e$

# AISC *Specification*

AISC *Specification* Section E3 (continued)

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2}$$

$A_g$ = cross sectional area
$E$ = modulus of elasticity
$F_y$ = yield strength
$K$ = effective length factor
$L$ = unbraced length
$r$ = radius of gyration

# Geometric Imperfections

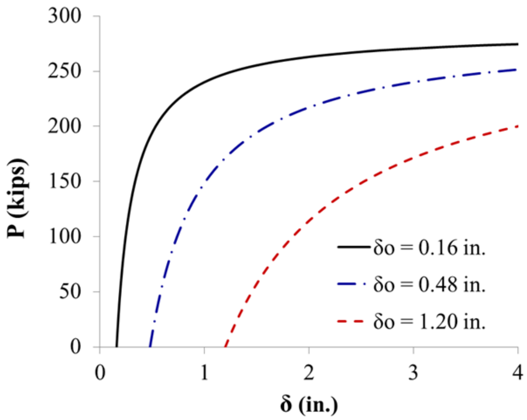
Sources

- Rolling, fabrication and erection, $\delta_i$
- $P$-$\delta$ deformations (prior to reinf.), $\delta_l$
- Weld shrinkage deformation, $\delta_w$

$\delta_d$ = total out-of-straightness after welding

$$\delta_d = \delta_i + \delta_l + \delta_w$$

# Geometric Imperfections



Influence of δ on column stability

# Geometric Imperfections

If $\delta_d > \delta_0$, design as a beam-column with combined axial load + moment per AISC *Specification* Chapter H

$$M = P_r\left(\delta_d - \delta_0\right)$$

$P_r$ = required axial compressive strength
$\delta_0$ = allowable out-of-straightness per COSP
$= L/1{,}000$

# Geometric Imperfections

Second Order Moment

AISC *Specification* Appendix 8, Section 8.2

$$M_r = B_1 M \qquad B_1 = \frac{C_m}{1 - \alpha \dfrac{P_r}{P_{e1}}} \geq 1.0$$

$C_m = 1.0$
$\alpha$ = 1.00 for LRFD, 1.60 for ASD

# Geometric Imperfections

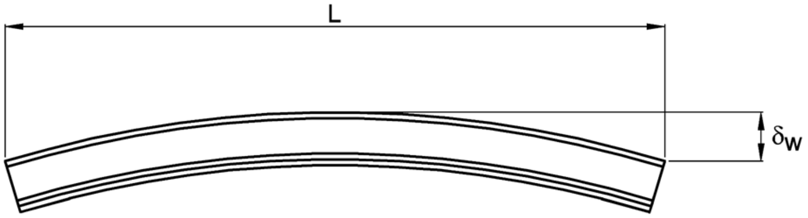
Elastic Buckling Load

$$P_{e1} = \frac{\pi^2 EI}{(KL)^2}$$

$E$ = modulus of elasticity
$I$ = moment of inertia of the built-up member
$K$ = effective length factor
$L$ = unbraced length of the member

# Geometric Imperfections

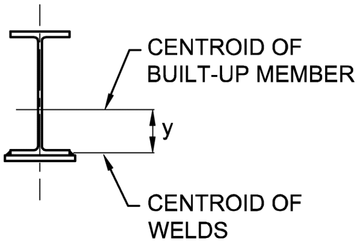
Weld Distortion



# Geometric Imperfections

Weld Distortion
REF: Blodgett (1966)

$$\delta_w = \frac{AyL^2}{200I}$$



$A$ = total cross sectional area of the welds
$I$ = moment of inertia of the built-up section
$L$ = length of member, plate and weld, assuming full-length reinforcement

# Geometric Imperfections

Over-Welding

$$\delta_w = \frac{AyL^2}{200I}$$

$A$ = total cross sectional area of the welds

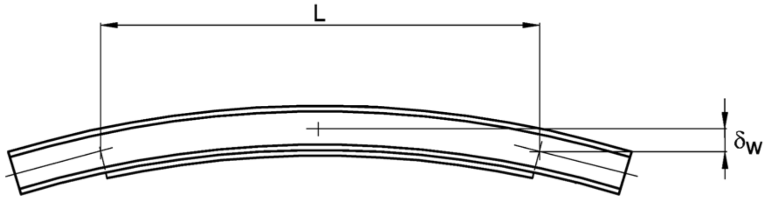
For two fillet welds, $A = w^2$

$w$ = size of weld leg

Distortion varies with the square of the weld size.

# Geometric Imperfections

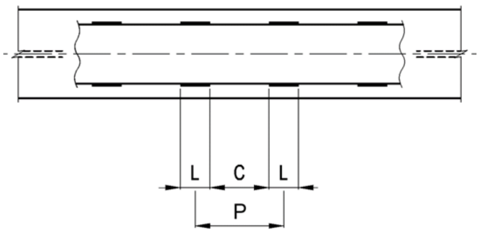
Weld Distortion with Partial-Length Reinforcement



Treat non-welded ends as a straight, rigid link

## Geometric Imperfections

$$\delta'_w = R_s \delta_w$$



$R_s$ = reduction factor for intermittent welds
= $L/P$

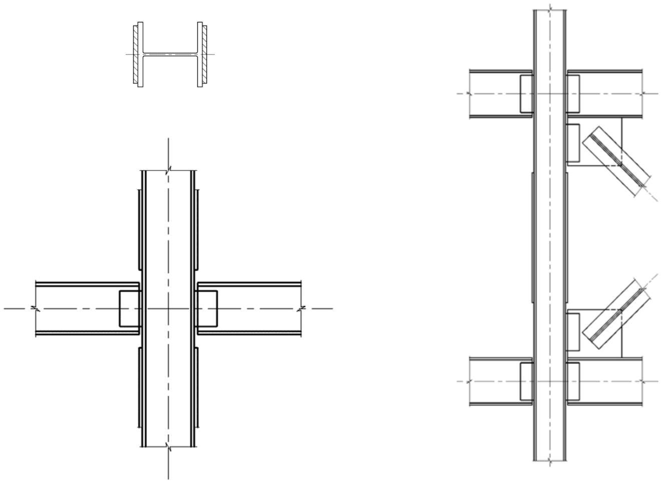
For members welded under load, see Huenersen et al. (1990)

## Geometric Imperfections

Drawing Notes

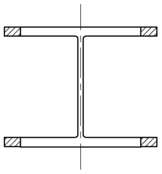
- Welding shall be performed using techniques and sequences that minimize distortion
- Post-weld tolerances should be addressed

# Partial-Length Reinforcement

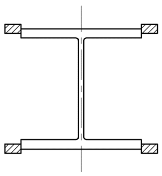


# Partial-Length Reinforcement

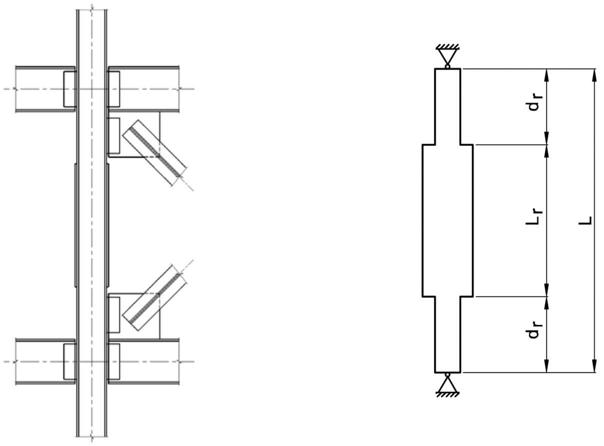
Supplementary reinforcement can be used at locations where primary reinforcement is obstructed.



Supplementary reinforcement is rarely required if a stepped column approach is taken.



# Partial-Length Reinforcement



# Partial-Length Reinforcement

REF: Dalal (1969)

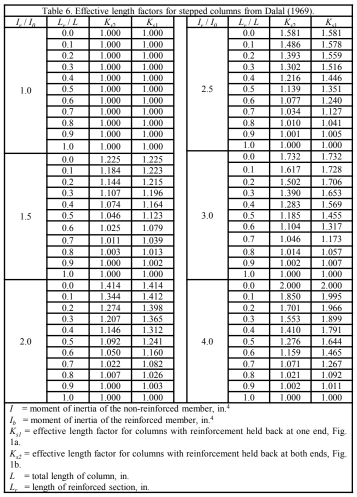


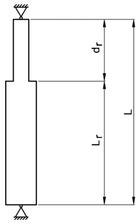
Table 6. Effective length factors for stepped columns from Dalal (1969).

| $I_r / I_0$ | $L_r / L$ | $K_{s2}$ | $K_{s1}$ | $I_r / I_0$ | $L_r / L$ | $K_{s2}$ | $K_{s1}$ |
|---|---|---|---|---|---|---|---|
| 1.0 | 0.0 | 1.000 | 1.000 | 2.5 | 0.0 | 1.581 | 1.581 |
| | 0.1 | 1.000 | 1.000 | | 0.1 | 1.486 | 1.578 |
| | 0.2 | 1.000 | 1.000 | | 0.2 | 1.393 | 1.559 |
| | 0.3 | 1.000 | 1.000 | | 0.3 | 1.302 | 1.516 |
| | 0.4 | 1.000 | 1.000 | | 0.4 | 1.216 | 1.446 |
| | 0.5 | 1.000 | 1.000 | | 0.5 | 1.139 | 1.351 |
| | 0.6 | 1.000 | 1.000 | | 0.6 | 1.077 | 1.240 |
| | 0.7 | 1.000 | 1.000 | | 0.7 | 1.034 | 1.127 |
| | 0.8 | 1.000 | 1.000 | | 0.8 | 1.010 | 1.041 |
| | 0.9 | 1.000 | 1.000 | | 0.9 | 1.001 | 1.005 |
| | 1.0 | 1.000 | 1.000 | | 1.0 | 1.000 | 1.000 |
| 1.5 | 0.0 | 1.225 | 1.225 | 3.0 | 0.0 | 1.732 | 1.732 |
| | 0.1 | 1.184 | 1.223 | | 0.1 | 1.617 | 1.728 |
| | 0.2 | 1.144 | 1.215 | | 0.2 | 1.502 | 1.706 |
| | 0.3 | 1.107 | 1.196 | | 0.3 | 1.390 | 1.653 |
| | 0.4 | 1.074 | 1.164 | | 0.4 | 1.283 | 1.569 |
| | 0.5 | 1.046 | 1.123 | | 0.5 | 1.185 | 1.455 |
| | 0.6 | 1.025 | 1.079 | | 0.6 | 1.104 | 1.317 |
| | 0.7 | 1.011 | 1.039 | | 0.7 | 1.046 | 1.173 |
| | 0.8 | 1.003 | 1.013 | | 0.8 | 1.014 | 1.057 |
| | 0.9 | 1.000 | 1.002 | | 0.9 | 1.002 | 1.007 |
| | 1.0 | 1.000 | 1.000 | | 1.0 | 1.000 | 1.000 |
| 2.0 | 0.0 | 1.414 | 1.414 | 4.0 | 0.0 | 2.000 | 2.000 |
| | 0.1 | 1.344 | 1.412 | | 0.1 | 1.850 | 1.995 |
| | 0.2 | 1.274 | 1.398 | | 0.2 | 1.701 | 1.966 |
| | 0.3 | 1.207 | 1.365 | | 0.3 | 1.553 | 1.899 |
| | 0.4 | 1.146 | 1.312 | | 0.4 | 1.410 | 1.791 |
| | 0.5 | 1.092 | 1.241 | | 0.5 | 1.276 | 1.644 |
| | 0.6 | 1.050 | 1.160 | | 0.6 | 1.159 | 1.465 |
| | 0.7 | 1.022 | 1.082 | | 0.7 | 1.071 | 1.267 |
| | 0.8 | 1.007 | 1.026 | | 0.8 | 1.021 | 1.092 |
| | 0.9 | 1.000 | 1.003 | | 0.9 | 1.002 | 1.011 |
| | 1.0 | 1.000 | 1.000 | | 1.0 | 1.000 | 1.000 |

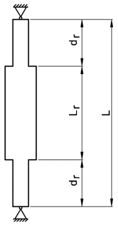
$I$ = moment of inertia of the non-reinforced member, in.$^4$
$I_b$ = moment of inertia of the reinforced member, in.$^4$
$K_{s1}$ = effective length factor for columns with reinforcement held back at one end, Fig. 1a.
$K_{s2}$ = effective length factor for columns with reinforcement held back at both ends, Fig. 1b.
$L$ = total length of column, in.
$L_r$ = length of reinforced section, in.

# Partial-Length Reinforcement

$K_{s1}$ = effective length factor for columns with reinforcement held back at one end



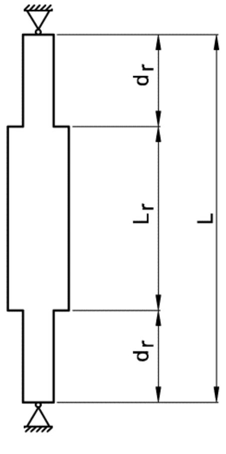
$K_{s2}$ = effective length factor for columns with reinforcement held back at both ends



# Partial-Length Reinforcement

$I_0$ = moment of inertia of the non-reinforced member, in.$^4$

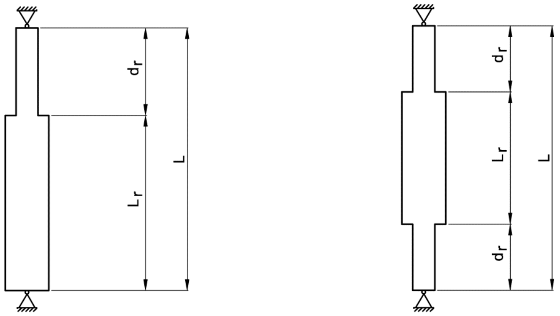
$I_r$ = moment of inertia of the reinforced member, in.$^4$



# Partial-Length Reinforcement

$L$ = total length of column, in.

$L_r$ = length of reinforced section, in.



# Partial-Length Reinforcement

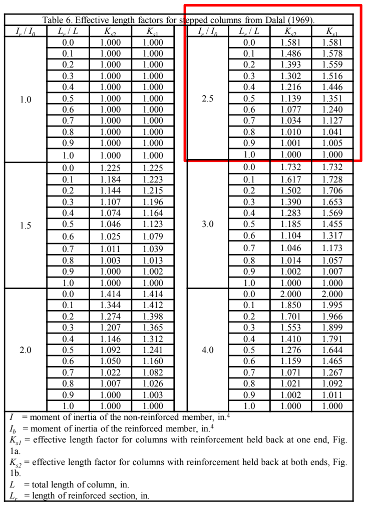


Table 6. Effective length factors for stepped columns from Dalal (1969).

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| | 0.1 | 1.184 | 1.223 | | 0.1 | 1.617 | 1.728 |
| | 0.2 | 1.144 | 1.215 | | 0.2 | 1.502 | 1.706 |
| | 0.3 | 1.107 | 1.196 | | 0.3 | 1.390 | 1.653 |
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| | 1.0 | 1.000 | 1.000 | | 1.0 | 1.000 | 1.000 |
| 2.0 | 0.0 | 1.414 | 1.414 | 4.0 | 0.0 | 2.000 | 2.000 |
| | 0.1 | 1.344 | 1.412 | | 0.1 | 1.850 | 1.995 |
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| | 0.3 | 1.207 | 1.365 | | 0.3 | 1.553 | 1.899 |
| | 0.4 | 1.146 | 1.312 | | 0.4 | 1.410 | 1.791 |
| | 0.5 | 1.092 | 1.241 | | 0.5 | 1.276 | 1.644 |
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# Partial-Length Reinforcement

| $I_r / I_0$ | $L_r / L$ | $K_{s2}$ | $K_{s1}$ |
|---|---|---|---|
| 2.5 | 0.0 | 1.581 | 1.581 |
| | 0.1 | 1.486 | 1.578 |
| | 0.2 | 1.393 | 1.559 |
| | 0.3 | 1.302 | 1.516 |
| | 0.4 | 1.216 | 1.446 |
| | 0.5 | 1.139 | 1.351 |
| | 0.6 | 1.077 | 1.240 |
| | 0.7 | 1.034 | 1.127 |
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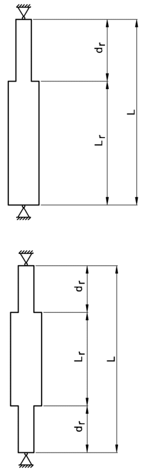
| | 1.0 | 1.000 | 1.000 |



# Pre-Load

Pre-Load: required load at the time of reinforcement

Two Cases

- Column with Stabilizing Reinforcement: Pre-load can be neglected
- Column with Non-Stabilizing Reinforcement: Pre-load must be considered

# Pre-Load

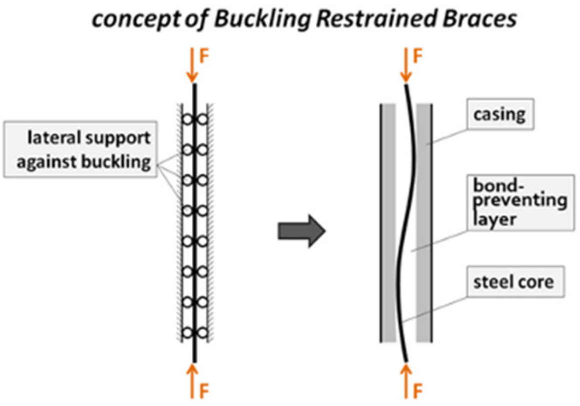
Definition of Stabilizing Reinforcement: For the axis of buckling: $r_r \geq 0.85 r_0$

$r_0 = r$ for original cross section
$r_r = r$ for reinforced cross section

---

# Pre-Load

Stabilizing Reinforcement Concept



# Pre-Load

If $r_r < 0.85r_0$, an allowable stress approach can be used.

$$\sigma = \sigma_0 + \sigma_r$$

$\sigma_0$ = stress in non-reinforced member using loads applied before the member is reinforced

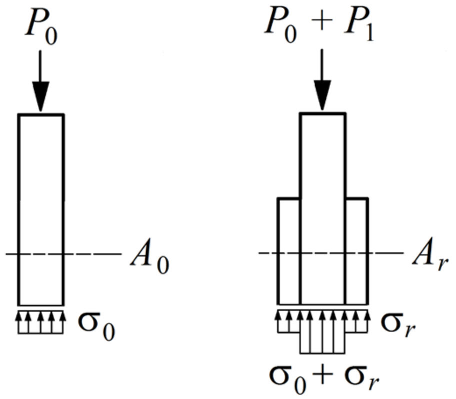
$\sigma_r$ = stress in reinforced member using loads applied after the member is reinforced

# Pre-Load

For $r_r < 0.85r_0$ (non-stabilizing reinforcement)

$$\sigma_0 = \frac{P_0}{A_0}$$

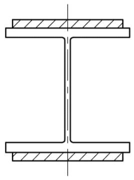
$$\sigma_r = \frac{P_1}{A_r}$$



# Pre-Load

Experimental Results

- Nagaraja and Tall (1963)
- Marzouk and Mohan (1990)


Type C

Columns: W6×15, W8×18, W8×31
$L =$ 8.00 ft to 10.2 ft
Reinf. Plates: 0.315 in. to 0.375 in. thick
Type C Reinforcement

# Pre-Load

$r_r > 0.85r_0$ for all tests

Notation

$P_0$ = initial pre-load at the time of reinforcement

$P_t$ = maximum total experimental load

$P_n$ = nominal compression strength (AISC *Specification* Section E3 neglecting pre-load)

# Pre-Load

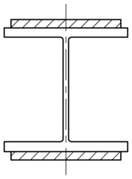
| Spec. | $KL/r$ | $P_0$ kips | $P_t$ kips | $P_n$ kips | $P_t / P_n$ |
|---|---|---|---|---|---|
| T-9 | 48 | 0 | 307 | 298 | 1.03 |
| T-17 | 48 | 0 | 517 | 475 | 1.09 |
| T-13 | 48 | 91 | 528 | 475 | 1.11 |
| 1 | 97 | 89 | 228 | 212 | 1.08 |
| 2 | 89 | 89 | 239 | 233 | 1.03 |
| 4 | 79 | 99 | 288 | 226 | 1.27 |
| 5 | 79 | 99 | 270 | 226 | 1.19 |
| 6 | 83 | 99 | 236 | 217 | 1.09 |

Specimen T-9 was not reinforced.

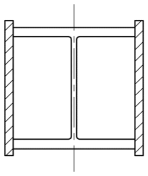
# Pre-Load

Wu and Grondin (2002)

- 317 FE models
- Pre-loads up to 60% of member strength
- Type C and D Reinforcement
- $r_r > 0.85r_0$ for all specimens
- Conclusion: The effect of pre-load is negligible



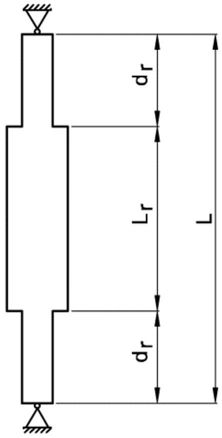
Type C



Type D

# Example

Design reinforcement for an existing W14×90 column using LRFD.



$F_y = 50$ ksi (yield strength)

$L_x = L_y = 18$ ft (unbraced lengths)

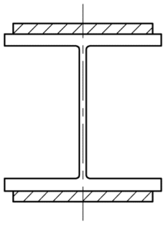
$P_0 = 300$ kips (factored pre-load)

$P_u = 1{,}100$ kips (total factored axial load)

# Example

The available strength of the column without reinforcement is 929 kips.

Try 2 Plates ¾ in. × 12 in. welded parallel to each flange. The properties of the reinforced cross section are



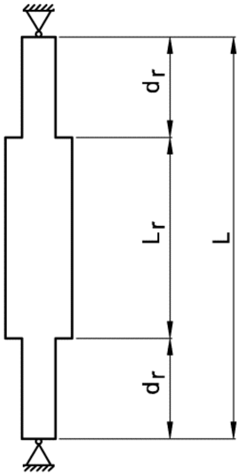
$A_r = 44.5\text{ in.}^2$

$I_{ry} = 578\text{ in.}^4$

$r_{ry} = 3.60\text{ in.}$

# Example

To accommodate existing obstructions, the reinforcement standoff dimension at each end of the column, $d_r = 4\text{ ft}$



# Example

Check for stabilizing reinforcement

The y-axis radius of gyration of the W14×90 is, $r_{0y} = 3.70\text{ in.}$

$r_r \geq 0.85 r_0$ ?

$3.60\text{ in.} > (0.85)(3.70\text{ in.})$; therefore, the 300 kip pre-load can be neglected

## Example

Calculate the effective length factor based on a stepped column approach.

The y-axis moment of inertia of the W14×90 is, $I_{0y} = 362 \text{ in.}^4$

$I_{ry} / I_{0y} = 578 \text{ in.}^4 / 362 \text{ in.}^4 = 1.60$

(continued)

## Example

$L_r = 18 \text{ ft} - (2)(4 \text{ ft}) = 10 \text{ ft}$

$L_r / L = (10 \text{ ft}) / (18 \text{ ft}) = 0.556$

$K_y = 1.04$ (interpolated from tables in Dalal, 1969)

# Example

AISC *Specification* Section E3

$$\left(\frac{KL}{r}\right)_y = \frac{(1.04)(18\text{ ft})(12\text{ in./ft})}{3.60\text{ in.}} = 62.4$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(62.4)^2} = 73.5\text{ ksi}$$

# Example

$73.5\text{ ksi} \geq (0.44)(50\text{ ksi})$, Use *Specification* Eqn. E3-2

$$F_{cr} = \left[0.658^{\left(\frac{F_y}{F_e}\right)}\right]F_y$$

$$= \left[0.658^{\left(\frac{50\text{ ksi}}{73.5\text{ ksi}}\right)}\right](50\text{ ksi})$$

$$= 37.6\text{ ksi}$$

# Example

$$P_n = F_{cr}A_g$$
$$= (37.6\text{ ksi})(44.5\text{ in.}^2)$$
$$= 1{,}670\text{ kips}$$

$$\phi P_n = (0.9)(1{,}670\text{ kips})$$
$$= 1{,}500\text{ kips}$$

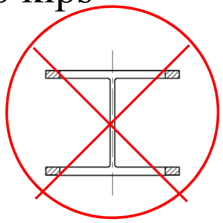
1,500 kips > 1,100 kips **o.k.**

# Example

Check non-reinforced segment of column for yielding

The area of the W14×90 is, $A_0 = 26.5\text{ in.}^2$

$\phi P_n = (0.9)(50\text{ ksi})(26.5\text{ in.}^2) = 1{,}190\text{ kips}$



1,190 kips > 1,100 kips; therefore, supplementary reinforcement is not required

# BEAM DESIGN

# General Design Procedure

1. Determine the Reinforcement Type
2. Estimate the reinforcement size
3. Calculate the properties of the reinforced cross section using the estimated reinforcement size from Step 2
4. Determine if the compression flange is braced laterally. If so, Steps 5 and 6 are not necessary.

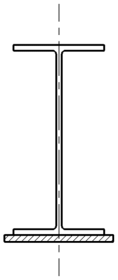
(continued)

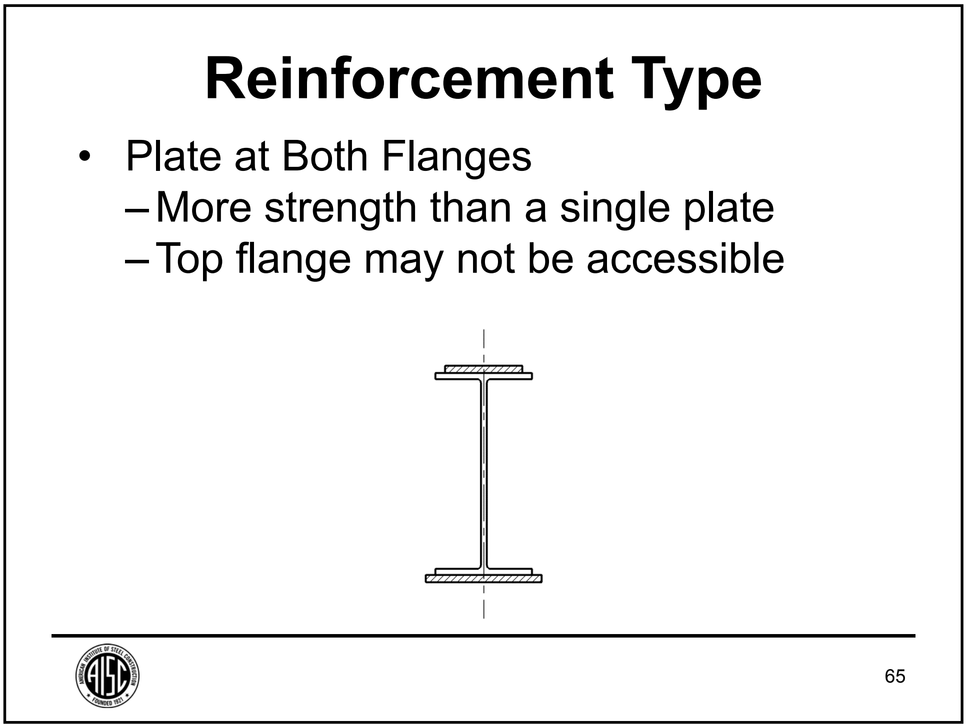
## General Design Procedure

5. Determine the effect of Pre-Load on lateral-torsional buckling
6. Calculate the effect of Partial-Length Reinforcement on lateral-torsional buckling
7. Check strength and stability using the AISC *Specification*
8. Check serviceability

## Reinforcement Type

- Plate at Bottom Flange
  - Plate easily clamped in place for fit-up
  - Welding in the horizontal position
  - Camber due to weld shrinkage is upward

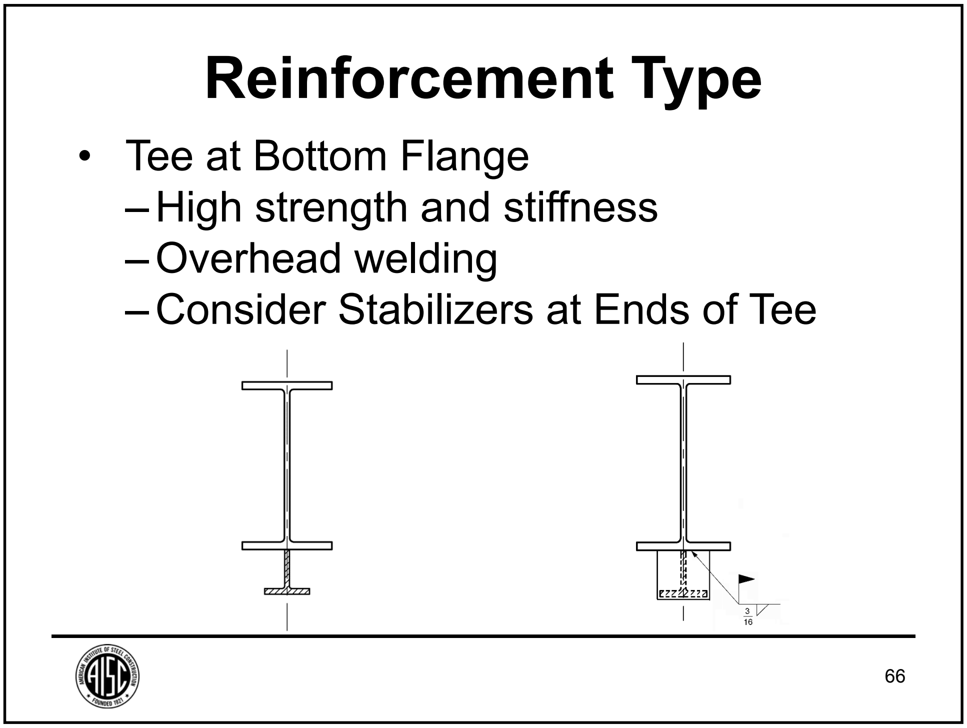




# Reinforcement Type

- Plate at Both Flanges
  - More strength than a single plate
  - Top flange may not be accessible

65



# Reinforcement Type

- Tee at Bottom Flange
  - High strength and stiffness
  - Overhead welding
  - Consider Stabilizers at Ends of Tee

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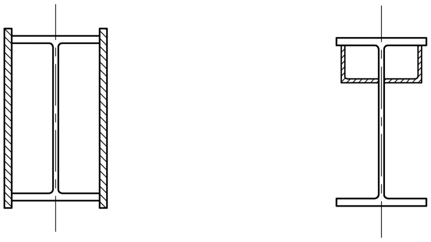
# Reinforcement Type

Weak-Axis Bending

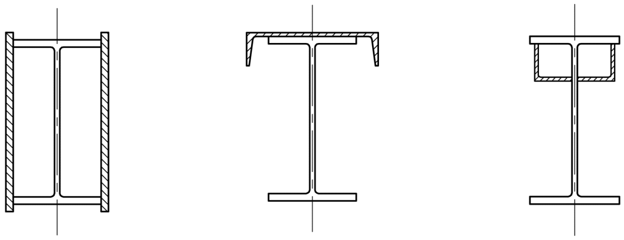
# Reinforcement Type

Torsion



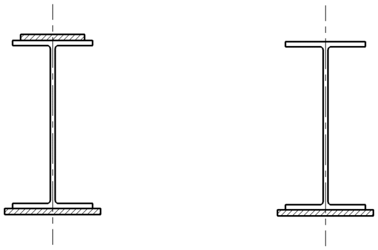
# Reinforcement Type

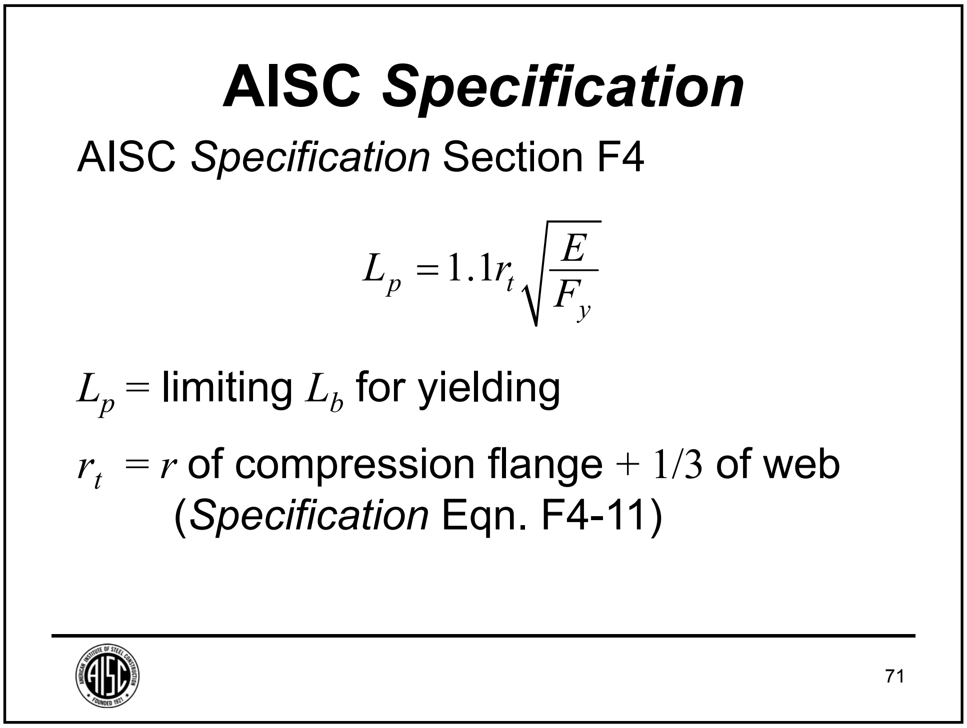
Lateral-Torsional Buckling



# AISC *Specification*

For flexural strength, AISC *Specification* Section F2 applies to doubly-symmetric I-shaped members. Section F4 must be used for singly-symmetric I-shaped members.





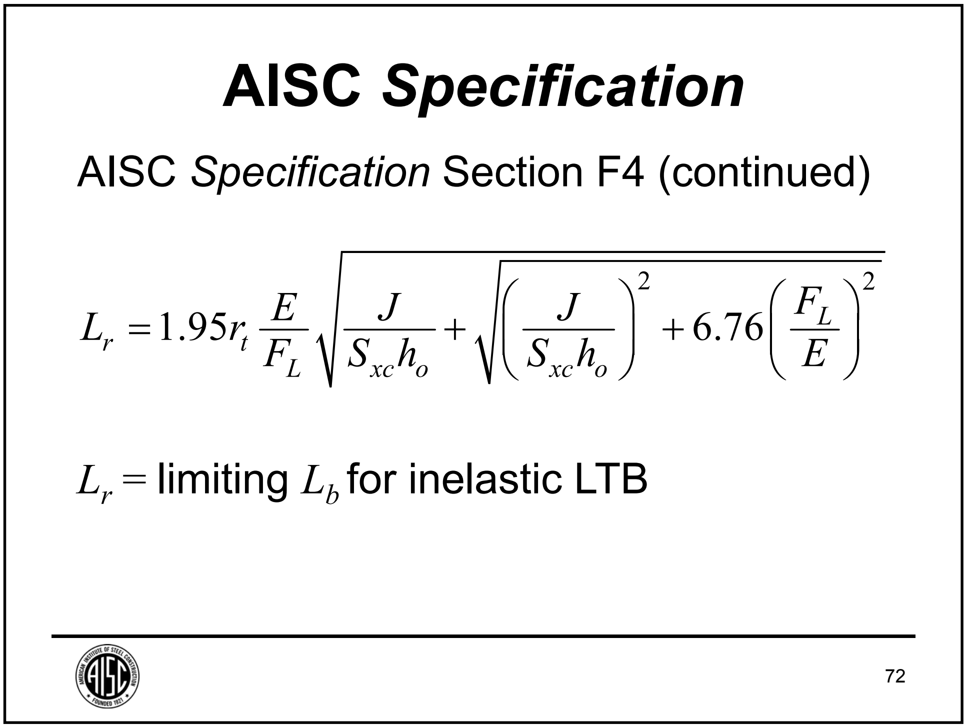
# AISC *Specification*

AISC *Specification* Section F4

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}}$$

$L_p$ = limiting $L_b$ for yielding

$r_t$ = $r$ of compression flange + $1/3$ of web (*Specification* Eqn. F4-11)



# AISC *Specification*

AISC *Specification* Section F4 (continued)

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o} + \sqrt{\left(\frac{J}{S_{xc} h_o}\right)^2 + 6.76\left(\frac{F_L}{E}\right)^2}}$$

$L_r$ = limiting $L_b$ for inelastic LTB

# AISC *Specification*

AISC *Specification* Section F4 (continued)

$F_L$ = magnitude of flexural stress in the compression flange at which lateral-torsional buckling is influenced by yielding

When $S_{xt}/S_{xc} \geq 0.7$ $\qquad F_L = 0.7F_y$

When $S_{xt}/S_{xc} < 0.7$ $\qquad F_L = \dfrac{S_{xt}}{S_{xc}}F_y$

# AISC *Specification*

AISC *Specification* Section F4 (continued)

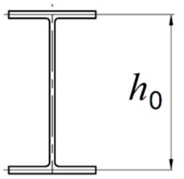
$S_{xc}$ = section modulus referred to compression flange

$S_{xt}$ = section modulus referred to tension flange

# AISC *Specification*

AISC *Specification* Section F4 (continued)

$h_o$ = distance between the flange centroids



# AISC *Specification*

AISC *Specification* Section F4 (continued)

$J$ = torsion constant (for $I_{yc}/I_y \leq 0.23$, $J$ shall be taken as zero)

$I_y$ = moment of inertia of the cross section about the y-axis

$I_{yc}$ = moment of inertia of the compression flange about the y-axis

# AISC *Specification*

AISC *Specification* Section F4

When $L_b \leq L_p$ No lateral-torsional buckling

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc}$$

$L_b$ = length between braced points

$M_{yc}$ = Yield moment in compression flange

# AISC *Specification*

AISC *Specification* Section F4

$R_{pc}$ = web plastification factor

For compact webs

When $I_{yc}/I_y > 0.23$ $R_{pc} = \frac{M_p}{M_{yc}}$

When $I_{yc}/I_y \leq 0.23$ $R_{pc} = 1.0$

$M_p = F_y Z_x \leq 1.6\, F_y S_{xc}$

# AISC *Specification*

AISC *Specification* Section F4 (continued)

When $L_p < L_b \le L_r$

$$M_n = C_b\left[R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$\le R_{pc}M_{yc}$$

$C_b =$ lateral-torsional buckling modification factor (*Specification* Eqn. F1-1)

# AISC *Specification*

AISC *Specification* Section F4 (continued)

When $L_b > L_r$

$$M_n = F_{cr}S_{xc} \le R_{pc}M_{yc}$$

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

## AISC *Specification*

AISC *Specification* Section F4 (continued)

Tension Flange Yielding

When $S_{xt} \geq S_{xc}$ tension flange yielding does not apply.

## AISC *Specification*

AISC *Specification* Section F4 (continued)

When $S_{xt} < S_{xc}$

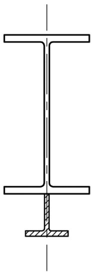
For compact webs $M_n = R_{pt} M_{yt} = M_p$

$$R_{pt} = \frac{M_p}{M_{yt}}$$

$$M_{yt} = F_y S_{xt}$$

# AISC *Specification*

AISC *Specification* Section F4 applies only to I-shaped members. Can a beam with WT reinforcement be considered I-shaped?



# AISC *Specification*

The answer can be found with the classical equation for singly-symmetric beams

$$M_{cr} = \frac{\pi^2 E I_y}{L_b^2}\left[\frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y} + \frac{GJL_b^2}{\pi^2 E I_y}}\right]$$

$C_w$ = warping constant

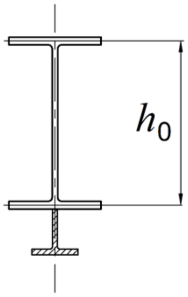
## AISC *Specification*

$$\beta_x = \frac{1}{I_x}\int_A y\left(x^2 + y^2\right)da - 2y_0$$

$x, y$ = centroidal coordinates
$y_0$ = distance from the shear center to the centroid of the cross section

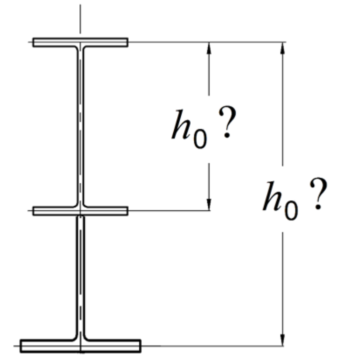
## AISC *Specification*

Conclusion: AISC *Specification* Section F4 is slightly conservative if $h_0$ is calculated as defined for the non-reinforced beam.



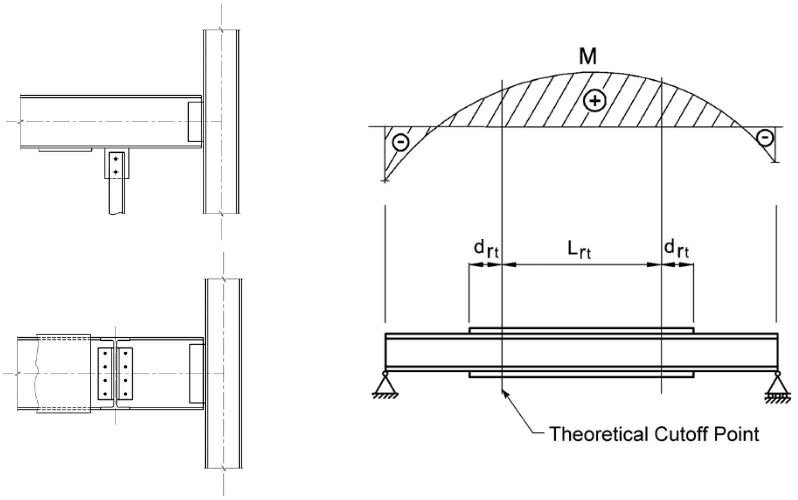
The level of conservatism increases as the size of the WT increases relative to the size of the W.

# AISC *Specification*



Where the WT is large, relative to the W, an alternative definition of $h_0$ may be appropriate.

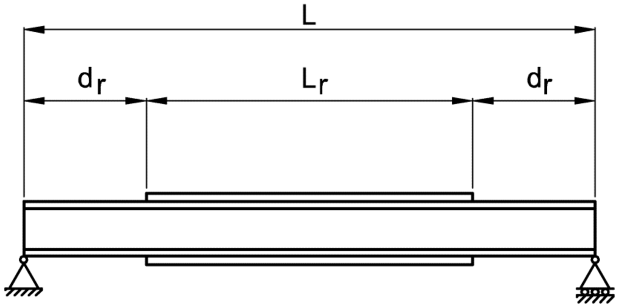
# Partial-Length Reinforcement



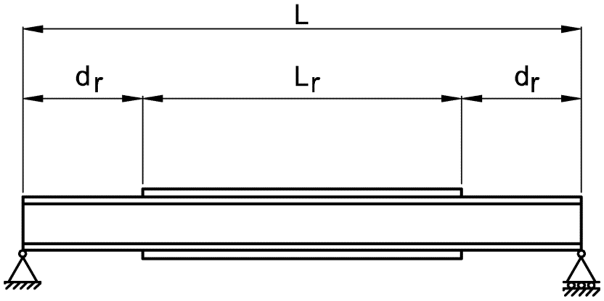
# Partial-Length Reinforcement

REF: Trahair and Kitipornchai (1971)

$$C_b' = \beta_{LTB} C_b$$



# Partial-Length Reinforcement



$$\beta_{LTB} = 1 + \frac{2d_r}{L}\left(\frac{M_{e0}}{M_{er}} - 1\right)$$

## Partial-Length Reinforcement

$M_{er}$ = elastic lateral-torsional buckling moment of reinforced member assuming full-length reinforcement

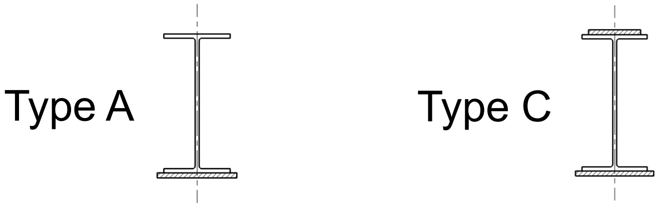
$M_{e0}$ = elastic lateral-torsional buckling moment of non-reinforced member

Use the actual value of $J$ (not $J = 0$) for all values of $I_{yc}/I_y$ when calculating $\beta_{LTB}$

## Partial-Length Reinforcement

Finite Element Results

- Lui and Gannon (2009b)
- Beams: W12×19×7.87 ft long
- Reinf. Plates: 0.394 in. × 5.32 in.
- Type A and C Reinforcement



# Partial-Length Reinforcement

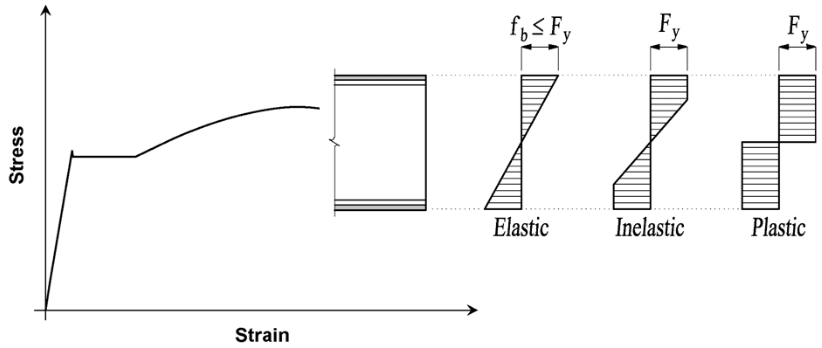
| Spec No. | Reinf Type | $d_r$ (in.) | $L_r / L$ | $\beta_{FE}$ (%) | $\beta_{LTB}$ (%) | $\beta_{FE} / \beta_{LTB}$ |
|---|---|---|---|---|---|---|
| A1 | A | 1.89 | 0.96 | 100 | 98.6 | 1.01 |
| A2 | A | 15.6 | 0.67 | 89.4 | 88.8 | 1.01 |
| A3 | A | 31.6 | 0.33 | 74.3 | 77.4 | 0.960 |
| C1 | C | 1.89 | 0.96 | 100 | 97.0 | 1.03 |
| C2 | C | 15.6 | 0.67 | 90.3 | 75.2 | 1.20 |
| C3 | C | 31.6 | 0.33 | 75.4 | 49.7 | 1.52 |

$\beta_{FE}$ = reduction in FE models

$\beta_{LTB}$ = nominal reduction factor

# Pre-Load

The magnitude of pre-load has no effect on flexural yielding strength.



# Pre-Load

Members with Stabilizing Reinforcement: The magnitude of pre-load has no effect on lateral-torsional buckling.

Definition of Stabilizing Reinf.: $r_{tr} \geq 0.85 r_{t0}$

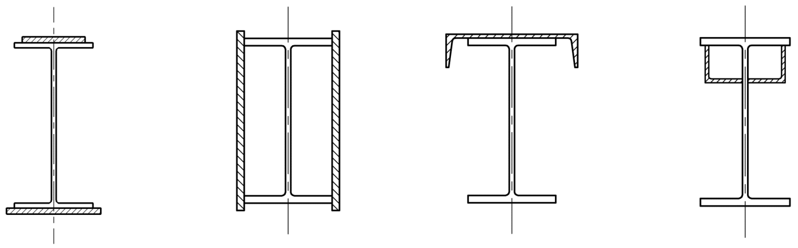
$r_{t0} = r_t$ for original cross section

$r_{tr} = r_t$ for reinforced cross section

$r_t = r$ of compression flange $+ 1/3$ of web

# Pre-Load

Examples of Stabilizing Reinforcement



# Pre-Load

Examples of Non-Stabilizing Reinforcement

# Pre-Load

Lateral-Torsional Buckling with non-stabilizing reinforcement, replace $F_L$ with $F'_L$ in AISC *Specification* Equation F4-2.

$$M_n = C_b \left[ R_{pc} M_{yc} - \left( R_{pc} M_{yc} - F'_L S_{xc} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right]$$
$$\leq R_{pc} M_{yc}$$

# Pre-Load

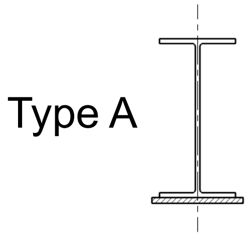
$$F'_L = F_L - \frac{f_{bxi}}{C_{bi}} \qquad f_{bxi} = \frac{M_{xi}}{S_x}$$

$C_{bi}$ = $C_b$ for the pre-load moment diagram
$M_{xi}$ = pre-load moment
$f_{bxi}$ = flexural stress due to pre-load
$S_x$ = $S$ for the non-reinf. member

# Pre-Load

Experimental Results

- Lui and Gannon (2009a)
- Beams: W12×19
- Reinf. Plates: 0.374 in. × 5.39 in.
- Type A Reinforcement



# Pre-Load

Notation

$M_0$ = initial pre-load moment at the time of reinforcement

$M_e$ = maximum experimental moment

$M_n$ = nominal moment calculated using AISC *Specification* Section F4 with $F'_L$ replacing $F_L$ in Equation F4-2.

# Pre-Load

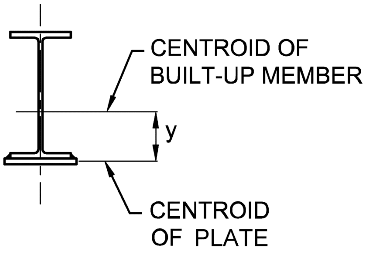
| Spec. | $L_b$ ft | $M_0$ kip-ft | $M_e$ kip-ft | $M_n$ kip-ft | $M_e / M_n$ |
|---|---|---|---|---|---|
| P1 | 7.87 | 0 | 55.8 | 57.1 | 0.977 |
| P2 | 3.94 | 0 | 99.5 | 83.2 | 1.20 |
| A1 | 7.87 | 0 | 99.2 | 78.6 | 1.26 |
| A2 | 7.87 | 14.7 | 99.2 | 69.9 | 1.42 |
| A3 | 7.87 | 29.5 | 92.6 | 61.2 | 1.51 |
| C1 | 3.94 | 0 | 127 | 107 | 1.19 |
| C2 | 3.94 | 25.1 | 118 | 103 | 1.15 |
| C3 | 3.94 | 50.2 | 111 | 98.2 | 1.13 |

Specimens P1 and P2 were not reinforced.

# Welding Issues

Shear Flow

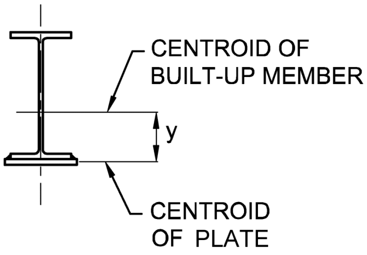
$$v = \frac{V_r Q}{I_{rx}} \qquad Q = A_p y$$



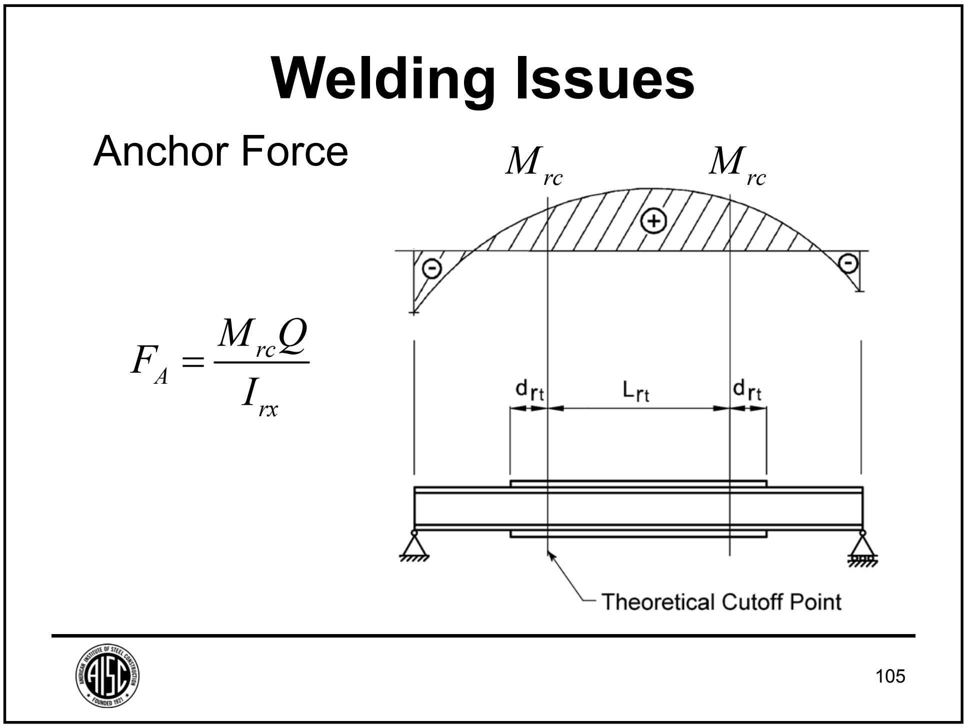
$A_p$ = area of the reinforcing plate
$I_{rx}$ = strong-axis moment of inertia of the built-up member
$V_r$ = required shear force
$v$ = shear force per unit length

# Welding Issues

For reinforcing plates with two welds, the shear force per unit length of weld is $v/2$



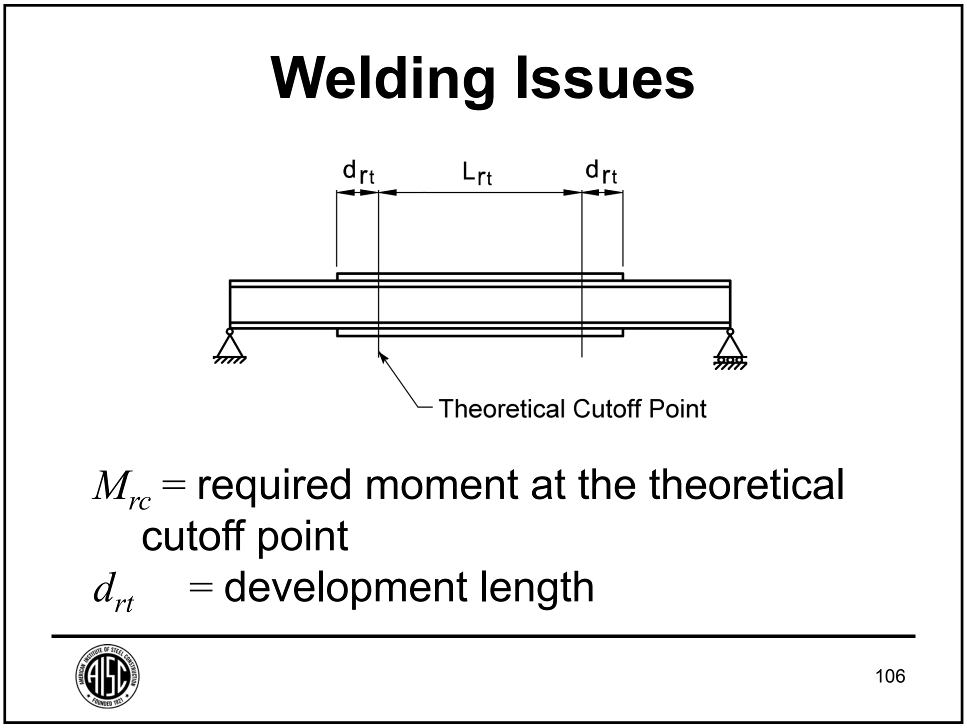
For shear flow at columns, see Goel and Aslani (1992)



# Welding Issues

Anchor Force

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



# Welding Issues

$M_{rc}$ = required moment at the theoretical cutoff point

$d_{rt}$ = development length

# Welding Issues

Members Welded Under Load

- Consider the effects of welding heat on member strength during the welding operation.
- Elevated temperatures near the weld cause a reduction in material properties of the base material.
- See Huenersen et al. (1990)

# Questions?

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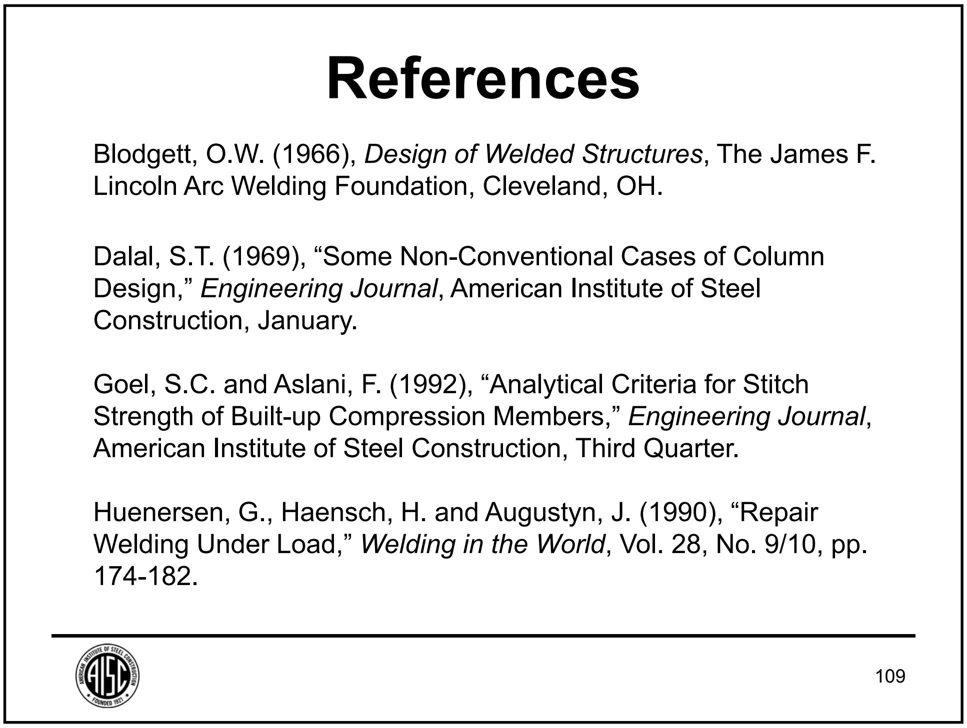

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Dalal, S.T. (1969), “Some Non-Conventional Cases of Column Design,” *Engineering Journal*, American Institute of Steel Construction, January.

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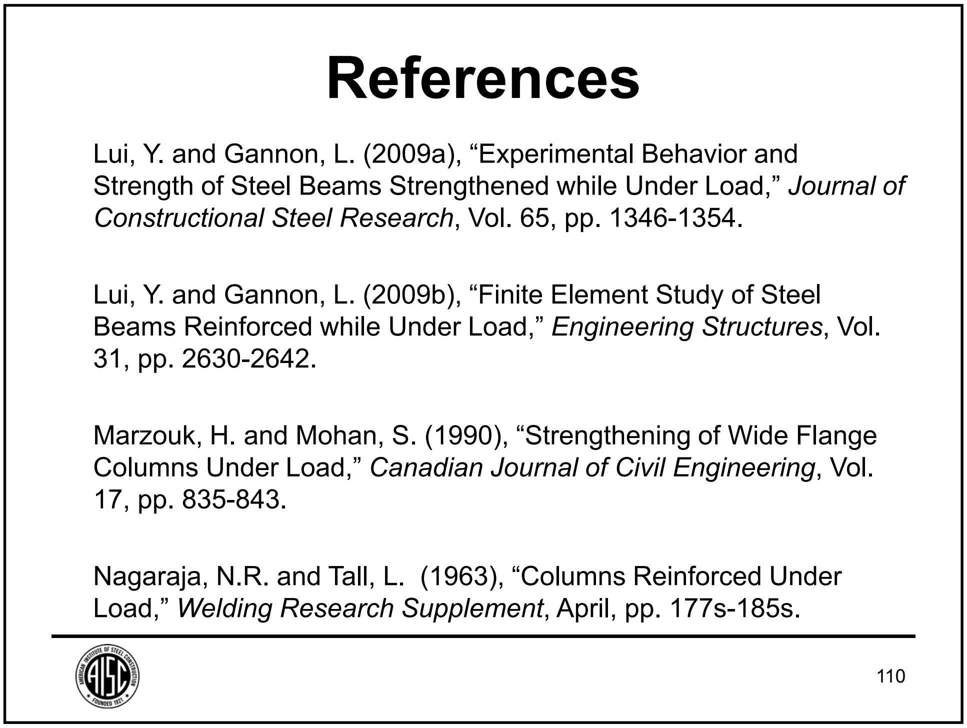

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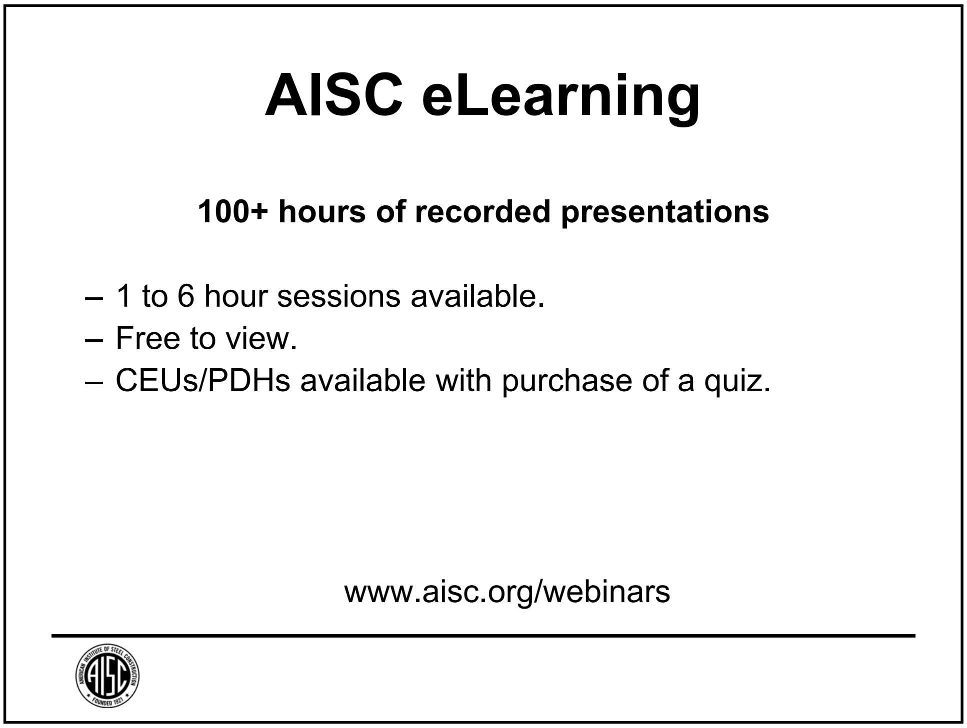

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