NCSEA Structural Engineering Exam Review Course

Lateral Forces Review

Steel Design – Summer 2017

Presented by Rafael Sabelli

Outline

- Lateral steel design
  - Wind
    - Covered under Force Distribution
  - Stability
    - Not covered
  - Seismic
    - AISC 341: Seismic Provisions for Structural Steel Buildings
    - AISC 358 (Prequalified Moment Connections)
      - Not covered

Structural Design Standards Relevant for Steel Design

- In order of precedence of controlling requirements for forces and steel design
  - International Building Code (IBC 2012 Edition)
  - Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) (forces only)
  - Specification for Structural Steel Buildings (AISC 360-10) with associated commentaries
  - Seismic Provisions for Structural Steel Buildings (AISC 341-10) with associated commentaries
  - Prequalified Connection (AISC 358-10) with associated commentaries

Structural Steel Buildings—Provisions

A. GENERAL PROVISIONS
A1. Scope
A2. Referenced Specifications, Codes and Standards
A3. Materials
A4. Structural Design Drawings and Specifications

B. GENERAL DESIGN REQUIREMENTS
B1. General Seismic Design Requirements
B2. Loads and Load Combinations
B3. Design Basis
B4. System Type

Materials

R_y and R_t

Expected material strength

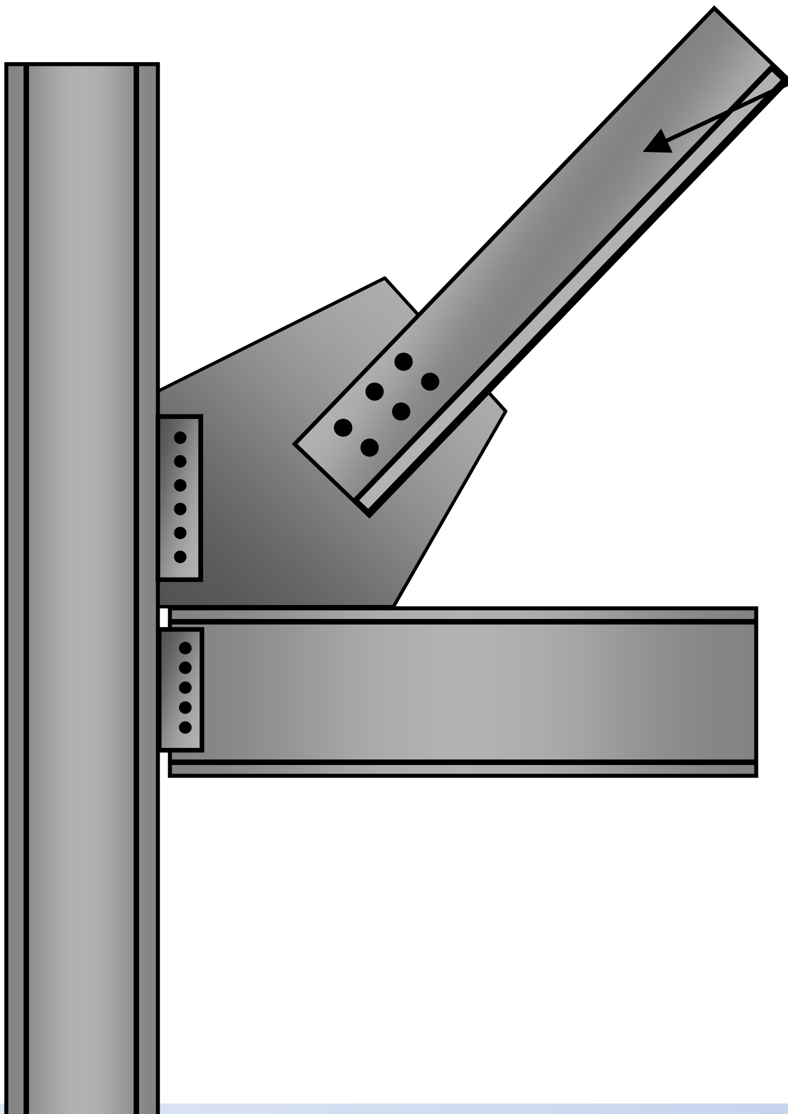
R_y Expected yield stress

- Typically used to calculate demand on adjacent element
- Special case: may be used to compute capacity to resist demand when demand is generated by the same member

R_t Expected rupture stress

- Special case: may be used to compute capacity to resist demand when demand is generated by the same member

Example: SCBF Brace and Brace Connection

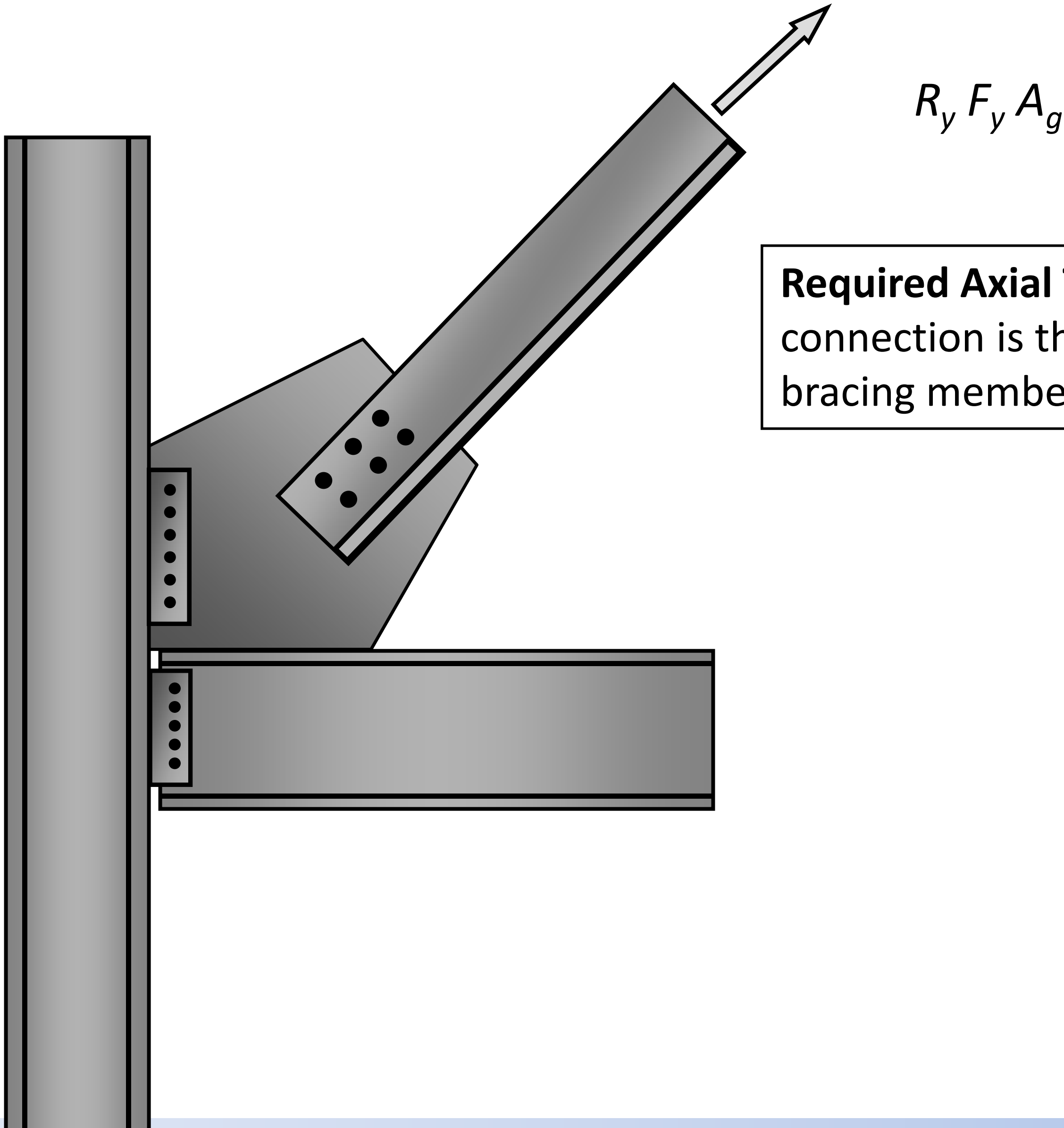


To size brace member:

Required Strength defined by code specified forces (using ASCE-7 load combinations)

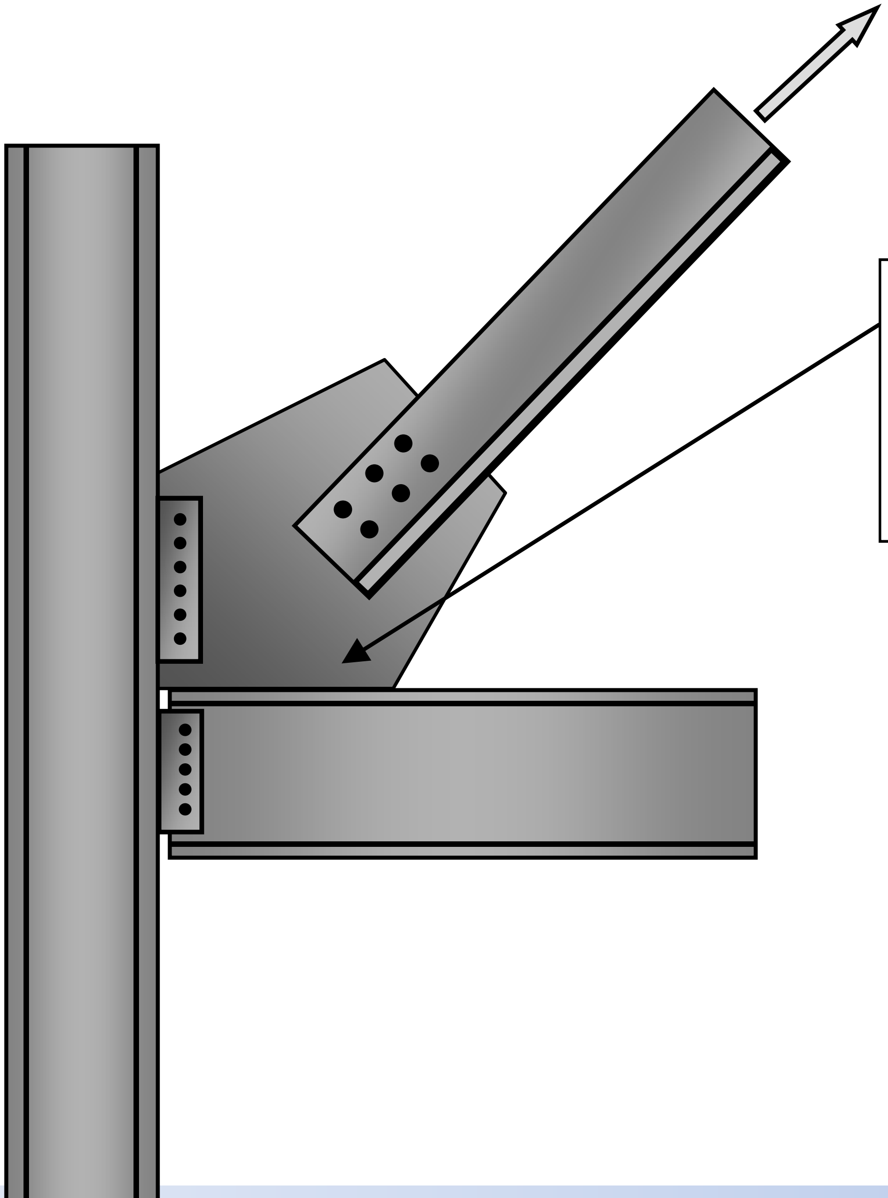
Design Strength of member computed using minimum specified F_y

Example: SCBF Brace and Brace Connection (continued)



Required Axial Tension Strength of brace connection is the expected yield strength of bracing member $= R_y\, F_y\, A_g$

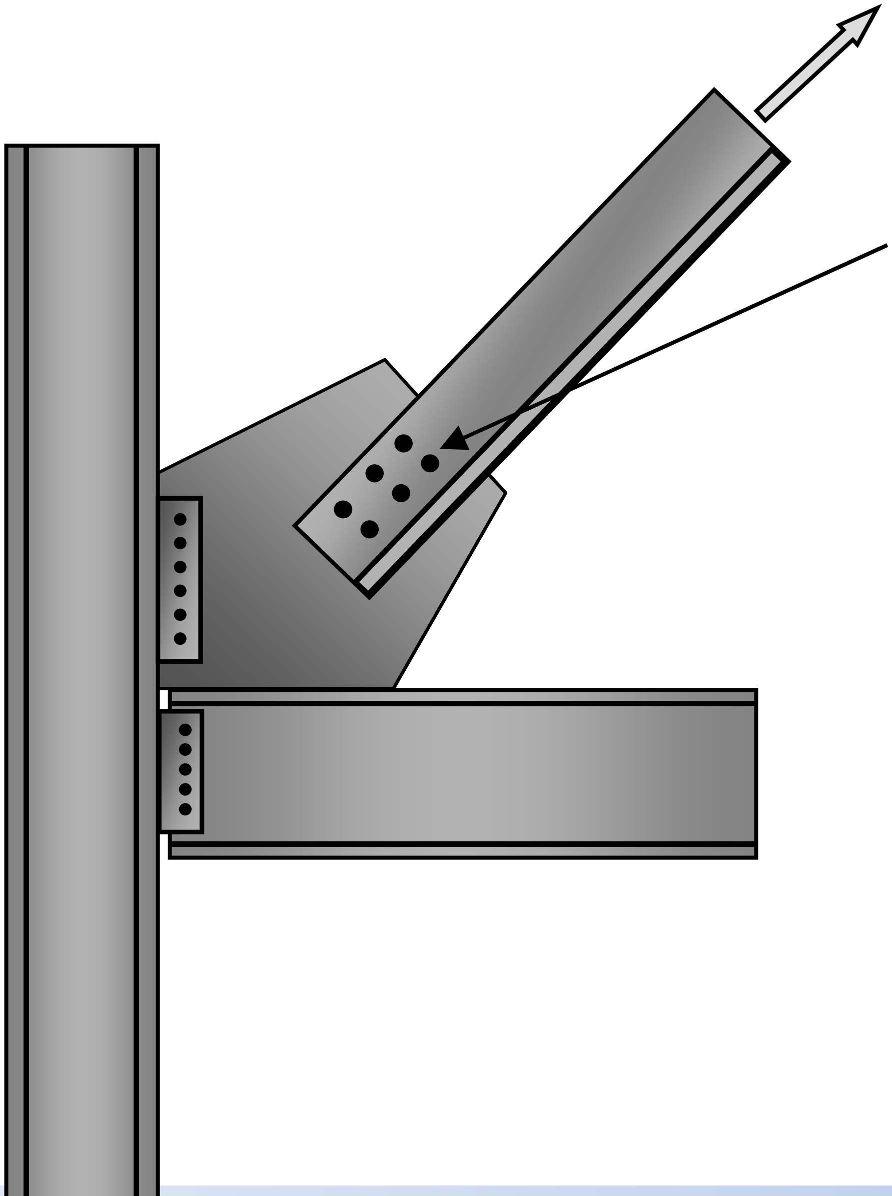
Example: SCBF Brace and Brace Connection (continued)



Gusset Plate:

Compute design strength using min specified F_y and F_u of gusset plate material

Example: SCBF Brace and Brace Connection (continued)

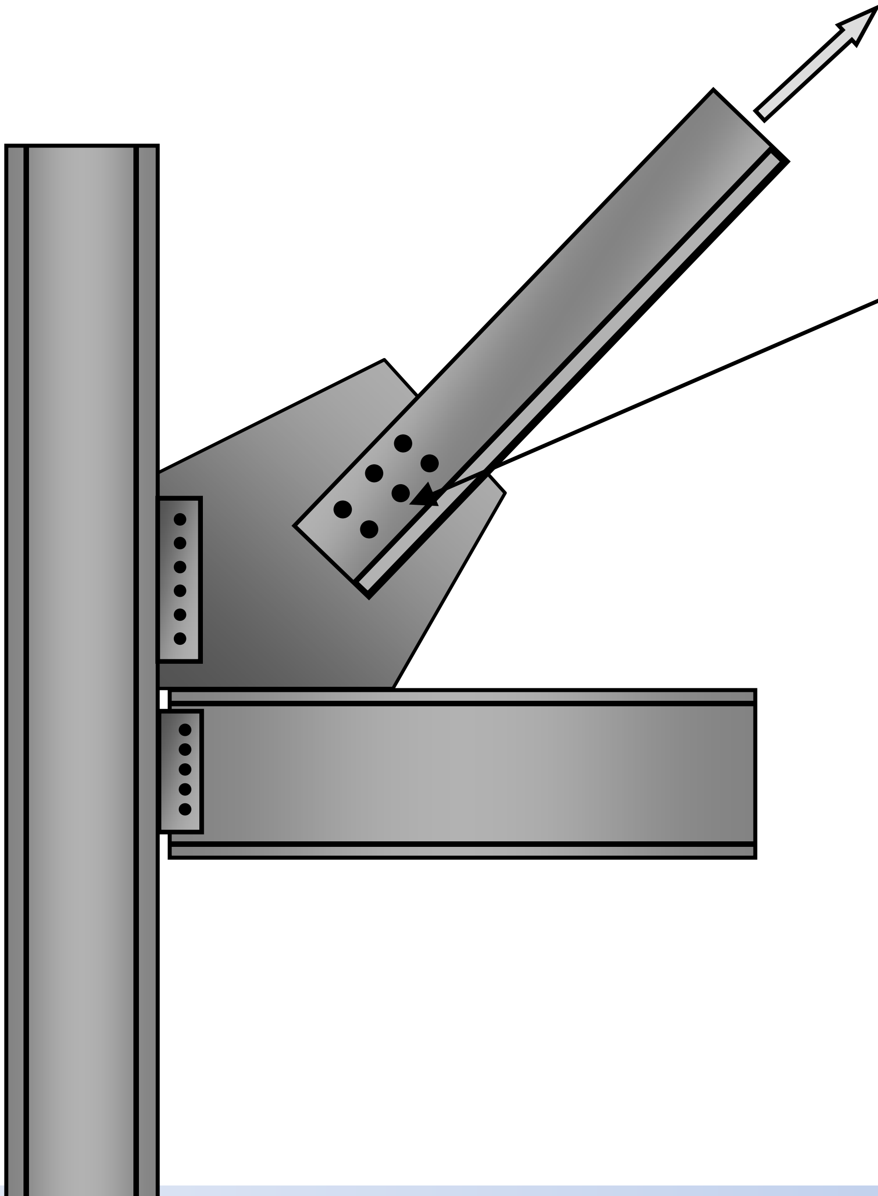


$R_y \, F_y \, A_g$

Bolts:

Compute design shear strength using min specified F_u of bolt

Example: SCBF Brace and Brace Connection (continued)



Net Section Fracture and Block Shear Fracture of Bracing Member:

Compute design strength using expected yield strength, R_yF_y and expected tensile strength, $R_t\ F_u$ of the brace material

Structural Steel Buildings—Provisions

C. ANALYSIS

C1. General

C2. Additional Requirements

C3. Nonlinear Analysis

D. GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS

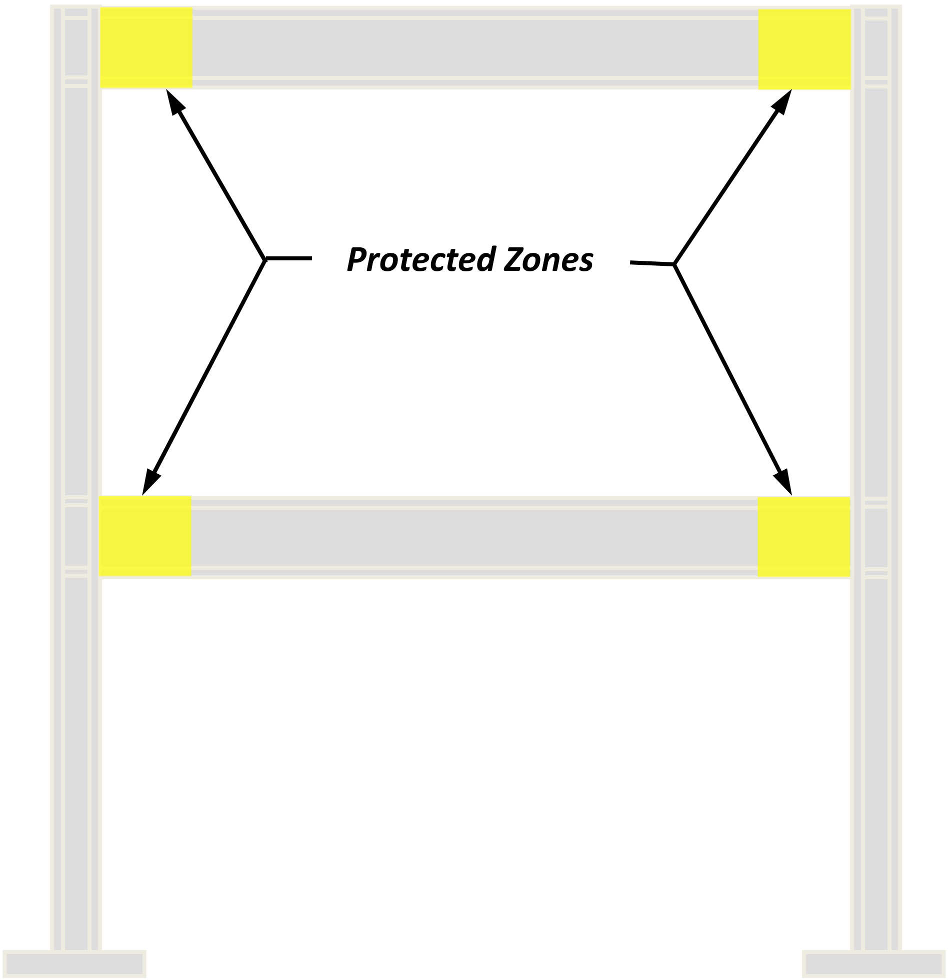
D1. Member Requirements

D2. Connections

D3. Deformation Compatibility of Non-SFRS Members and Connections

D4. H-Piles

Examples of *Protected Zones:* SMF

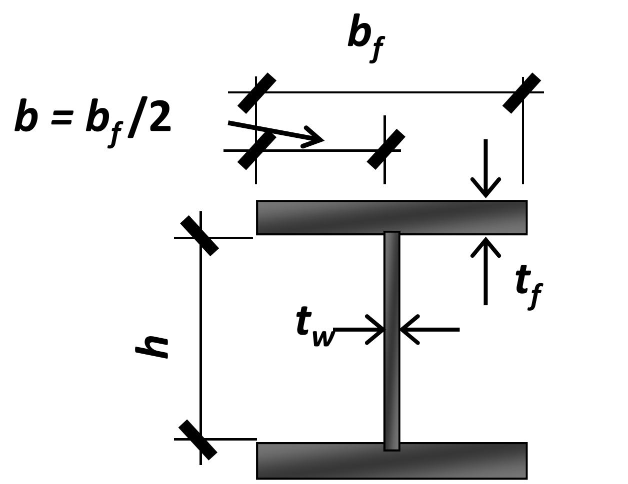


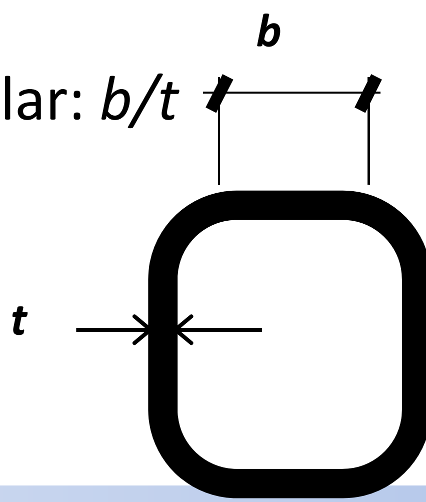
Compactness

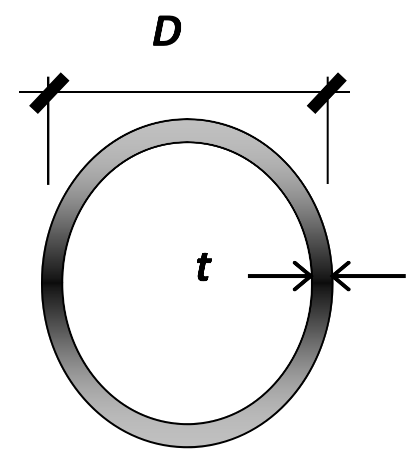
- Compactness

  $\lambda \leq \lambda_{ps}$

  – Per AISC Seismic (AISC 341) Table D1.1
    - As directed by footnotes
    - WF: b/t, h/t_w
    - Square and rectangular: b/t
    - Round: D/t
    - etc.







Compactness

TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements For Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Unstiffened Elements	Flanges of rolled or built-up I-shaped sections, channels and tees; legs of single angles or double angle members with separators; outstanding legs of pairs of angles in continuous contact	b/t	$0.30\sqrt{E/F_y}$	$0.38\sqrt{E/F_y}$	
Unstiffened Elements	Flanges of H-pile sections per Section D4	b/t	$0.45\sqrt{E/F_y}$	not applicable	
Unstiffened Elements	Stems of tees	d/t	$0.30\sqrt{E/F_y}$ [a]	$0.38\sqrt{E/F_y}$	
Stiffened Elements	Walls of rectangular HSS	b/t	$0.55\sqrt{E/F_y}$ [b]	$0.64\sqrt{E/F_y}$ [c]	
Stiffened Elements	Flanges of boxed I-shaped sections and built-up box sections	b/t			
Stiffened Elements	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t			
Stiffened Elements	Webs of rolled or built-up I-shaped sections used as diagonal braces	h/t_w	$1.49\sqrt{E/F_y}$	$1.49\sqrt{E/F_y}$	

TABLE D1.1 (CONTINUED)
Limiting Width-to-Thickness Ratios for Compression Elements For Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_{hd} Highly Ductile Members	Limiting Width-to-Thickness Ratio: λ_{md} Moderately Ductile Members	Example
Stiffened Elements	Webs of rolled or built-up I-shaped sections used as beams or columns[d]	h/t_w	For $C_a \le 0.125$ $2.45\sqrt{E/F_y}(1-0.93C_a)$	For $C_a \le 0.125$ $3.76\sqrt{E/F_y}(1-2.75C_a)$	
Stiffened Elements	Side plates of boxed I-shaped sections used as beams or columns	h/t	For $C_a > 0.125$ $0.77\sqrt{E/F_y}(2.93-C_a)$ $\ge 1.49\sqrt{E/F_y}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	For $C_a > 0.125$ $1.12\sqrt{E/F_y}(2.33-C_a)$ $\ge 1.49\sqrt{E/F_y}$ where $C_a = \frac{P_u}{\phi_c P_y}$ (LRFD) $C_a = \frac{\Omega_c P_a}{P_y}$ (ASD)	
Stiffened Elements	Webs of built-up box sections used as beams or columns	h/t			
Stiffened Elements	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$	not applicable	
Stiffened Elements	Walls of round HSS	D/t	$0.038E/F_y$	$0.044E/F_y$ [e]	
Composite Elements	Walls of rectangular filled composite members	b/t	$1.4\sqrt{E/F_y}$	$2.26\sqrt{E/F_y}$	
Composite Elements	Walls of round filled composite members	D/t	$0.076E/F_y$	$0.15E/F_y$	

[a] For tee shaped compression members, the limiting width-to-thickness ratio for highly ductile members for the stem of the tee can be increased to $0.38\sqrt{E/F_y}$ if either of the following conditions are satisfied:
(1) Buckling of the compression member occurs about the plane of the stem.
(2) The axial compression load is transferred at end connections to only the outside face of the flange of the tee resulting in an eccentric connection that reduces the compression stresses at the tip of the stem.

[b] The limiting width-to-thickness ratio of flanges of boxed I-shaped sections and built-up box sections of columns in SMF systems shall not exceed $0.6\sqrt{E/F_y}$.

[c] The limiting width-to-thickness ratio of walls of rectangular *HSS* members, flanges of boxed I-shaped sections and flanges of built-up box sections used as beams or columns shall not exceed $1.12\sqrt{E/F_y}$.

[d] For I-shaped beams in SMF systems, where C_a is less than or equal to 0.125, the limiting ratio h/t_w shall not exceed $2.45\sqrt{E/F_y}$. For I-shaped beams in IMF systems, where C_a is less than or equal to 0.125, the limiting width-to-thickness ratio shall not exceed $3.76\sqrt{E/F_y}$.

[e] The limiting diameter-to-thickness ratio of round *HSS* members used as beams or columns shall not exceed $0.07E/F_y$.

D1.4a Column Strength

- Required axial strength in the absence of any applied moment is the *amplified seismic load*
- This need not exceed:

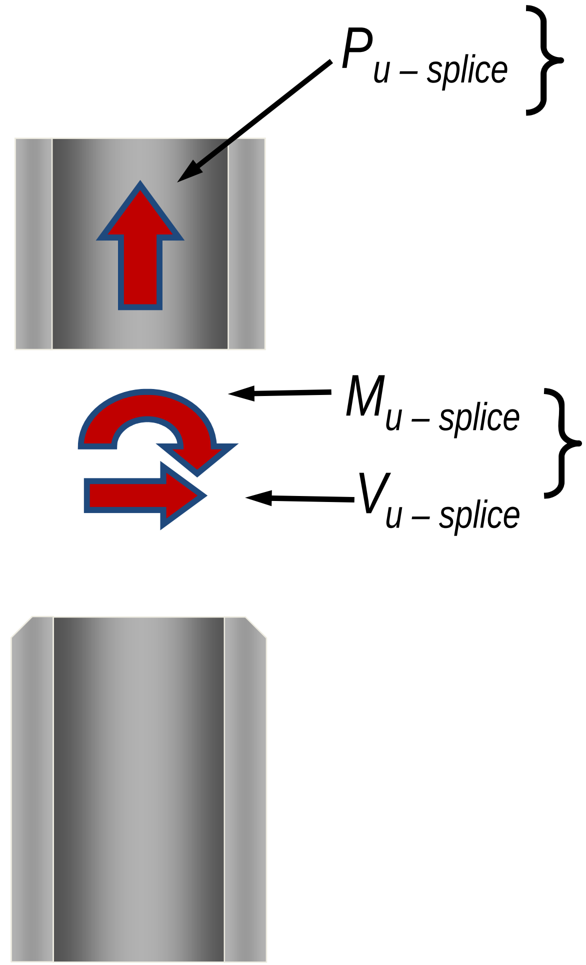
$(1.2 + 0.2\ S_{DS})\ D + \Omega_o\ Q_E + 0.5L + 0.2S$

$(0.9 - 0.2\ S_{DS})\ D + \Omega_o\ Q_E$

a) The maximum load transferred to the column based on expected, strain-hardened strengths of the connecting beam or brace elements

b) The limit as determined from the resistance of the foundation to overturning uplift

D2.5 Column Splices



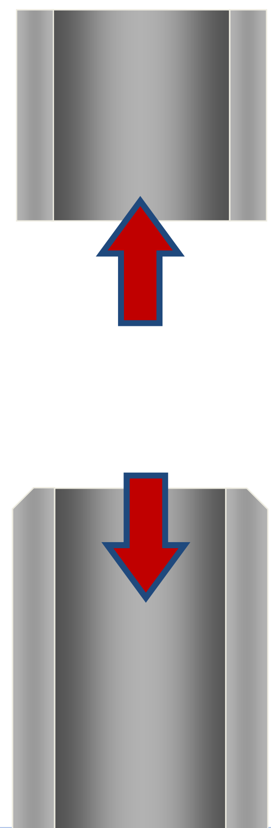
Based on amplified seismic loads or capacity design analysis

Based on code level forces (except as required for SMF, IMF, SCBF, BRBF)

The required strength of column splices shall equal the required strength of columns, including that determined from Section 8.3

D2.5 Column Splices

Welded column splices subjected to net tension when subjected to amplified seismic loads, shall satisfy both of the following requirements:



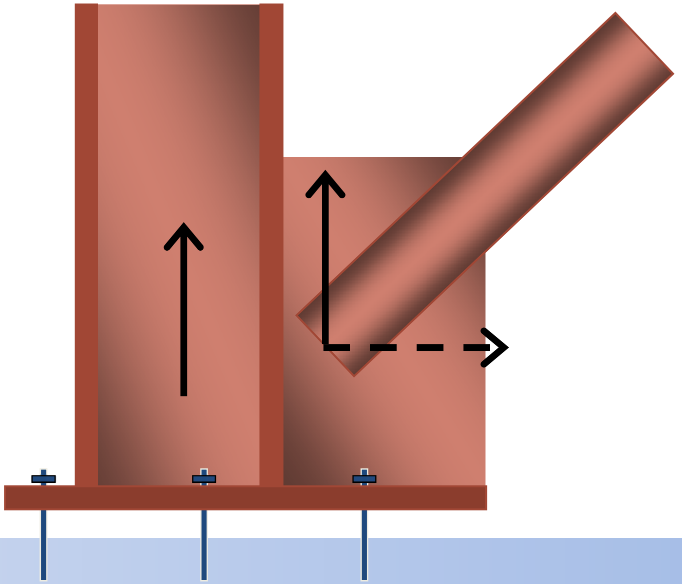
1. If partial joint penetration (PJP) groove welded joints are used, the design strength of the PJP welds shall be at least 200 percent of the required strength; and

2. The design strength of each flange splice shall be at least 0.5 R_y F_y A_f for the smaller flange

D2.6 Column Bases: Required Axial Strength

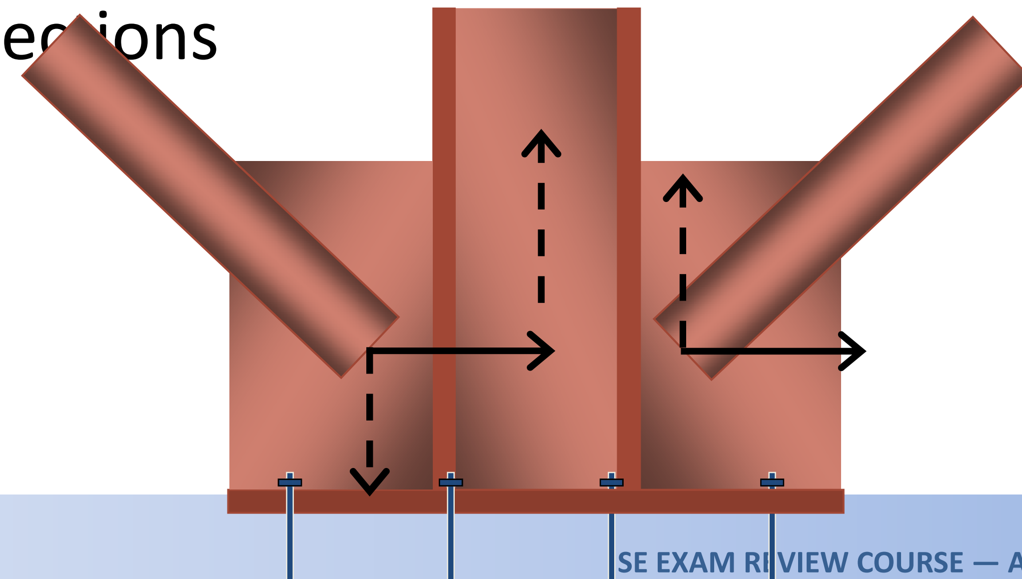
- Required axial strength of column bases and attachment to foundation is summation of vertical components of required strength of steel elements connected to column base

Example: Vertical components = column axial load plus vertical component of brace



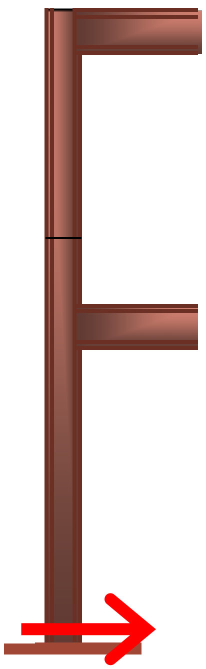
D2.6 Column Bases: Required Shear Strength

- Required shear strength of column bases and attachments to foundation is sum of horizontal components of required strength of steel elements connected to column base
  - For diagonal bracing, use required strength of bracing connections



D2.6 Column Bases: Required Shear Strength

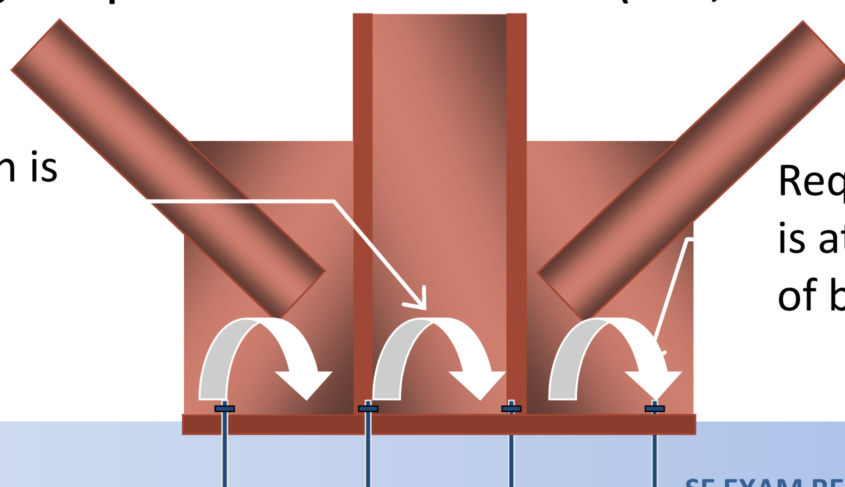
- Required shear strength of column bases and attachments to foundation
  - For columns: horizontal component V_{col} not less than
    - $2R_yF_yZ_x/H$
    - Shear calculated using load combinations of code load including amplified seismic load (i.e., use Ω_o)



D2.6 Column Bases: Required Flexural Strength

- Required flexural strength of column bases and attachments to foundation
  - For columns: required flexural strength greater than
    - $1.1R_yF_yZ$
    - Moment calculated using load combinations of code including amplified seismic load (i.e., use Ω_o)

Required flexural strength is at least $1.1R_yF_yZ$ (LRFD) or $(1.1/1.5)\ R_yF_yZ$ (ASD)



Required flexural strength is at least required strength of bracing connections

Structural Steel Buildings—Provisions

E. MOMENT-FRAME SYSTEMS

E1. Ordinary Moment Frames

E2. Intermediate Moment Frames

E3. Special Moment Frames

E4. Special Truss Moment Frames

E5. Ordinary Cantilever Column Systems

E6. Special Cantilever Column Systems

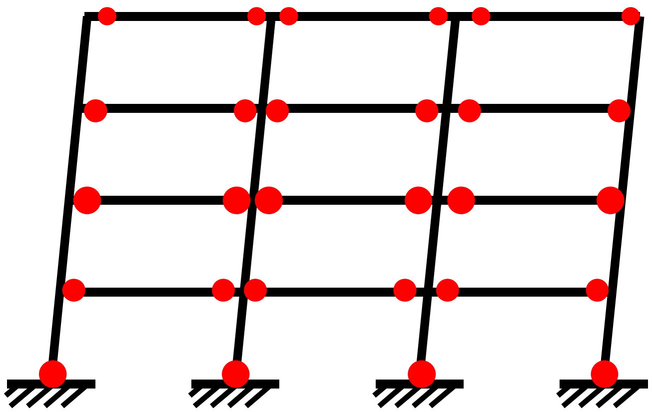
E3. Special Moment Frames (SMF)

1. Scope
2. Basis of Design
3. Analysis
4. Ductile Elements
5. System Requirements
6. Members
7. Connections

Fundamental Approach to SMF

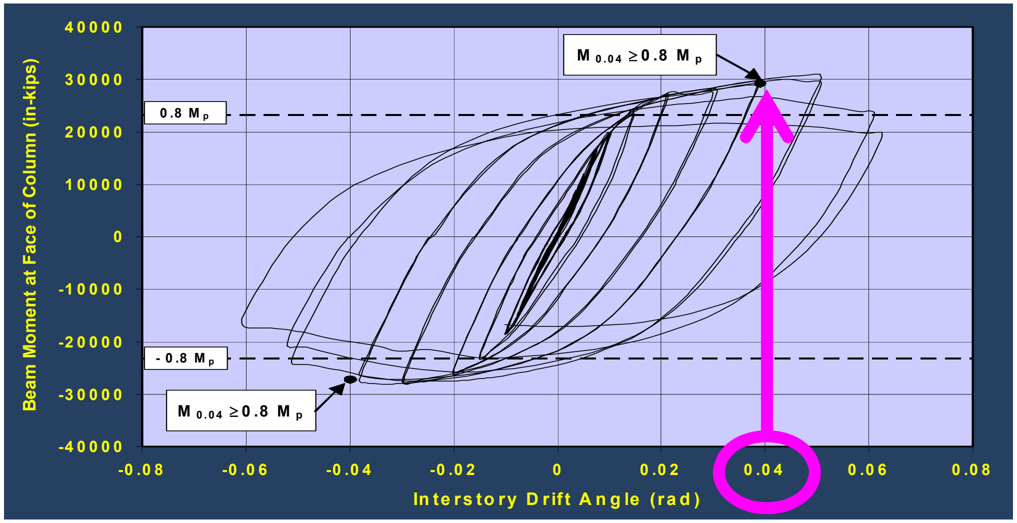
SMF provide stable inelastic drift capacity through beam hinging

- Provide for ductile hinging
  - Tested or prequalified connection
  - Lateral bracing
  - Compactness
- Design elements outside of hinge for forces corresponding to beam hinging
  - Columns
  - Beam shear



E3.2 Basis of Design

- All beam-to-column connections in SLRS shall satisfy
  - Measured flexural resistance of connection, at face of column, is at least 80% of M_p of connected frame beam at interstory drift angle of 0.04 radian



E3.4 Column-Beam Moment Ratio

$$\boxed{\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0}$$

$\sum M^*_{pc}$ = the sum of the moments in the column above and below the connection

It is permitted to take $\sum M^*_{pc} = \sum Z_c\,(F_{yc} - P_{uc}/A_g)$

M^*_{pc} is based on minimum specified yield stress of column

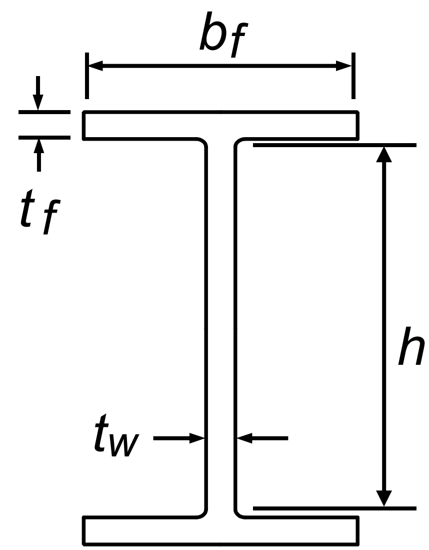
$\sum M^*_{pb}$ = the sum of the moments in the beams at the intersection of the beam and column centerlines.

M^*_{pb} is based on expected yield stress of beam and includes allowance for strain hardening

$$\boxed{M^*_{pb} = 1.1\, R_y\, M_p + V_{beam}\,(s_h + d_{col}/2)}$$

E3.5 Beam and Column Limitations

Beam and column sections must satisfy the width-thickness limitations for *highly ductile members* given in Table D1.1



Beam and Column Flanges

$$\frac{b_f}{2t_f} \le 0.30\sqrt{\frac{E_s}{F_y}}$$

Beam Web

$$\frac{h}{t_w} \le 2.45\sqrt{\frac{E_s}{F_y}}$$

Column Web

$$\frac{P_u}{\phi P_y} > 0.125$$

$$\frac{h}{t_w} \le 0.77\sqrt{\frac{E_s}{F_y}}\left[2.93 - \frac{P_u}{\phi P_y}\right] > 1.49\sqrt{\frac{E_s}{F_y}}$$

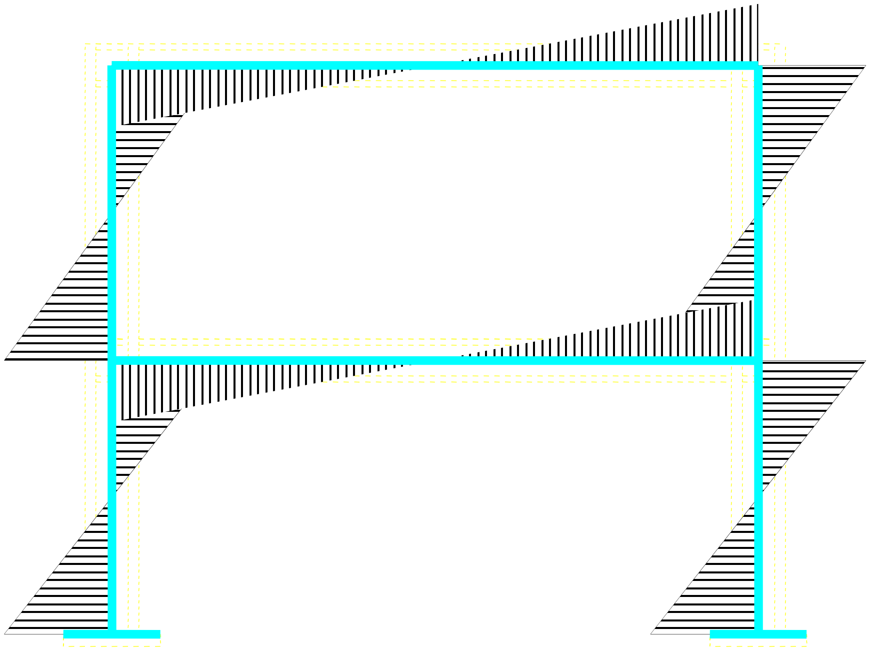
E3.6c Conformance Demonstration

Demonstrate conformance with requirements of Sect. E3.2 by one of the following methods:

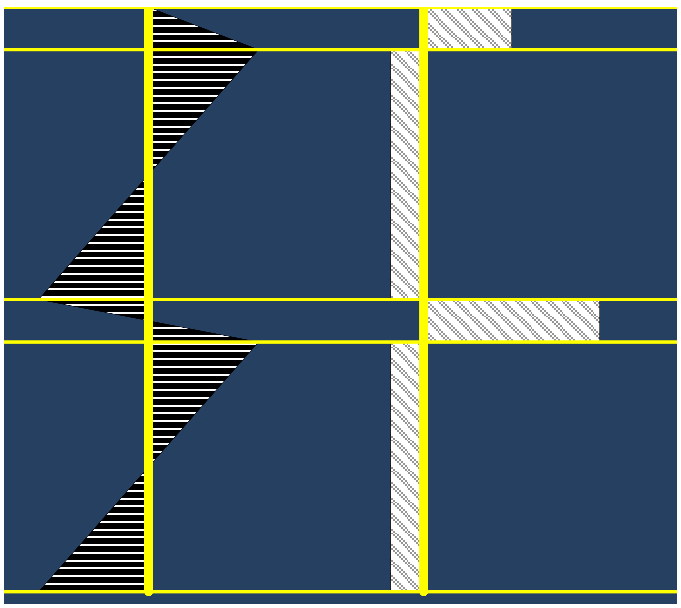
I. Use connections prequalified for SMF in accordance with Section J.1 (AISC 358 or similar)

II. Conduct qualifying cyclic tests in accordance with Section J.2

Accounting for Member Depth: Panel Zone Shear



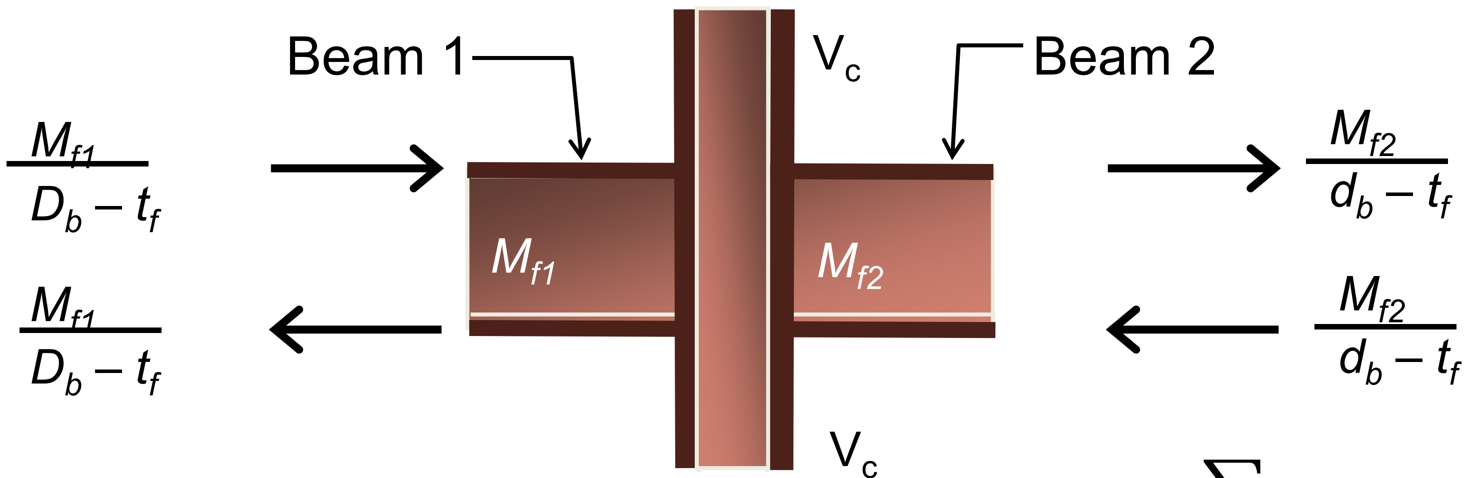
Centerline Moment Diagram



Column Moment Shear

Panel Zone Shear

- Required strength (shear) based on demands generated by beams framing into column



Panel Zone Required Shear Strength = $R_u = \dfrac{\sum M_f}{(d_b - t_f)} - V_c$

Panel Zone Shear

To compute nominal shear strength, R_v, of panel zone

When $P_u < 0.75\ P_y$ in column

(AISC Spec EQ J10-12)

$$R_v = 0.6F_y d_c t_p \left[1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right]$$

E3.7 Connections

- The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect *E:*

$$E = 2\,[\,1.1\,R_y\,M_p\,]\,/\,L_h \qquad \text{(E3-6)}$$

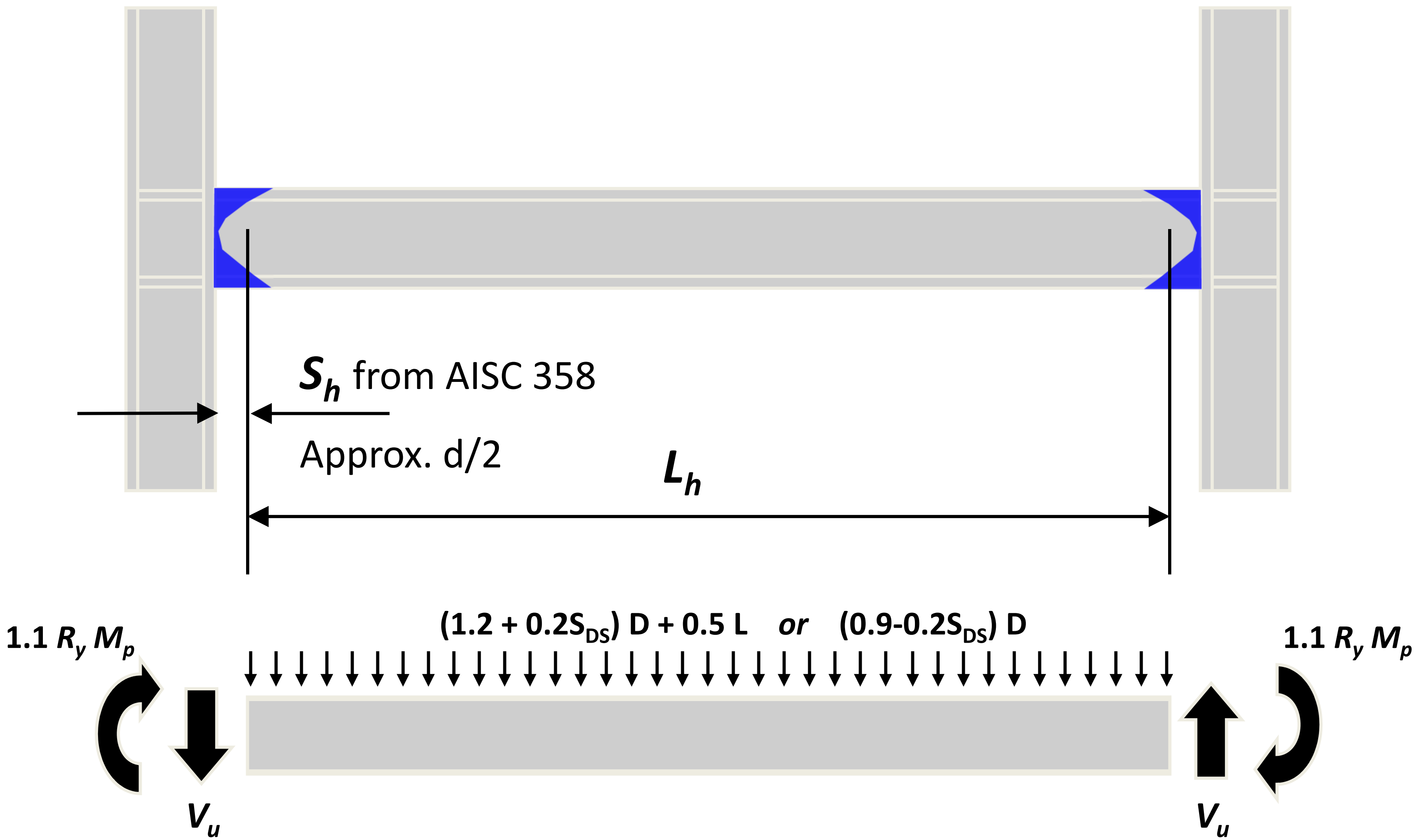
where:

R_y = ratio of the expected yield strength to the minimum specified yield strength

$M_p = Z\,F_y$ = plastic flexural strength

L_h = distance between plastic hinge locations

Required Shear Strength of Beam-to-Column Connection



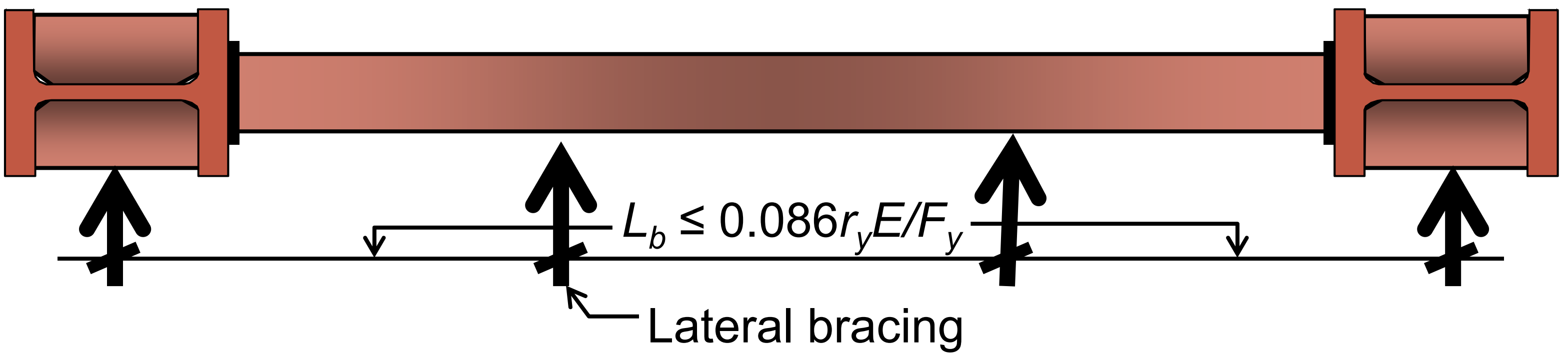
$$V_u = 2\ [\ 1.1\ R_y\ M_p\ ]\ /\ L_h\ +\ V_{gravity}$$

Lateral Bracing of Beams

- Both flanges of beams shall be laterally braced
- Unbraced length between lateral braces shall not exceed

$$L_b = 0.086 r_y E/F_y \text{ (SMF; ×2 for IMF)}$$

- Braces need to possess sufficient strength and stiffness (Appendix 6 of *Specification*)



Structural Steel Buildings—Provisions

F. BRACED-FRAME AND SHEAR-WALL SYSTEMS

F1. Ordinary Concentrically Braced Frames

F2. Special Concentrically Braced Frames

F3. Eccentrically Braced Frames

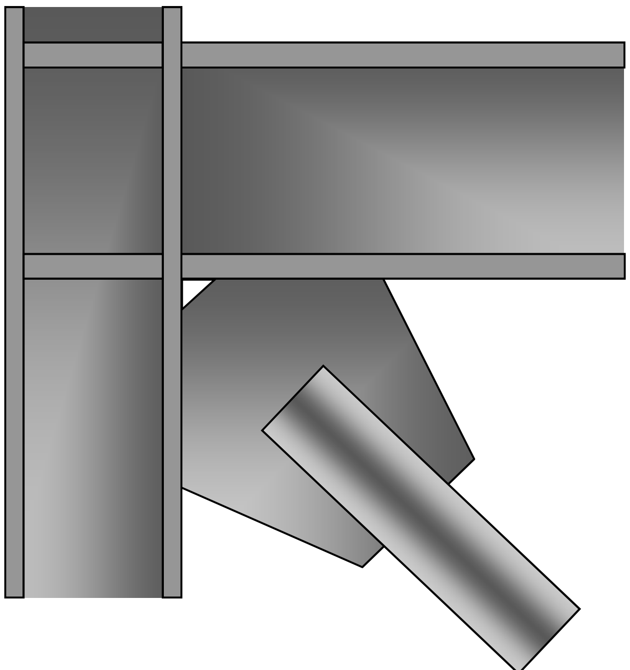
F4. Buckling-Restrained Braced Frames

F5. Special Plate Shear Walls

F2 Special Concentrically Braced Frames (SCBF)

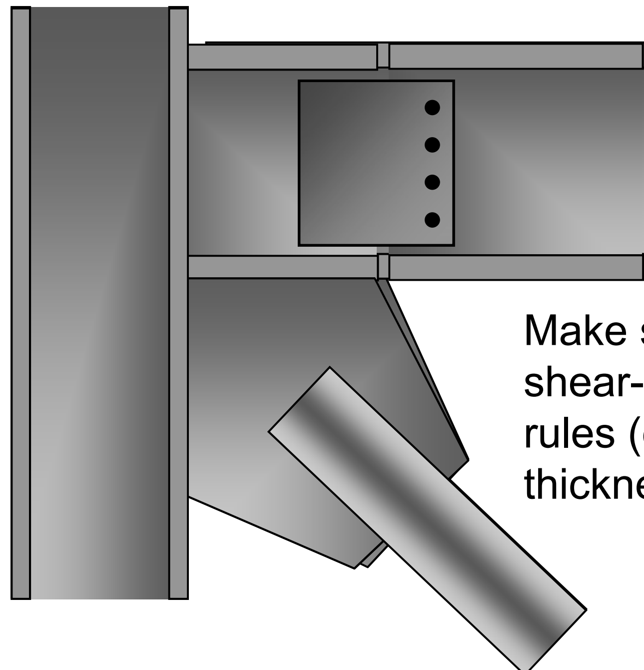
1. Scope
2. Basis of Design
3. Analysis
4. Ductile Elements
5. System Requirements
6. Members
7. Connections

Fixity of Beam-Column Connection



<u>Rigid Connection</u>

Moments are accounted for in design



Make sure to follow shear-plate design rules (e.g., max. plate thickness)

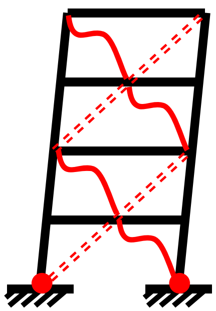
<u>Connection Similar to Shear Plate</u>

Rotational ductility provided via bolt deformation

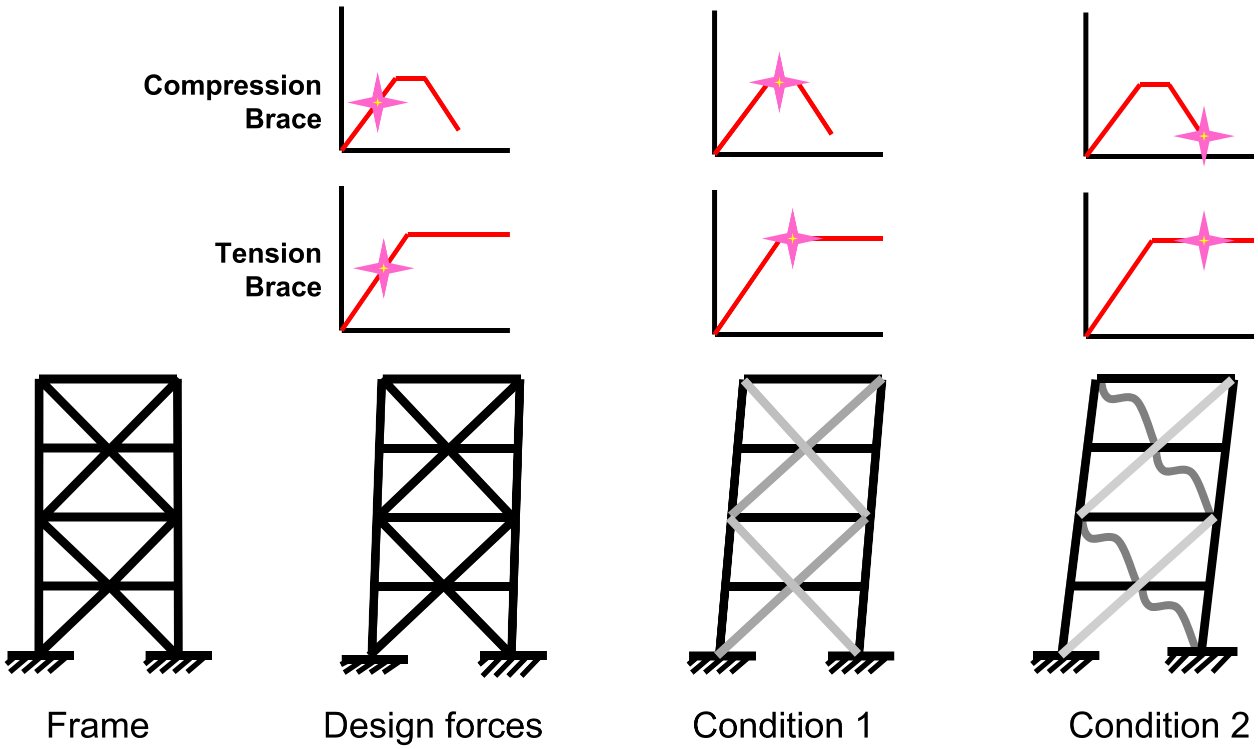
Fundamental Approach to SCBF

SCBF provide stable inelastic drift capacity through brace buckling and yielding

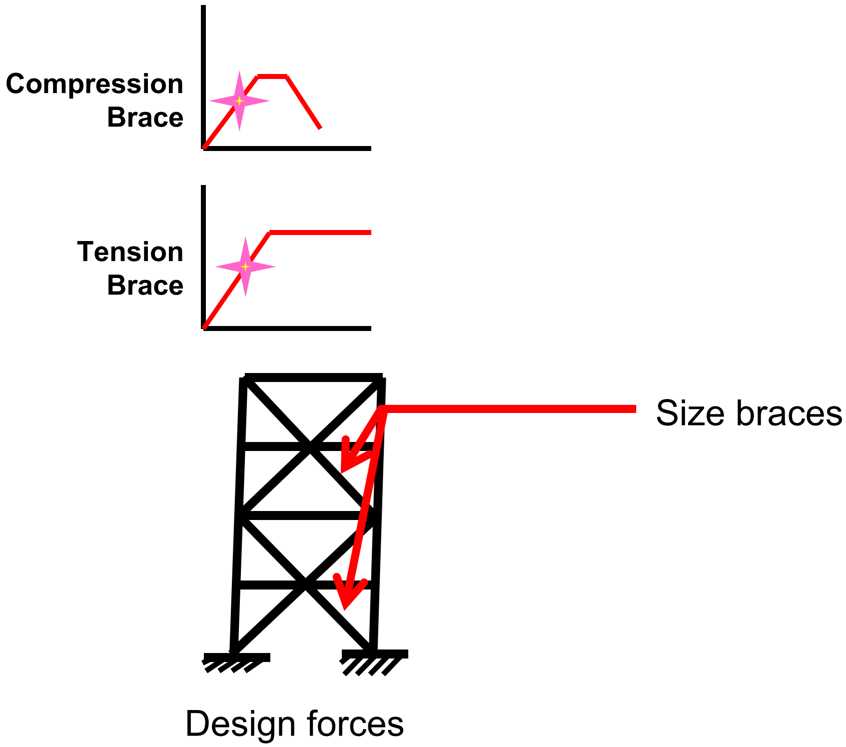
- Most post-elastic resistance is in tension-yielding brace
- Compression brace works in tension braced upon load reversal
- Provide for ductile brace behavior
  - Accommodate buckling
  - Compactness
- Design elements outside of hinge for forces corresponding to maximum brace forces
  - Columns
  - Beams
- Provide for good system behavior
  - Balance tension and compression
  - Accommodate redistribution of forces after brace buckling



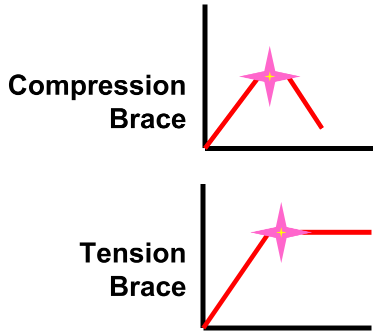
F2.3 Plastic mechanism analyses



Design forces

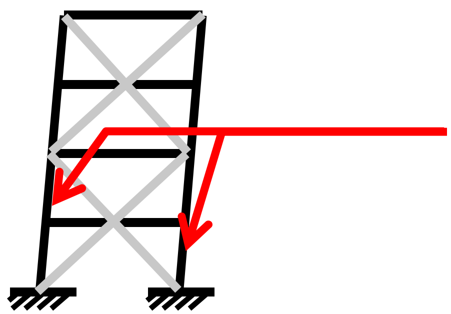


Plastic mechanism analyses



$$C_{max} = 1.14\ F_{cr}(R_yF_y) \leq R_yF_yA$$

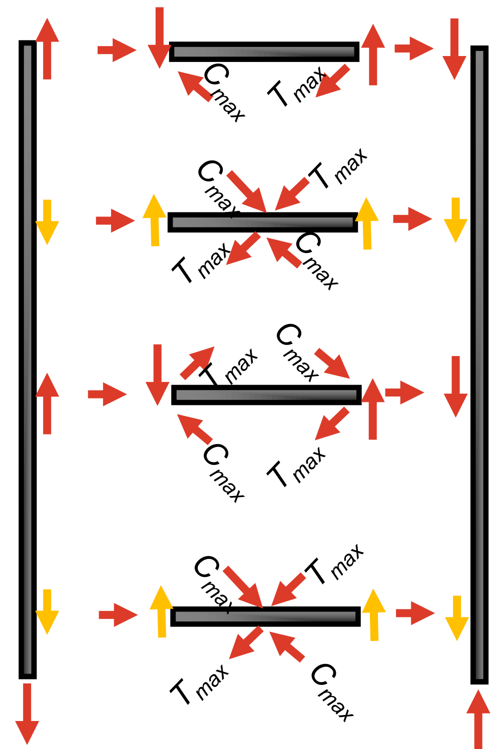
$$T_{max} = R_yF_yA_g$$



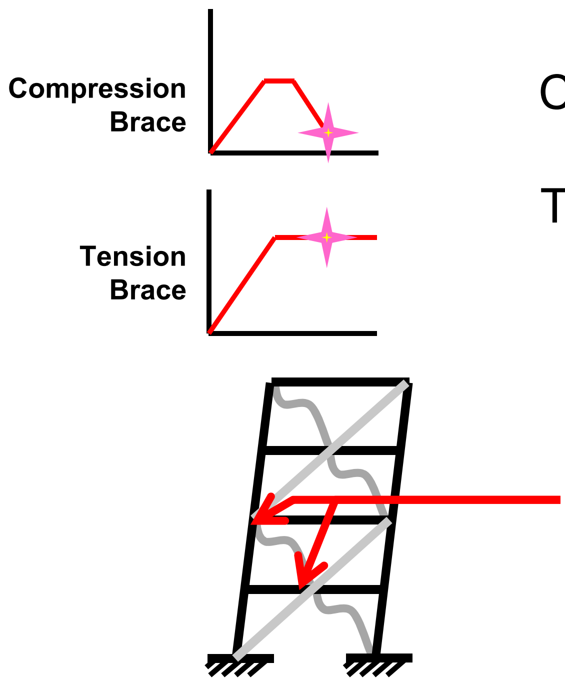
Condition 1

Maximum overturning

Size Columns
Gussets
Base plates



Plastic mechanism analyses



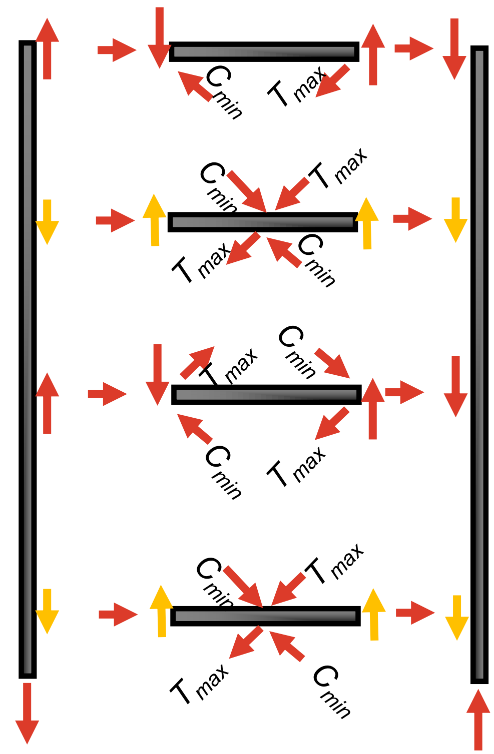
Condition 2

$C_{max} = 0.3\ C_{max}$

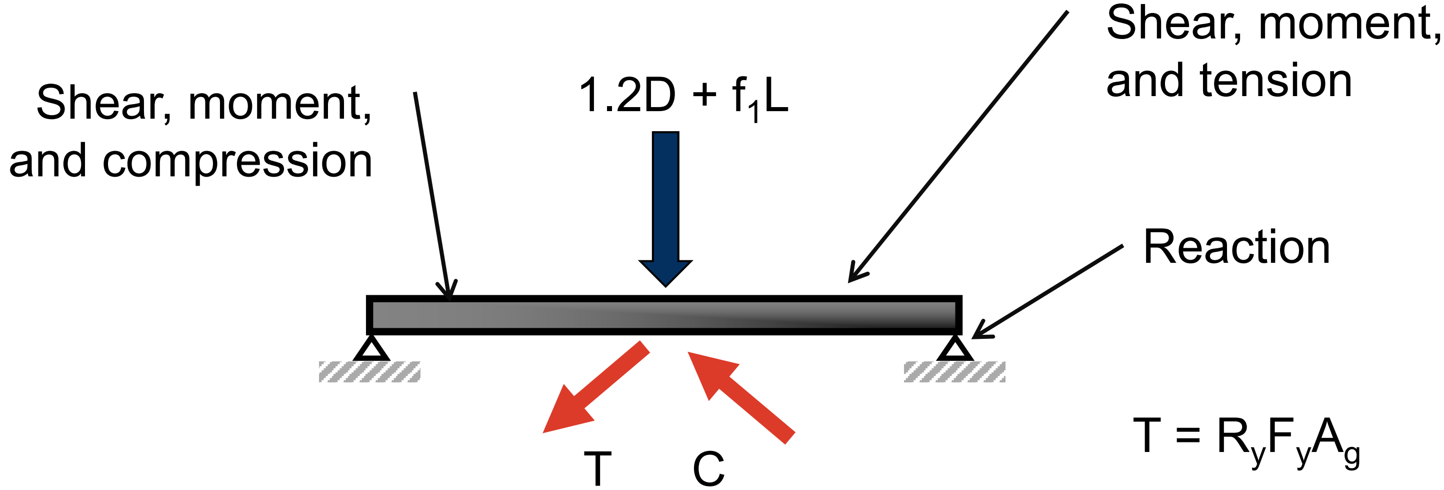
$T_{max} = R_yF_yA_g$

Force redistribution:
Compression braces participate less
Force zig-zags

Size beams
Interior columns



Chevron Configuration

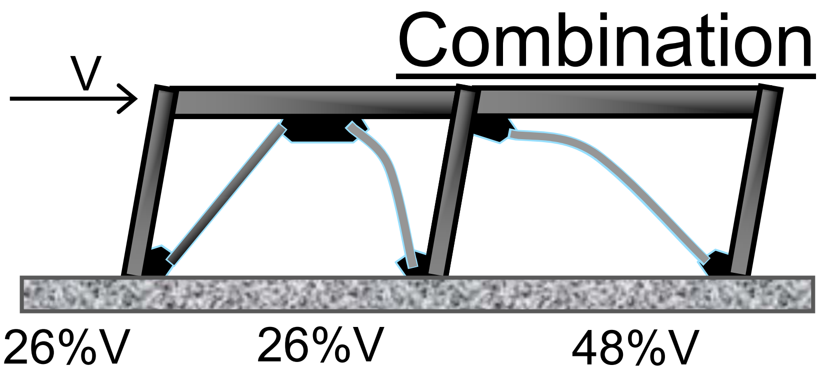


Chevron

$T = R_y F_y A_g$

$C = 0.3P_n$

F2.4a Configurations



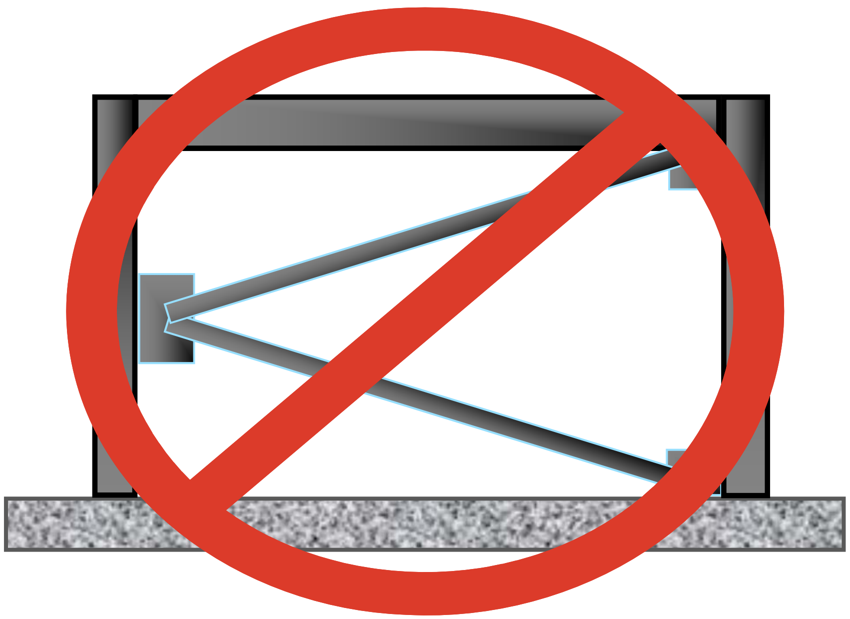
Tension and compression force distribution based on relative stiffness of frame members

0.30V ≤ Compression = 0.74 ≥ 0.7
0.30V ≥ Tension = 0.26 ≤ 0.7

No Good

F2.4 Configurations

K-Bracing



F2.5 Limitations

Column

- *Highly ductile member* (compactness)

Beam

- *Moderately ductile member* (compactness)

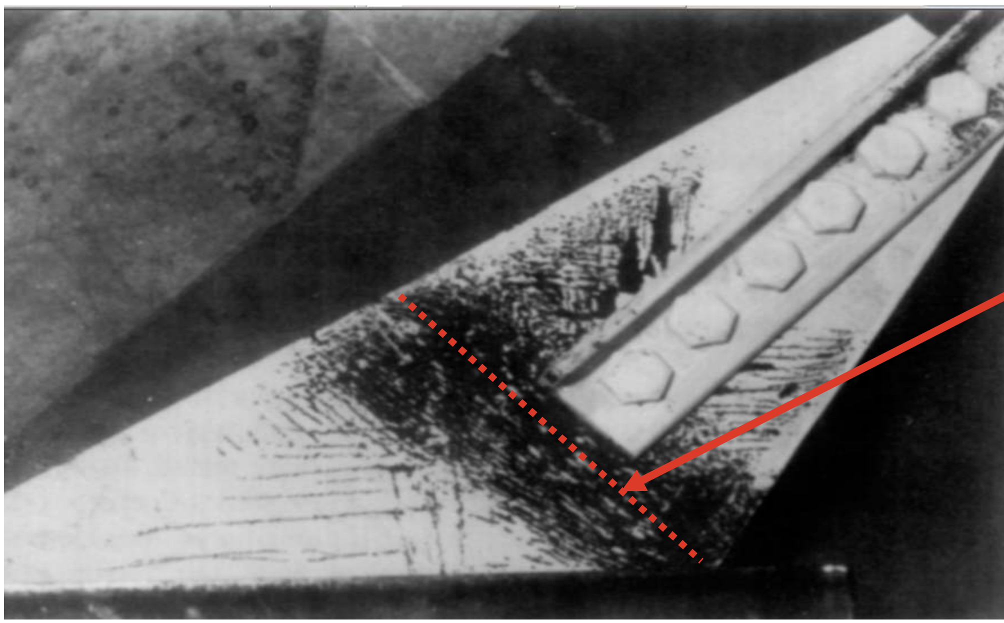
Brace

- *Highly ductile member* (compactness)
- Slenderness
  - ${}^{KL}/_{r} \leq 200$

SCBF Connection Design

- Limit states for brace
  - Brace net section fracture
  - Brace shear fracture
- Limit states for gusset
  - Gusset block shear fracture
  - Gusset tension yield or fracture
  - Gusset failure at column
  - Gusset failure at beam
  - Gusset buckling
- Limit states for beam and column
  - Web yielding
  - Web crippling
  - Web shear
- Limit states for welds
  - Brace-to-gusset weld fracture
  - Gusset-to-beam weld fracture
  - Gusset-to-column weld fracture

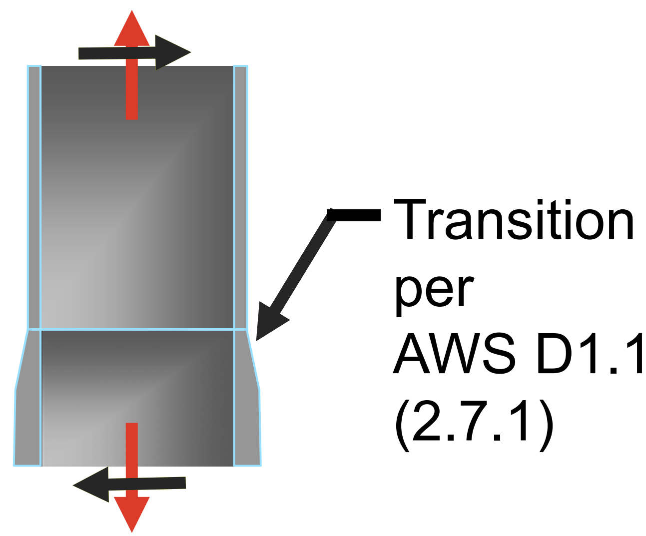
Folding of Gusset (Hinge Zone)



Gusset plate fold line

Design Column Splice

- Axial force: same as column
  - Compression: $1.2D + f_1 L + E_m$
  - Tension: $0.9D \pm E_m$
- AISC 341 Requirements
  - Shear: Shear strength of member
    - Interpreted as $\Sigma M_p/h_c$
  - Moment: Flexural strength of member
    - CJP of flanges typically done
  - Locate splice in middle ⅓ of clear height



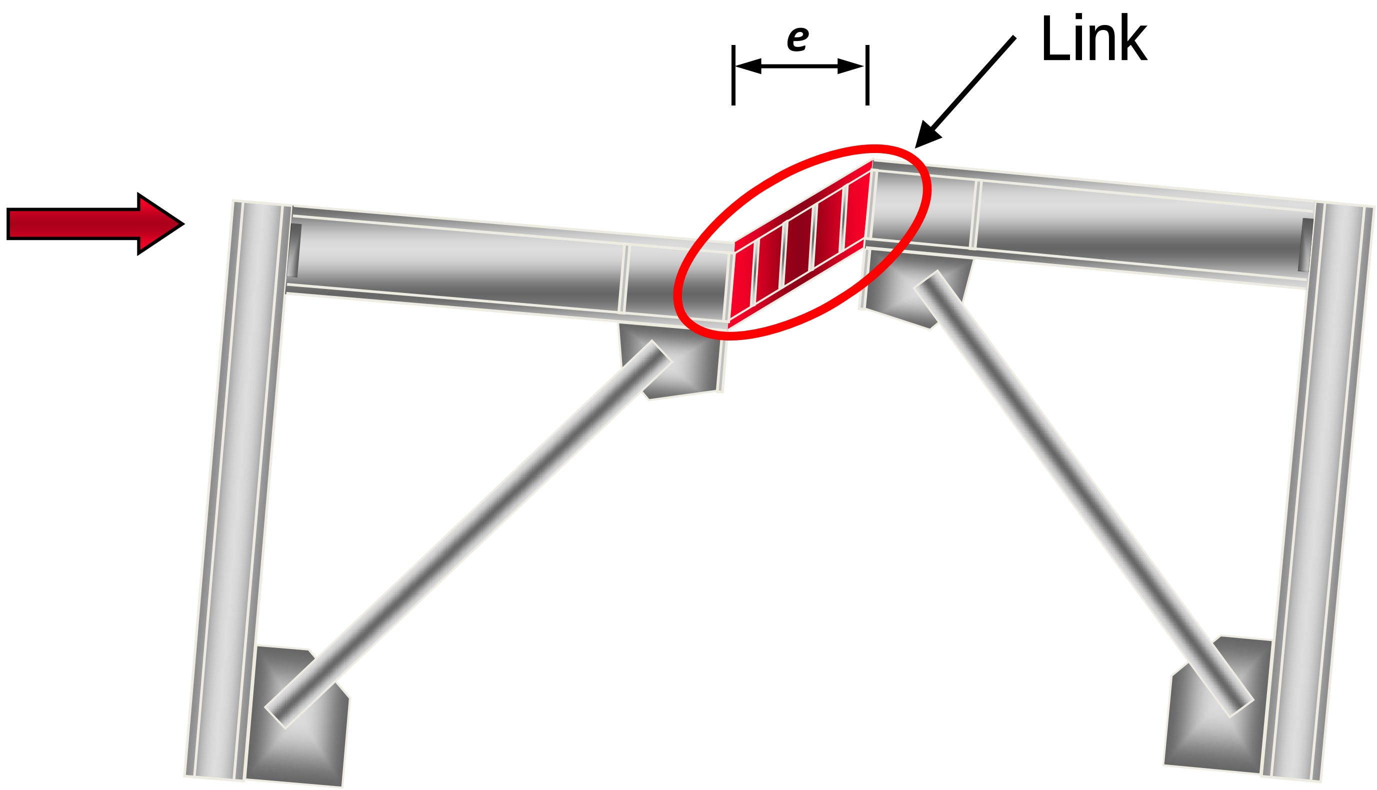
F2 Other Provisions

- F2.4b V- and Inverted-V configurations
    - Lateral bracing of beam
- F2.5c Protected Zone = braces and gussets
- Demand Critical Welds

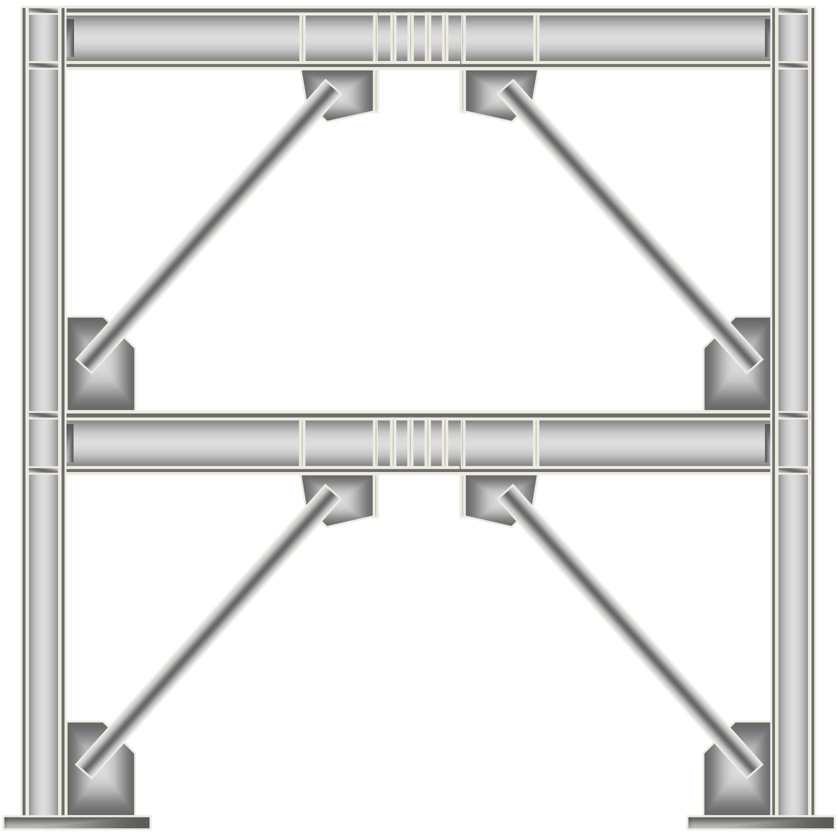
F3. Eccentrically Braced Frames (EBF)

1. Scope
2. Basis of Design
3. Analysis
4. Ductile Elements
5. System Requirements
6. Members
7. Connections

Link is Fuse



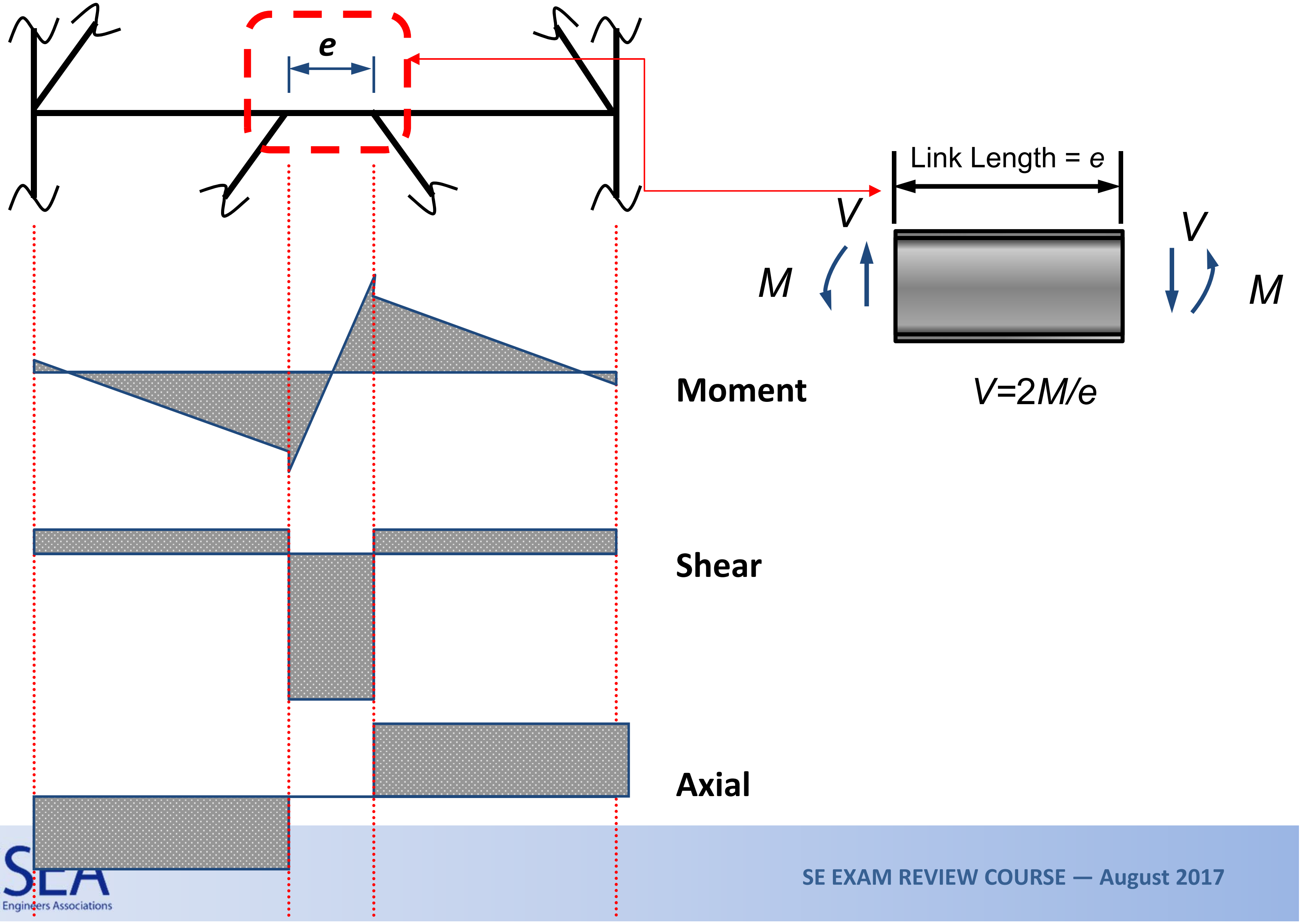
Design of EBFs



<u>General Approach</u>

1. Size links for code levels forces.
2. Size all other members and connections for maximum forces that can be generated by links.
3. Estimate ductility demand on links; check that links can supply the required ductility
4. Detail links to supply high ductility (stiffeners and lateral bracing)

Link Behavior: Forces in Links



Link Behavior: Forces in Links

Shear governed:

$V_n = V_p$

$M(V_n) = ½\, V_p e$

$M(V_n) < M_p$

For this to be true…………... $e < 2M_p/V_p$

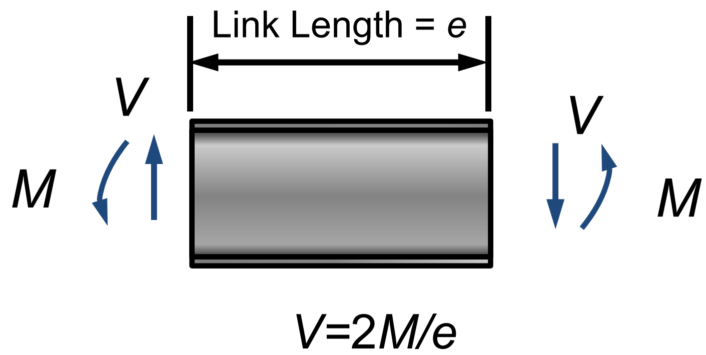
Flexure governed:

$M_n = M_p$

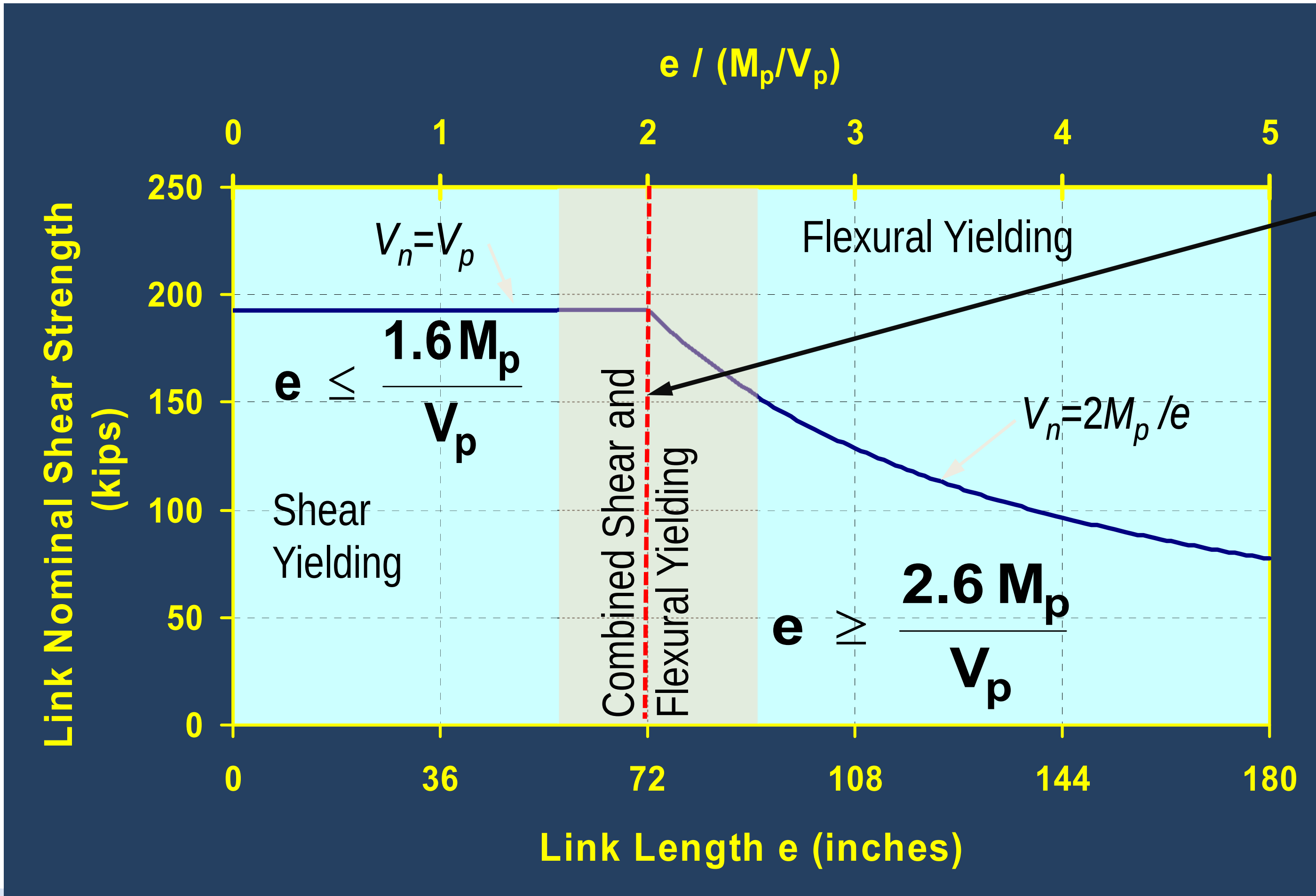
$V(M_n) = 2M_p/e$

$V(M_n) < V_p$

For this to be true…………... $e > 2M_p/V_p$



Link Nominal Shear Strength



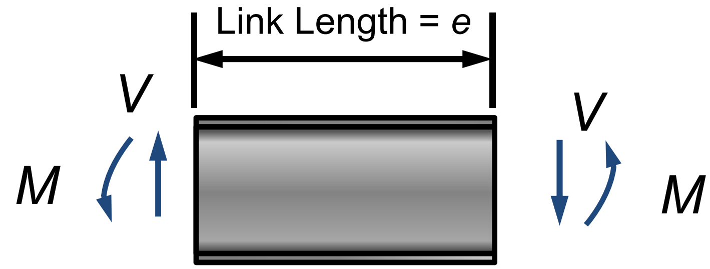
$$e = \frac{2M_p}{V_p}$$

F3.3 Analysis

- The required strengths of the connections, column, diagonal brace, and the beam outside of the link are based on the maximum forces that can be generated by the fully yielded and strain hardened link.

F3.3 Analysis

Determining link ultimate shear and end moment for design of diagonal brace and beam outside of link



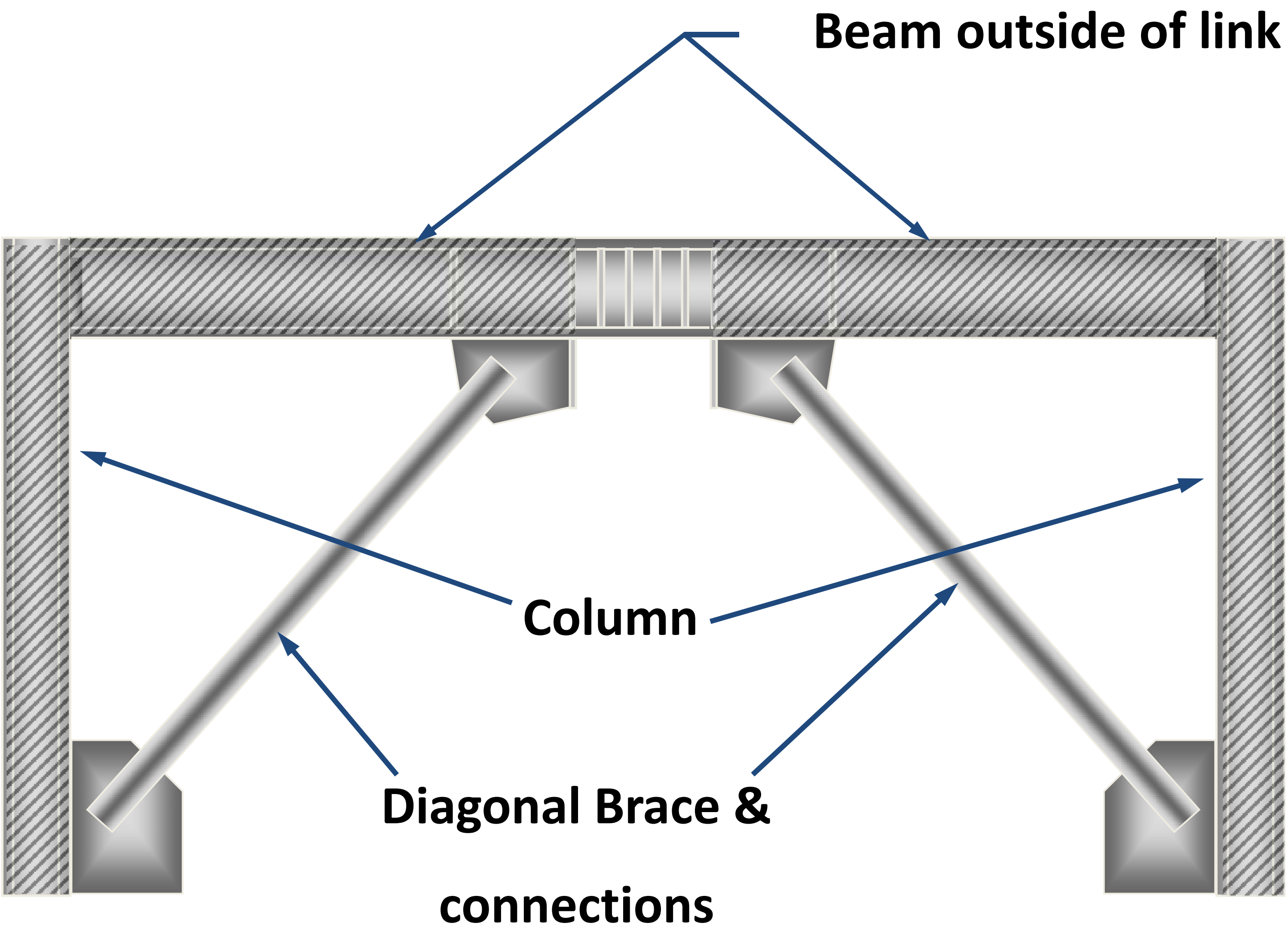
$V=2M/e$

For design of diagonal brace and connections: Take $V_{ult} = 1.25\ R_y\ V_n$

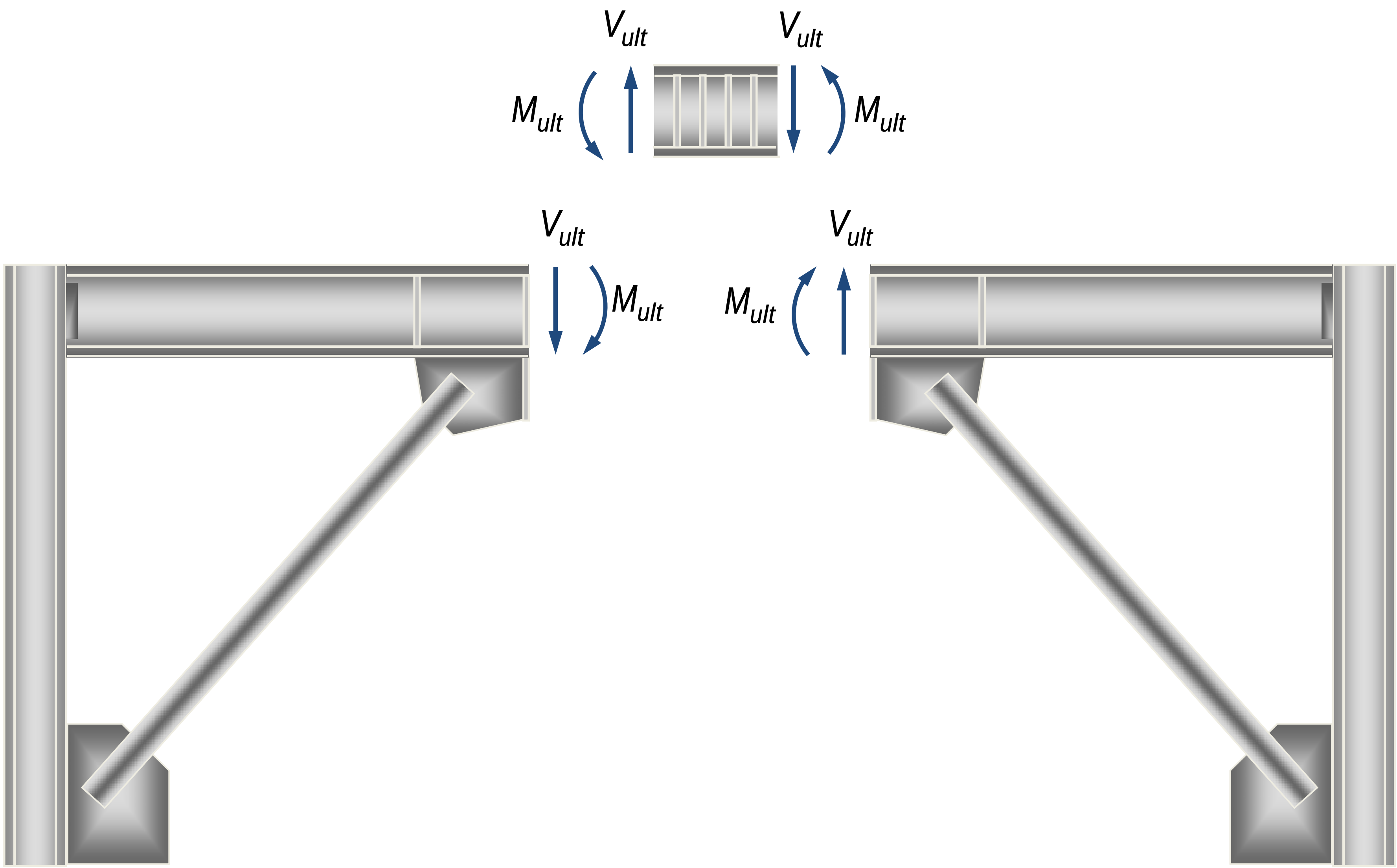
For design of column and beam outside of link: Take $V_{ult} = 1.1\ R_y\ V_n$

V_n = link nominal shear strength = lesser of V_p or $2\ M_p\ /\ e$

F3.3 Analysis

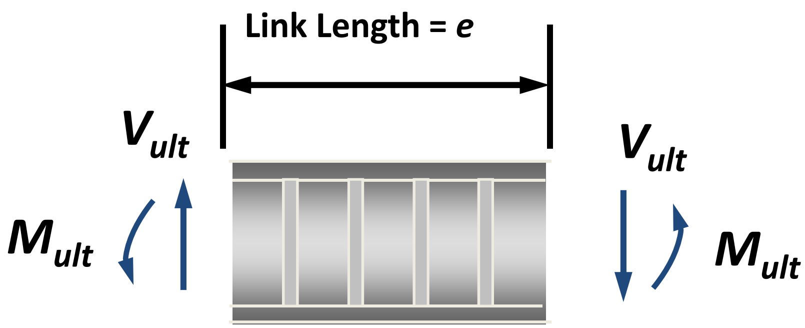


F3.3 Analysis



F3.3 Analysis

Determining link ultimate shear and end moment for design of diagonal brace and beam outside of link



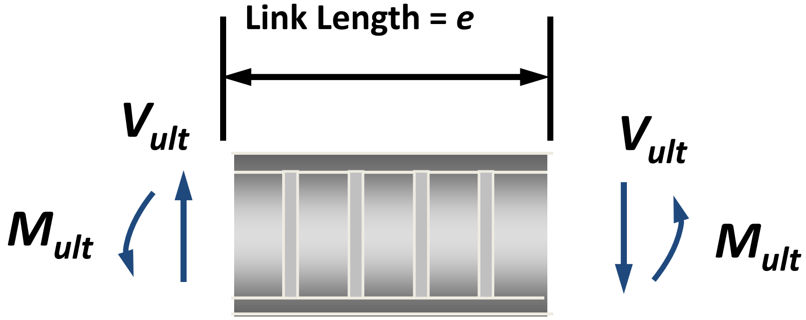
For design of diagonal brace: Take $V_{ult} = 1.25\ R_y\ V_n$

For design of column beam outside of link: Take $V_{ult} = 1.1\ R_y\ V_n$

V_n = link nominal shear strength = lesser of V_p or $2\ M_p / e$

F3.3 Analysis

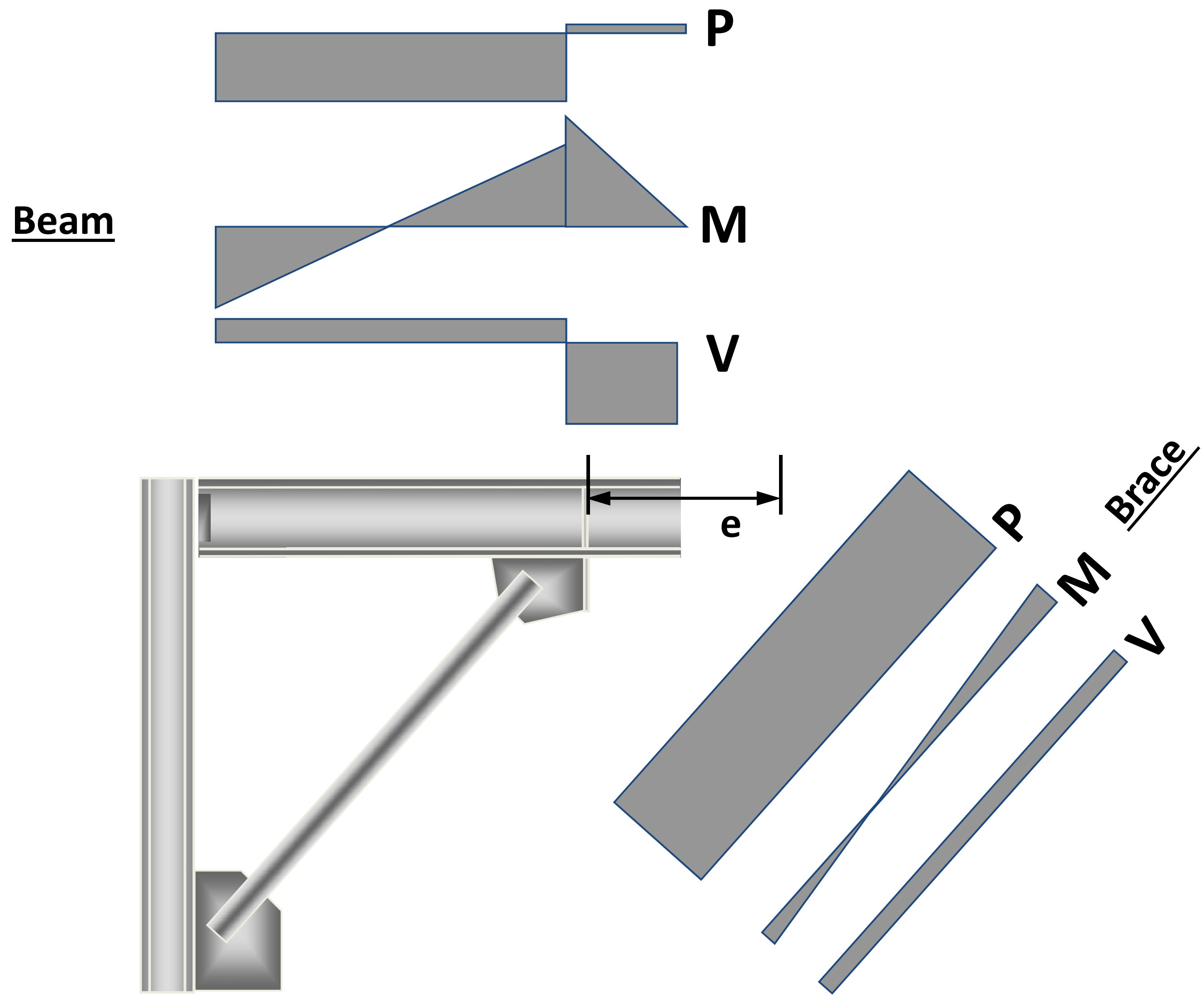
Determining link ultimate shear and end moment for design of diagonal brace and beam outside of link



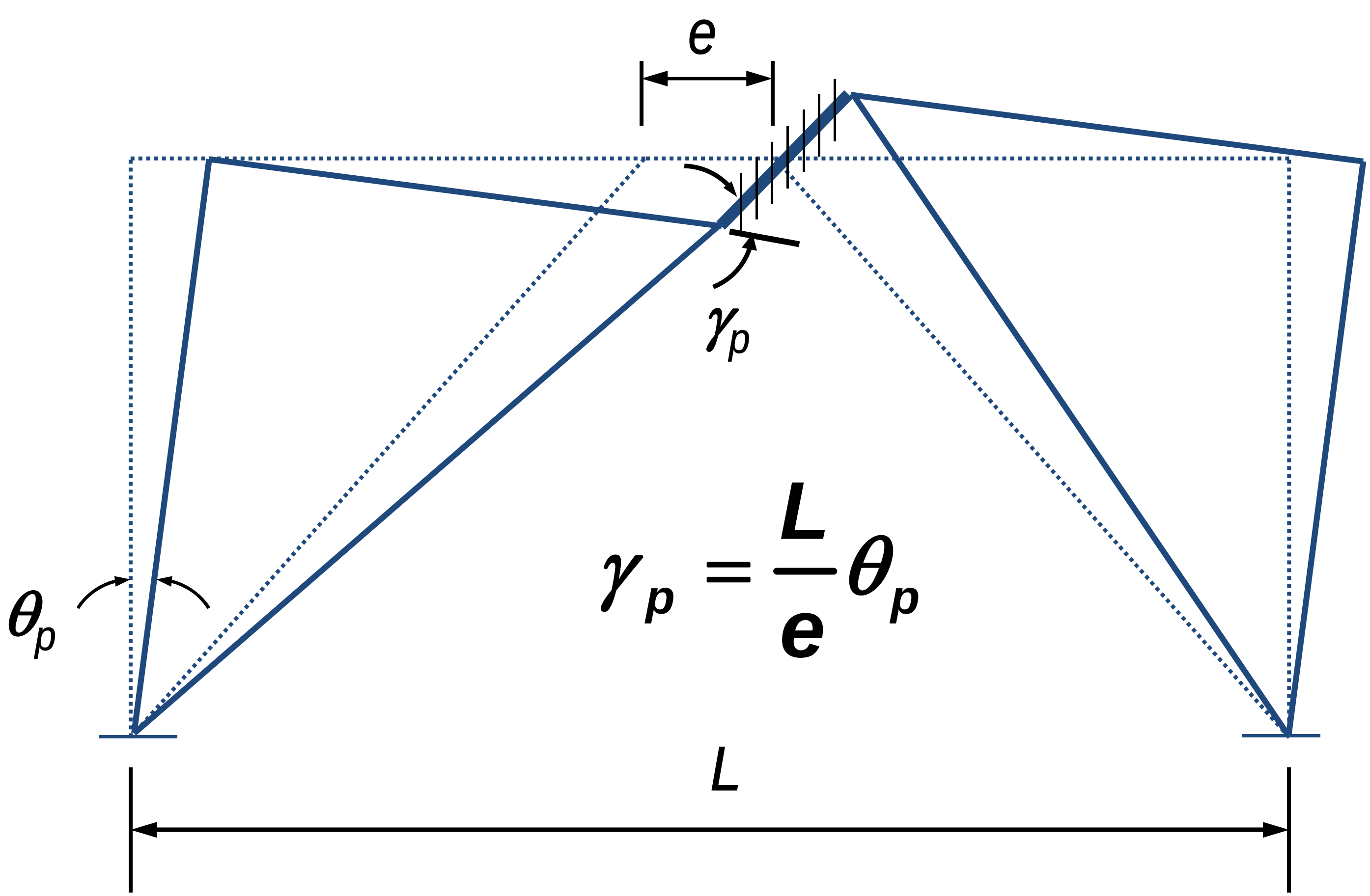
Given V_{ult} , determine M_{ult} from link equilibrium:

$$M_{ult} = \frac{e\,V_{ult}}{2}$$ (assumes link end moment equalize)

F3.3 Analysis



Link Rotation



$$\gamma_p = \frac{L}{e}\theta_p$$

F3.4a Link Rotation Angle

The **link rotation angle** is the inelastic angle between the link and the beam outside of the link when the story drift is equal to the design story drift, *Δ*.

The link rotation angle shall not exceed the following values:

a) 0.08 radians for: $e \leq 1.6\, M_p / V_p$

b) 0.02 radians for: $e \geq 2.6\, M_p / V_p$

c) a value determined by linear interpolation between the above values for: $1.6\, M_p / V_p < e < 2.6\, M_p / V_p$

F3.4a Link Rotation Angle

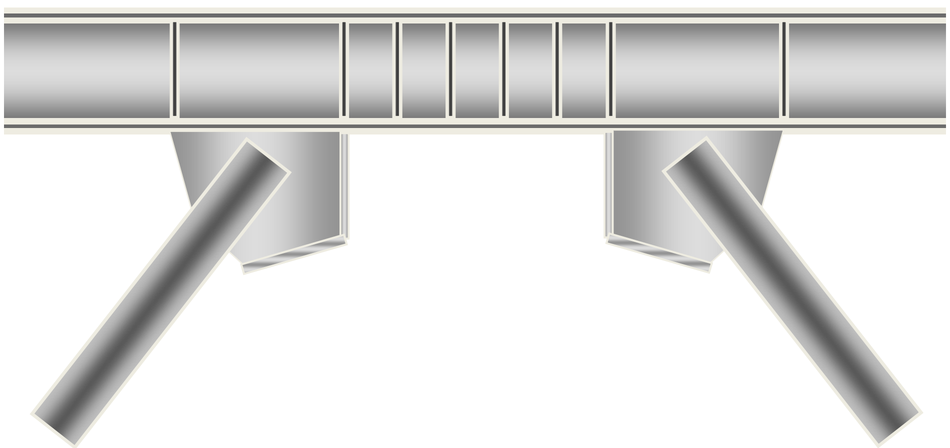
Design Approach to Check Link Rotation Angle, θ_p

1. Compute elastic story drift under code specified earthquake forces: Δ_E
2. Compute *Design Story Drift*: $\Delta = C_d \times \Delta_E$
   (C_d = 4 for EBF)
3. Estimate Plastic Story Drift: $\Delta_p \approx \Delta - \Delta_E$ (or $\approx \Delta$)
4. Compute plastic story drift angle θ_p

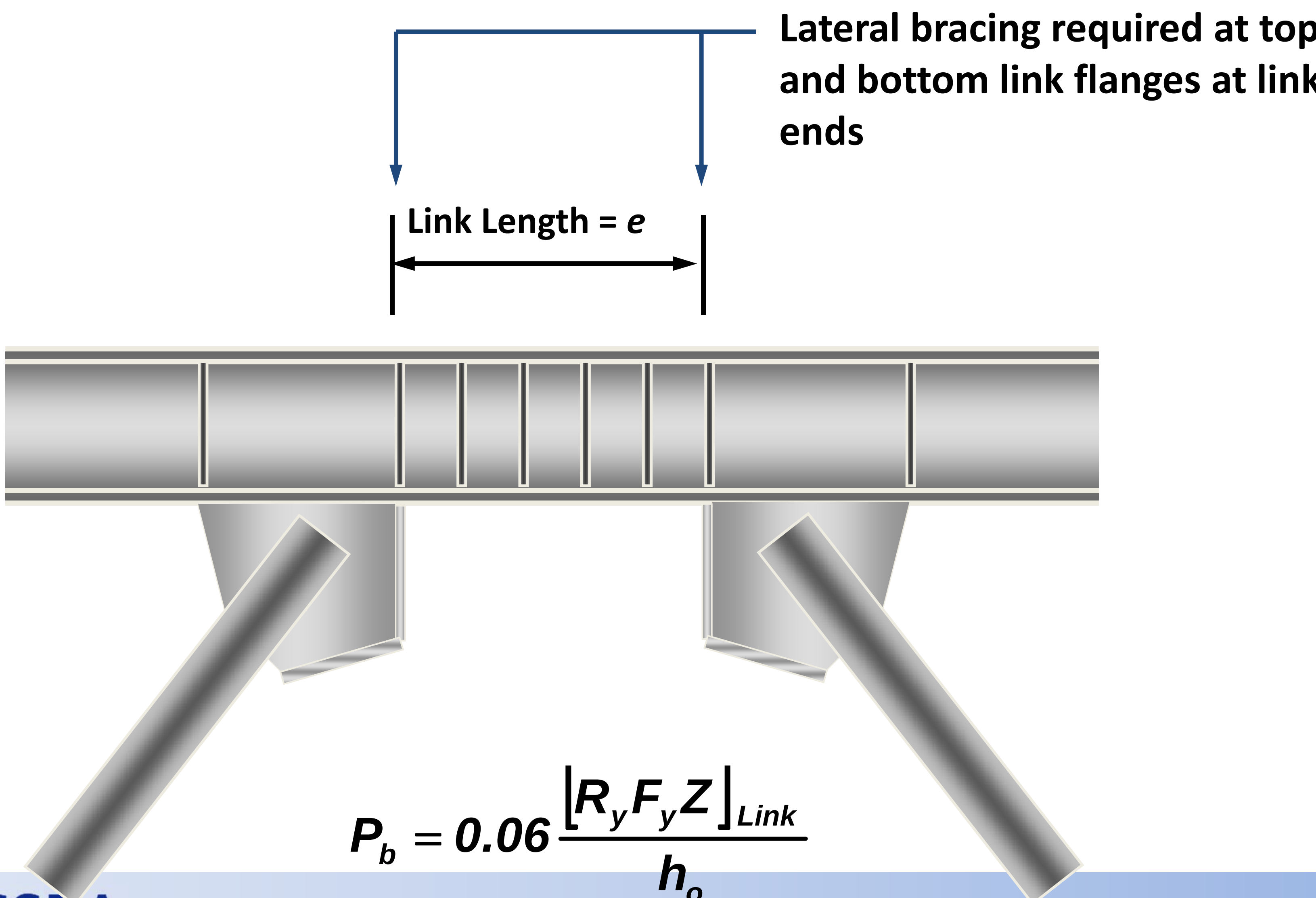
   $\theta_p \approx \Delta_p / h$ where h = story height
5. Compute link rotation angle γ_p based on EBF kinematics
   $\gamma_p = (L / e)_p$ for common EBFs
6. Check link rotation limit per Section F4.4a

F3.5b(4) Link Stiffeners

- Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link
- Intermediate stiffeners based on:
  - Link length
  - Link rotation
  - Beam depth
  - Beam web thickness



F3.4b Lateral Bracing of Link



$$P_b = 0.06\frac{\left[R_y F_y Z\right]_{Link}}{h_o}$$

F3 Other Provisions

- F3.5b Links
  - Reduction for high axial stress
    - Shear and flexural strength
    - Maximum link length
  - Detailed requirements for stiffener attachment
- F3.5c Protected Zone = Link
- Column Splices (similar to SCBF)
- Link-to-column connections require testing
- Demand Critical Welds

Structural Steel Buildings—Provisions

G. COMPOSITE MOMENT FRAME SYSTEMS

G1. Composite Ordinary Moment Frames

G2. Composite Intermediate Moment Frames

G3. Composite Special Moment Frames

G4. Composite Partially Restrained (PR) Moment Frames

H. COMPOSITE BRACED FRAME AND SHEAR WALL SYSTEMS

H1. Composite Ordinary Concentrically Braced Frames

H2. Composite Special Concentrically Braced Frames

H3. Composite Eccentrically Braced Frames

H4. Ordinary Reinforced Concrete Shear Walls Composite with Structural Steel Elements

H5. Special Reinforced Concrete Shear Walls Composite with Structural Steel Elements

H6. Composite Steel Plate Shear Walls

Structural Steel Buildings—Provisions

I. FABRICATION, ERECTION, QUALITY CONTROL AND QUALITY ASSURANCE

I1. Shop and Erection Drawings

I2. Fabrication and Erection

I3. Quality Control and Quality Assurance

J. QUALIFICATIONS AND PREQUALIFICATION TESTING PROVISIONS

J1. Prequalification of Beam-Column and Link-to-Column Connections

J2. Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections

J3. Qualifying Cyclic Tests of Buckling-Restrained Braces

Recommended References & Additional Study Materials

- *Structural Engineering PE License Review Problems & Solutions* 8th Ed., Williams.
- *Steel Structures: Design and Behavior*, Salmon and Johnson.
- *Manual of Steel Construction* (Design Examples), AISC (http://www.aisc.org/content.aspx?id=24314).
- AISC *Seismic Design Manual*.
- 2009 IBC *Structural/Seismic Design Manual*

Questions?