The Louis F. Geschwindner Seminar Series

presents

AISC *Seismic Design Manual,* 3rd Ed. and Applications of the 2016 *Seismic Provisions*

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Third Edition of AISC Seismic Design Manual

Applications of the 2016 *Seismic Provisions* – AISC 341

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Smarter.
Stronger.
Steel.

Seminar Goals

1. Introduce the *Seismic Design Manual*, Third Edition (SDM)
2. Review selected portions of:
    a. 2016 *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16)
    b. 2016 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358-16)
    c. 2016 *Specification for Structural Steel Buildings* (ANSI/AISC 360-16)
3. Present examples from the SDM to illustrate application of the *Seismic Provisions*
4. Highlight commonly misapplied or new portions of the *Seismic Provisions*

Seminar Goals

Please send in questions when they occur to you

(rather than saving them up to the end)

SDM Organization and Content

Since the *Seismic Design Manual* (SDM) contains all the information we will discuss today, we start by reviewing what is in this new edition

SDM Part 1: General Design Considerations

- ✓ Specifications, Codes and References
- ✓ Seismic Design Overview – general discussion of seismic design philosophy, ASCE 7-related requirements and miscellaneous seismic design issues
- X Identification of SFRS and sample details
- ✓ Design Tables **(illustrated via examples in this seminar)**

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 2: Analysis

- ✓ Role of Structural Analysis in Design – understand behavior especially capacity design
- X Analysis Procedures – elastic, inelastic and plastic methods and stability methods
- X Structural Modeling – strength, stiffness and gravity load considerations

SDM Part 2 not covered explicitly, but its application illustrated through examples (e.g.,5.2.8 and 5.3.2)

✓ Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 3: Systems Not Specifically Detailed for Seismic Resistance

- ✓ General Discussion (a.k.a. “R = 3 Systems”)
- ✓ Moment Frames (e.g., Example 3.4.4)
- X Braced Frames

✓ Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 4: Moment Frames

- ✓ Ordinary Moment Frames (OMF) (overview, compared to SMF)
- ✓ Intermediate Moment Frames (IMF) (overview, compared to SMF)
- ✓ Special Moment Frames (SMF) (e.g., Examples 4.3.2, 4.3.4 and 4.3.6)
- X Special Truss Moment Frames (STMF)
- ✓ Column Splice and Column Base (e.g., Examples 4.5.1, 4.5.2 and 4.5.3)
- ✓ Design Tables (illustrated via various examples in this seminar)

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 5: Braced Frames

- ✓ Ordinary Concentrically Braced Frames (OCBF) (e.g., Examples 5.2.7 and 5.2.8)
- ✓ Special Concentrically Braced Frames (SCBF) (e.g., Examples 5.3.1, 5.3.2, 5.3.4, and 5.3.7)
- X Eccentrically Braced Frames (EBF)
- ✓ Buckling Restrained Braced Frames (BRBF) (e.g., Example 5.5.1)
- ✓ Connection Design (illustrated via examples in this seminar emphasizing calculation of required strength)

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 6: Composite Moment Frames

- X • Composite Ordinary Moment Frames (C-OMF)
- X • Composite Intermediate Moment Frames (C-IMF)
- X • Composite Special Moment Frames (C-SMF)
- X • Composite Partially Restrained Moment Frames (C-PRMF)
- X • Connection Design

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 7: Composite Braced Frames and Shear Walls

- X • Composite Ordinary Braced Frames (C-OBF)
- X • Composite Special Concentrically Braced Frames (C-SCBF)
- X • Composite Eccentrically Braced Frames (C-EBF)
- X • Composite Shear Walls (C-OSW and C-SSW)

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 8: Diaphragms, Collectors and Chords

- X General
- X Flexural and Torsional Buckling of Collector Elements
- X Design Examples

✓Covered in this seminar X Not covered in this seminar

SDM Organization and Content

SDM Part 9: Provisions and Standards

- ✓ *Seismic Provisions for Structural Steel Buildings* (AISC 341-16)
- ✓ *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (AISC 358-16)

✓Covered in this seminar X Not covered in this seminar

Seminar Organization

- Review of SDM Content
- Discussion of LRFD vs. ASD
- SDM Part 1: General Design Considerations
- SDM Part 3: Systems Not Specifically Detailed for Seismic Resistance

SDM Part 2 not
covered explicitly, but
application illustrated
through examples
(e.g.,5.2.8 and 5.3.2)

Seminar Organization

- SDM Part 9: *Seismic Provisions* Chapters A, B and D (illustrated via examples from SDM Part 4)
- SDM Part 9: *Seismic Provisions* Chapter E – Moment Frames (illustrated via examples from SDM Part 4)
- *Seismic Provisions* Chapter F – Braced Frames (illustrated via examples from SDM Part 5)

Seminar Organization

Seminar Notes

- These notes contain summaries of selected parts of the *Seismic Provisions* and *Connection Prequalification Standard*
- As summaries, the notes aim at the center of the *Seismic Provisions* and do not always address the boundaries and limits
- Consult the actual standards (e.g., ASCE 7, AISC 341, AISC 358 and AISC 360) for detailed design requirements

LRFD vs. ASD

- LRFD and ASD have their own load combinations to obtain "required strength" (e.g., P_u *and* P_a). See ASCE 7 Chapter 2.
- Required strength usually obtained from building code (e.g., ASCE 7) but *Seismic Provisions* prescribe required strength in some instances
  - e.g., in Section D1.2a, when calculating the required strength of bracing, M_r is specified as $R_y F_y Z/\alpha_s$ for use in *Specification* Appendix 6 equations).

α_s will be introduced in a couple of slides

LRFD vs. ASD

- Since 2005 *Specification*, nominal strength (e.g., M_n) has been the same for LRFD and ASD (i.e., only one formula is given)
- Use of the AISC *Steel Manual* Ninth Edition (green cover – 1989 *Specification*!) for ASD is NOT recognized by current building codes
- Nominal strength is almost always obtained from *Specification*, but *Seismic Provisions* prescribe nominal strengths in some instances – these govern

LRFD vs. ASD

- LRFD “available strength” uses *resistance factor*, ϕ (e.g., ϕM_n)
- ASD “available strength” uses *safety factor*, Ω (e.g., M_n/Ω)
- Examples of basic design requirement:
  - LRFD: $M_u \leq \phi M_n$
  - ASD $M_a \leq \dfrac{M_n}{\Omega}$

LRFD vs. ASD

- α_s: 2016 *Seismic Provisions* introduce α_s, the LRFD-ASD force level adjustment factor
- For LRFD: $\alpha_s = 1.0$
- For ASD: $\alpha_s = 1.5$
- Intended to simplify the *Seismic Provisions*' specification of required strength by using a single equation for both design methods
    - Example: Section D2.5b: Required strength for each flange splice in both LRFD and ASD is: $\dfrac{0.5R_yF_yb_ft_f}{\alpha_s}$

The values of α_s are only given the first time they are used in a *Seismic Provisions* chapter

LRFD vs. ASD

- All SDM examples are worked in LRFD and ASD side-by-side (with limited exceptions, such as anchor rods per ACI 318 that have only strength-based provisions)

LRFD	ASD
$\phi M_n = \phi_b M_p$ $= 0.90(833\text{ kip-ft})$ $= 750\text{ kip-ft} > 273\text{ kip-ft}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{M_p}{\Omega_b}$ $= \dfrac{833\text{ kip-ft}}{1.67}$ $= 499\text{ kip-ft} > 136\text{ kip-ft}$ **o.k.**

- In this seminar, some examples will be worked in LRFD and others in ASD to illustrate each design methodology

SDM Part 1: General Design Considerations

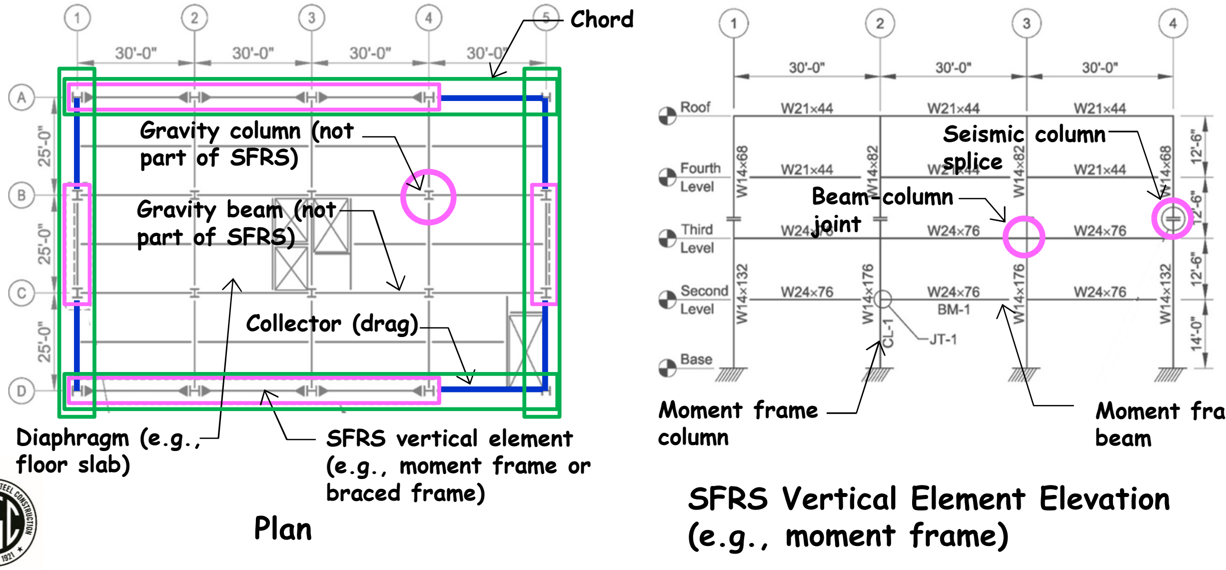
SDM Part 1: General Design Considerations

Seismic Performance Goals of the Building Code

- Primary Objective: Prevent collapse of the building during a very large earthquake.
- Objectives are not necessarily to :
  - limit damage
  - maintain function
  - provide for easy post-earthquake repair

SDM Part 1: General Design Considerations

Example Seismic Force Resisting System (SFRS) Terminology



Chord

Gravity column (not part of SFRS)

Gravity beam (not part of SFRS)

Collector (drag)

Diaphragm (e.g., floor slab)

SFRS vertical element (e.g., moment frame or braced frame)

Seismic column splice

Beam-column joint

Moment frame column

Moment frame beam

Plan

SFRS Vertical Element Elevation (e.g., moment frame)

SDM Part 1: General Design Considerations

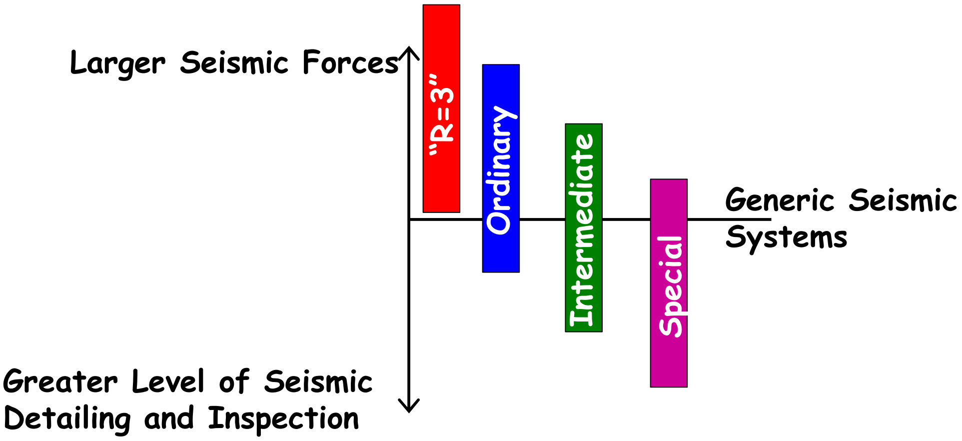
System Classifications in ASCE 7 and AISC *Seismic Provisions*

- Seismic systems classified into three* levels of inelastic response
  - Ordinary – limited ductility anticipated; provide seismic resistance using strength (i.e., design using greater code-level force)
  - Intermediate – moderate ductility anticipated; incorporate some seismic detailing using intermediate level of code force
  - Special – high ductility anticipated; incorporate most stringent seismic detailing but uses lowest level of seismic force

*"R = 3" systems discussed separately

SDM Part 1: General Design Considerations

System Classifications in ASCE 7 and AISC *Seismic Provisions*

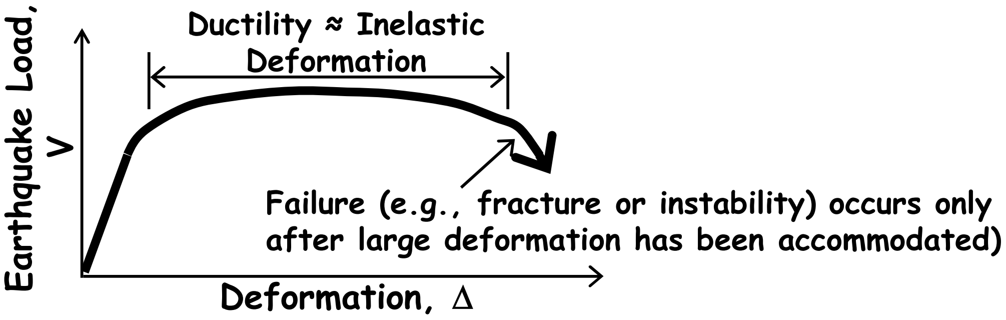


Conceptual Trade-Off Between
Larger Seismic Forces and Greater Levels of Detailing and Inspection

SDM Part 1: General Design Considerations

System Classifications in ASCE 7 and AISC *Seismic Provisions*

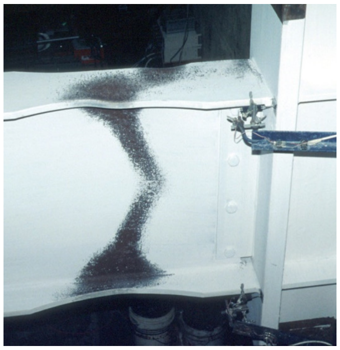
- *Seismic Provisions* attempt to accommodate inelastic behavior by:
  - Identifying specific elements where ductility is required (yielding element)
  - Limiting potential for untimely fracture or instability by requiring adequate strength in other elements



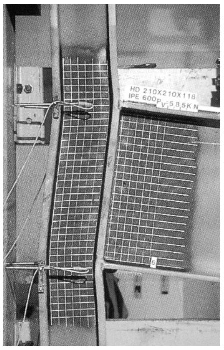
SDM Part 1: General Design Considerations

System Classifications in ASCE 7 and AISC *Seismic Provisions*

- Examples of yielding elements:



Yielding in moment frame beam (plastic hinge)



Yielding in column panel zone

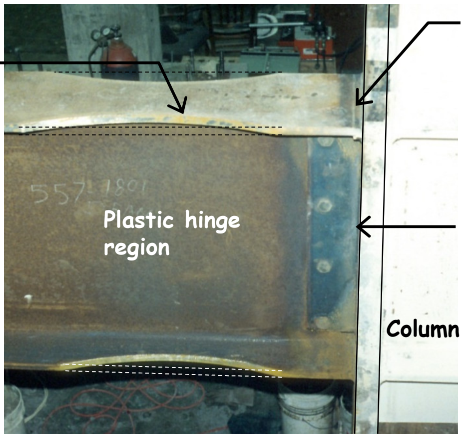
Out-of-plane bending of braced frame gusset plate

SDM Part 1: General Design Considerations

System Classifications in ASCE 7 and AISC *Seismic Provisions*

- Examples of yielding elements: Reduced Beam Section (RBS) as a plastic hinge

Beam flanges trimmed to reduce flexural strength and shift plastic hinge away from face of column



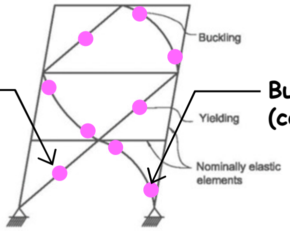
Complete joint penetration (CJP) weld between beam and column flanges

Complete joint penetration (CJP) weld between beam web and column flange

Reduced Beam Section

SDM Part 1: General Design Considerations

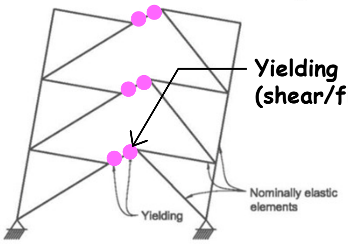
Examples of ductile system response mechanisms (deformation control)



Yielding element (tension)

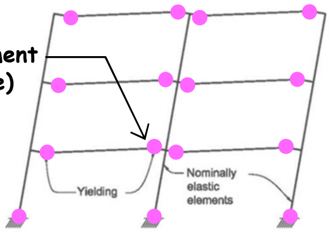
Buckling element (compression)

Braced Frame (e.g. SCBF)



Yielding element (shear/flexure)

Eccentrically Braced Frame (EBF)

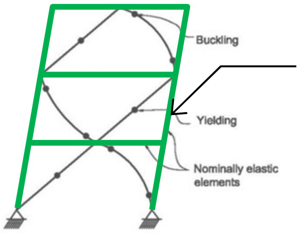


Yielding element (plastic hinge)

Moment Frame (e.g., SMF)

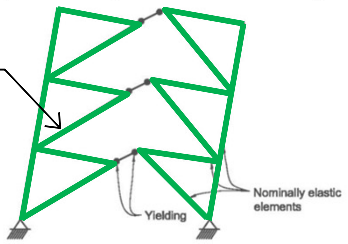
SDM Part 1: General Design Considerations

Examples of nominally elastic elements in seismic systems (force control)

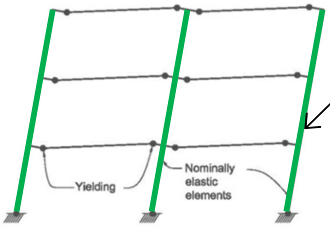


Nominally elastic element

Braced Frame (e.g. SCBF)



Eccentrically Braced Frame (EBF)



Nominally elastic element

Moment Frame (e.g., SMF)

SDM Part 1: General Design Considerations

Why don't we just design for a large force and skip the *Seismic Provisions**?

- It's a code requirement (as well as prudent practice) – building codes generally require seismic detailing in areas of moderate and high seismicity.
  - Impossible to predict with certainty how large of a force is required – sensitive to the properties of a specific building, its site, etc.
  - Potentially catastrophic consequences of premature failure (e.g., in a column) if we've guessed incorrectly regarding the load to use

*"R = 3" systems discussed separately

SDM Part 1: General Design Considerations

What do the *Seismic Provisions* require to promote good seismic behavior?

- Account for unexpected overstrength in materials by using "expected strength" where appropriate
- Specify materials with acceptable properties (e.g., ductility and toughness)
- Require member proportions and bracing that delay local buckling and lateral-torsional buckling

SDM Part 1: General Design Considerations

What else do the *Seismic Provisions* require to promote good seismic behavior?

- Require connection design and detailing that allow the intended element to deform inelastically
- Proportion other members in SFRS (i.e., those not intended to yield) to be stronger than loads causing yielding (e.g., strong column-weak beam)

SDM Part 1: General Design Considerations

Relevant ASCE 7-16 requirements found in the examples

- Although not strictly within the purview of AISC 341 and AISC 358, some ASCE 7-16 requirements are relevant to the SDM examples
- Selected ASCE 7 requirements will be discussed in the following slides

SDM Part 1: General Design Considerations

Combination of Loads for Seismic Load Effects – ASCE 7 Sections 2.3.6 and 2.4.5

- *E* in load combinations is earthquake load
- *E* is a function of horizontal and vertical seismic effects (i.e., E_h and E_v)
- Basic combinations with seismic load effects
  - $1.2D + E_v + E_h + L^* + 0.2S$ (LRFD Combination 6)
  - $0.9D - E_v + E_h$ (LRFD Combination 7)

    \- or -
  - $1.0D + 0.7E_v + 0.7E_h$ (ASD Combination 8)
  - $1.0D + 0.525E_v + 0.525E_h + 0.75L + 0.75S$ (ASD Combination 9)
  - $0.6D - 0.7E_v + 0.7E_h$ (ASD Load Combination 10)

New expressions for prior ASCE 7 load combinations

*By exception, *L* typically is 0.5*L*

SDM Part 1: General Design Considerations

Seismic Design Category (SDC) – ASCE 7 Section 11.6

- Used by building code (ASCE 7) to determine level of seismic design and detailing required: SDC A though F
- SDC determined by the “societal importance” of a structure (risk) and site seismicity (magnitude and frequency of earthquakes)
- SDC A:
  - Areas of low seismicity and low seismic risk,
  - Seismic design not considered essential to prevent unacceptable structural behavior (i.e., basic requirements for wind, integrity and stability design are believed sufficient)

SDM Part 1: General Design Considerations

Seismic Design Category (SDC)

- SDC B and C:
    - Areas of moderate seismicity and moderate seismic risk, so some seismic considerations needed to prevent unacceptable structural behavior
    - Even if wind governs, seismic detailing per AISC 341 is required unless design uses *R* for "systems not specifically detailed for seismic resistance" (i.e., *R* = 3)* – see ASCE 7 Table 12.2-1 and Section 14.1.2.2.1
    - Designer must also confirm that load combinations with overstrength (i.e., with Ω_O) do not govern, unless *R* = 3 approach is used

*"R = 3" systems discussed separately

SDM Part 1: General Design Considerations

Seismic Design Category (SDC)

- SDC D:
    - Applies where intense shaking anticipated or structure is societally important where moderate to high shaking is anticipated
    - Stringent seismic design and detailing required (e.g., AISC 341) to limit unacceptable seismic behavior
    - Even if wind forces govern member selection, seismic detailing requirements still apply

SDM Part 1: General Design Considerations

Seismic Design Category (SDC)

- SDC E and F:
  - Apply in areas of high seismicity ($S_1 \geq 0.75$ g)
  - SDC F applies to most important structures: Risk Category IV

SDM Part 1: General Design Considerations

Structures assigned to SDC B through F are subject to many seismic considerations

- *Seismic considerations may apply even if seismic detailing is not required*
- Example ASCE 7 seismic requirements applicable to SDC B through F
  - Horizontal Irregularities (Table 12.3-1)
  - Vertical Irregularities (Table 12.3-3)
  - Seismic Load Effects and Combinations (Section 12.4)
  - Direction of Loading (Section 12.5)
  - Amplification of Accidental Torsional Moment (Section 12.8.4.3)
  - Collector Elements (Section 12.10.2)
  - Foundation Design (Section 12.13)

SDM Part 1: General Design Considerations

Seismic Design Category (SDC)

- Except for SDM Chapter 3 examples using “R = 3” approach, SDM examples assume that requirements of AISC 341 and AISC 358 apply even if the SDC is not stated explicitly

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SDM Part 1: General Design Considerations

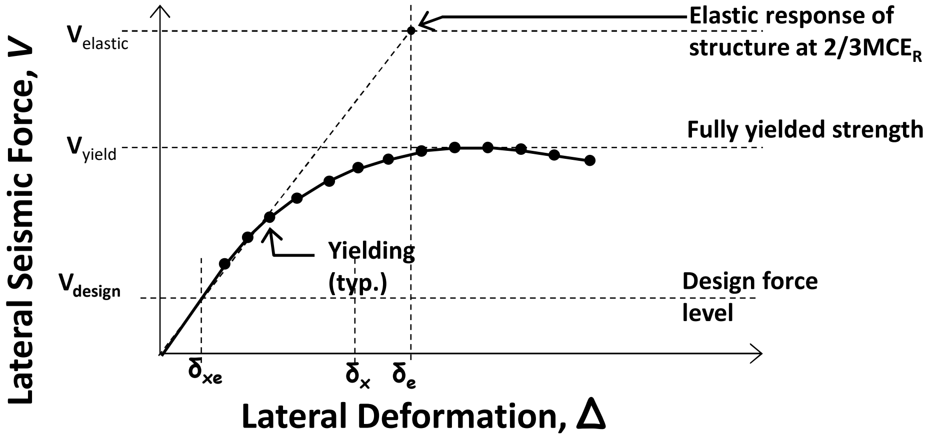
Seismic Importance Factor, I_e

- I_e considers relative importance of a structure based on ASCE 7 Risk Category (see Table 1.5-1) and amplifies seismic loads accordingly
- See ASCE Tables 1.5-2 for I_e values: range from 1.0 to 1.5
- Most structures: $I_e = 1.0$
- Essential facilities: 1.5
- SDM examples use $I_e = 1.0$

SDM Part 1: General Design Considerations

Response Modification Coefficient, *R* – ASCE 7 Table 12.2-1

- Accounts for assumed level of ductility provided by different SFRS.
- Effectively reduces design force (but design drift must be consider resulting inelastic deformation, i.e., increased)



$$R = \frac{V_{elastic}}{V_{design}}$$

$$\Omega_o = \frac{V_{yield}}{V_{design}}$$

$$C_d = \frac{\delta_x}{\delta_{xe}}$$

SDM Part 1: General Design Considerations

Response Modification Coefficient, *R*

- Varies by SFRS from 1 (least assumed available ductility) to 8 (most assumed available ductility)
- Smaller values of R result in larger required strength for design, larger values of R result in smaller required strength

Note: this is just one of 4 equations to calculate *V*

$$V = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} W$$

R is in the denominator, so *V* decreases with larger *R*-factors

Example Building Code Seismic Load Equation for Equivalent Lateral Force Method

SDM Part 1: General Design Considerations

Redundancy Factor, ρ - ASCE 7 Sections 12.3.4 and 12.4.1

- Redundant structures have multiple load paths
- ASCE 7 encourages redundancy by penalizing structures with an insufficient number of primary SFRS elements (e.g., not enough moment frames)
- ρ amplifies seismic load effect via its appearance in load combinations
- In SDC C, ρ = 1.0
- In SDC D, E and F, ρ is either 1.0 or 1.3 and is determined based on system type and irregularities (e.g., see ASCE 7 – Table 12.3-1 and 12.3-2 for a list of vertical and horizontal irregularities)
- In the SDM, ρ is given in the problem statement

SDM Part 1: General Design Considerations

Seismic Load Effects including Overstrength

- Two approaches in AISC 341: Ω_o and E_{cl} (AISC 341 spells out which to use)
- Ω_o attempts to account for *system* overstrength
  - Sometimes, the actual strength of the system is significantly greater than that implied by basic code loads (e.g., when *drift*, not *strength*, determines moment frame girder and column sizes)
  - Ω_o is an *estimate* of the ratio between minimum required strength and actual strength
  - Ω_o allows linear analysis to approximate results from nonlinear analysis
  - When specified, Ω_o provides for proper proportioning one member relative to another member, regardless of other strength requirements

SDM Part 1: General Design Considerations

Seismic Load Effects including Overstrength

- Ω_o attempts to account for *system* overstrength (continued)
  - Used in situations where an overly strong member might unintentionally overload another member otherwise intended to remain nominally elastic (i.e., prevents circumventions of intended yield mechanism)
  - *Example:* If SMF beam is stronger than SMF column, hinging will occur in the column rather than as intended in the beam, which can lead to premature collapse.

SDM Part 1: General Design Considerations

Seismic Load Effects including Overstrength

- *Seismic Provisions* may allow for a limit on the overstrength load based on “capacity limited load effect”, E_{cl}
- E_{cl} recognizes what may limit load on a member or connection
  - *Example*: required shear strength of beam-to-column connection in OMF limited to $E_{cl} = 2(1.1R_yM_p)/L_{cf}$ because that is the shear associated with strain-hardened yielding of the frame beam.
- Capacity limited load effect replaces instances in prior *Seismic Provisions* utilizing the “maximum force delivered by the system” exception

SDM Part 1: General Design Considerations

Overstrength factor, Ω_o – ASCE 7 Section 12.4.3 and Table 12.2-1

- Ω_o used in specific load combinations when *Seismic Provisions* invoke "overstrength seismic load" (formerly called "amplified seismic load")
- ASCE 7 may also invoke overstrength seismic load (e.g., elements supporting discontinuous frames per ASCE 7 Section 12.3.3.3)
- Per ASCE 7, Ω_o varies from 2 to 3 for steel systems

SDM Part 1: General Design Considerations

Combination of Loads for Seismic Load Effects – ASCE 7 Sections 2.3.6 and 2.4.5

- See ASCE 7 Section 12.4.2 and 12.4.3 for complete definitions
- Combinations with seismic load effects **including overstrength**, where $E_{mh} = \Omega_o E$ or E_{cl}
  - $1.2D + E_v + E_{mh} + L + 0.2S$ (LRFD Combination 6)
  - $0.9D - E_v + E_{mh}$ (LRFD Combination 7)

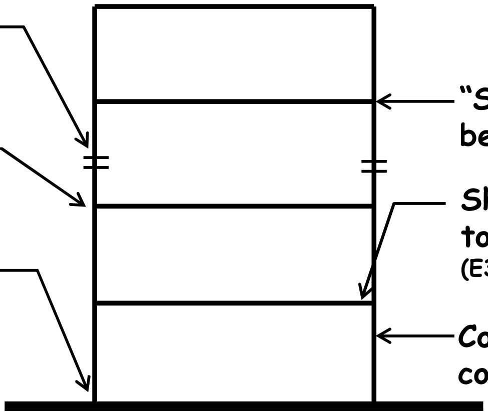
    \- or -
  - $1.0D + 0.7E_v + 0.7E_{mh}$ (ASD Combination 8)
  - $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S$ (ASD Combination 9)
  - $0.6D - 0.7E_v + 0.7E_{mh}$ (ASD Load Combination 10)

New expressions for prior ASCE 7 load combinations

SDM Part 1: General Design Considerations

Overstrength factor, Ω_o

- Example for SMF where Ω_o or E_{cl} is invoked by *Seismic Provisions* (see *SDM* pg. 1-15 for a comprehensive list):



Column splice strength (D2.5b)

Column strength for unbraced beam-to-column connections (E3.4c)

Axial, shear and flexural strength of column base (D2.6a,b,c)

"Strong-column-weak-beam" check (E3.4a)

Shear strength of beam-to-column connections (E3.6d)

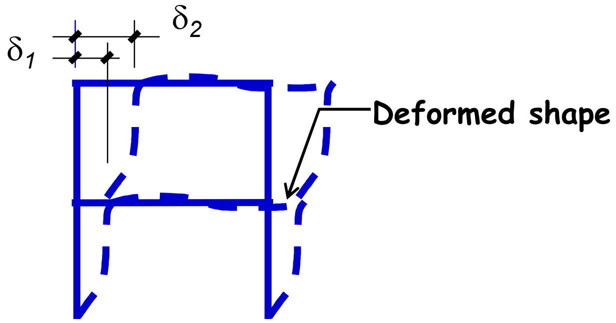
Column tensile and compression strength (D1.4a)

SDM Part 1: General Design Considerations

Story Drift

- Design story drift, Δ, shall be computed as the difference of the deflection (displacement) between two adjacent levels, typically at the center of mass on one floor and a point directly below on the floor below.

$$\text{Drift}_{2\text{-}1} = \Delta_{2\text{-}1} = \delta_2 - \delta_1$$



SDM Part 1: General Design Considerations

Story Drift – ASCE Section 12.8.6

- Since we used a “reduced” seismic force to design the building assuming ductile deformation, we won’t get consistent deflections across different SFRS if we use that force for estimating the total (inelastic) drift of the structure
  - Drifts need to be calibrated for the different systems for comparison with drift limits in ASCE 7
  - Therefore, the “elastic” drift, δ_{xe}, (e.g., that calculated using the ELF force) is amplified using a system-specific factor to reflect the estimated inelastic drift

---

SDM Part 1: General Design Considerations

Story Drift – ASCE Section 12.8.6

- The “deflection amplification factor,” C_d, takes the elastic drift, δ_{xe}, and amplifies it to estimate the inelastic drift, δ_x (i.e., allows use of linear analysis in lieu of nonlinear analysis)

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \qquad \text{Eq. 12.8-15}$$

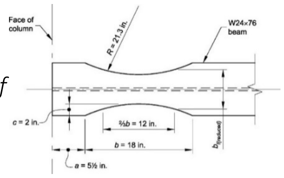
- This amplification factor, C_d, varies by SFRS (e.g., 2.5 to 6 for various steel SFRS, see ASCE 7 Table 12.2-1)

SDM Part 1: General Design Considerations

Story Drift – AISC 358 Drift Amplification

Not the same as C_d amplification

- A reduced beam section (RBS) connection reduces stiffness of frame beam
- AISC 358 requires that impact of RBS on drift be considered when gross section properties are used to model frame beam.
  - Reduce beam stiffness by 10% for flange cuts up to $0.5b_f$ (per NIST publication on SMF)
  - Linear interpolation permitted for lesser cuts
- May model stiffness of RBS directly with no prescribed amplification – some analysis programs can modify stiffness matrix to reflect RBS dimensions



SDM Part 1: General Design Considerations

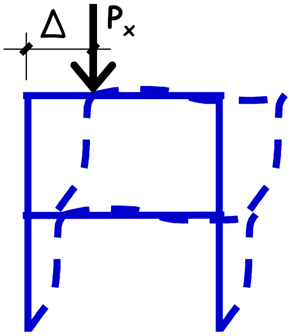
P-Δ – ASCE Section 12.8.7

- P-Δ is moment caused by axial load at a given level, P_x, times the design story drift, Δ, which can further amplify drift in flexible or heavily loaded structures
- ASCE 7 requires consideration of P-Δ effects if they are judged to be significant (i.e., if θ > 0.1)

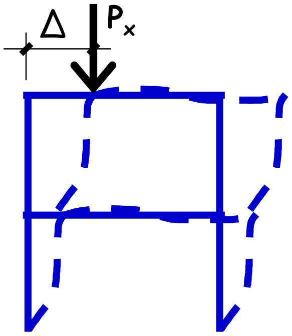
P-Δ moment

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

Eq. 12.8-16



SDM Part 1: General Design Considerations



$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$ (ASCE 7, Eq. 12.8-16)

where:

P_x = Total vertical design load at and above Level x

Δ = Design story drift

I_e = Importance Factor

V_x = Seismic force acting between Levels *x* and *x*-1

h_{sx} = Story height below Level *x*

C_d = Deflection amplification factor

SDM Part 1: General Design Considerations

P-Δ – ASCE Section 12.8.7

- $\theta \leq \theta_{max} \leq 0.25$, where

$$\theta_{\max} = \frac{0.5}{\beta C_d}$$ Eq. 12.8-17

  - β is ratio of shear demand to shear capacity for the story between levels *x* and *x*-1, and may conservatively be taken as 1.0

SDM Part 1: General Design Considerations

P-Δ – ASCE Section 12.8.7

- Where $0.1 < \theta \leq 0.25$, incremental effects of P-Δ shall be determined via rational analysis or displacement and member forces may be multiplied by $1.0/(1-\theta)$
- When P-Δ obtained via automated analysis (e.g., using a finite element program that includes P-Δ effects), θ must still be less than 0.25
  - When evaluating θ from Eq. 12.8-16, may divide θ by $(1 + \theta)$ before checking θ_{max}

SDM Part 1: General Design Considerations

Second Order Effects – *Specification* Section C1, Appendix 7 and Appendix 8

- *Specification* Section C1 requires consideration of second-order effects, including P-Δ and P-δ effects, regardless of the value of θ from ASCE 7
- P-Δ considers effects of building lateral translation (drift) (similar in concept to ASCE 7's θ)
  - B_2 is multiplier to account for P-Δ effects per Appendix Section 8.2.2
  - When $\theta < 0.1$, usually $B_2 < 1.1$ so $K = 1.0$ for equivalent length method
- P-δ considers effects of member deformation
  - B_1 is multiplier to account for P-δ effects per Appendix Section 8.2.2

SDM Part 1: General Design Considerations

Second Order Effects – *Specification* Section C1, Appendix 7 and Appendix 8

- Required second-order flexural and axial strength
  - $M_r = B_1 M_{nt} + B_2 M_{lt}$ (Eq. A-8-1)
  - $P_r = P_{nt} + B_2 P_{lt}$ (Eq. A-8-2)

where

M_{nt} = first order moment of laterally restrained structure

M_{lt} = first order moment due to lateral translation

P_{nt} = first order axial force of laterally restrained structure

P_{lt} = first order axial force due to lateral translation

SDM Part 1: General Design Considerations

Second-Order Effects – *Specification* Section C1, Appendix 7 and Appendix 8

- Appendix 7 presents alternative methods for design for stability (e.g., effective length method)
- Appendix 8 presents an approximate second-order analysis method (i.e., B_1 and B_2multiplier method)
- SDM examples utilize effective length and approximate second-order analysis methods
- Seminar Example 5.2.8 illustrates detailed application of second-order effect requirements (although second-order are considered in many other SDM examples)

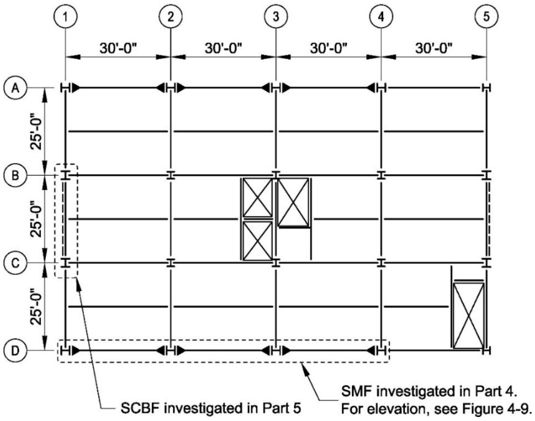
Example 4.3.1
SMF Story Drift and Stability Check

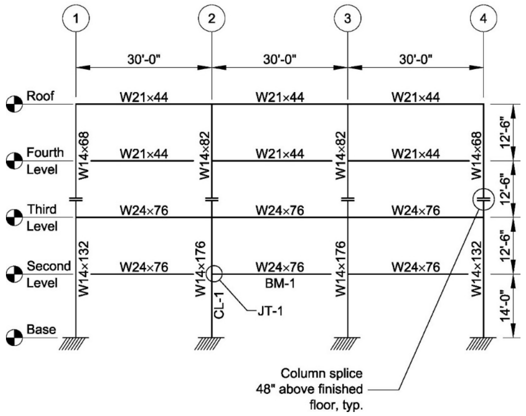
- Complete example emphasizing calculation of amplified RBS drift, comparison to allowable drift and calculation of stability coefficient, θ (see SDM page 4-40)
- RBS geometry is verified in Example 4.3.3 (not presented here) and building deformation is taken as given information

---

Example 4.3.1 SMF Story Drift and Stability Check

Given:
Refer to the floor plan shown in Figure 4-8 and the SMF elevation shown in Figure 4-9. Determine if the frame satisfies the ASCE 7 drift and stability requirements based on the given loading.





Example 4.3.1 SMF Story Drift and Stability Check

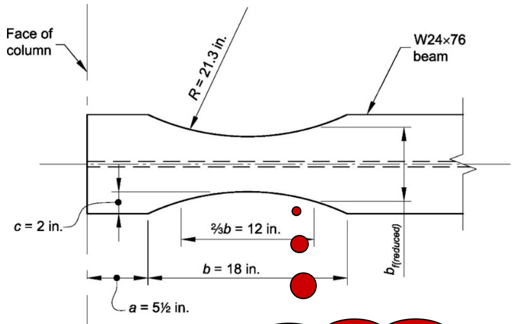
Solution:

From AISC *Manual* Table 1-1: W24×76:

b_f = 8.99 in.

Reduced beam section (RBS) connections are used at the frame beam-to-column connections and the flange cut will reduce the stiffness of the beam.

Figure 4-10 of Example 4.3.3 illustrates the design of the RBS geometry, and the flange cut on one side of the web is c = 2 in.



Total flange cut is
2c = 2(2″) = 4″

Example 4.3.1 SMF Story Drift and Stability Check

Section 5.8, Step 1, of AISC 358 states that the calculated elastic drift, based on gross beam section properties, may be multiplied by 1.1 for flange reductions up to 50% of the beam flange width in lieu of specific calculations of effective stiffness.

Amplification of drift values for cuts less than the maximum may be linearly interpolated between 1.0 and 1.1.

For b_f = 8.99 in., the maximum cut is:

0.5(8.99 in.) = 4.50 in.

Example 4.3.1 SMF Story Drift and Stability Check

Thus, the total 4” cut is:

$$\left(\frac{4.00\text{ in.}}{4.50\text{ in.}}\right)100\% = 88.9\%\text{ of the maximum cut}$$

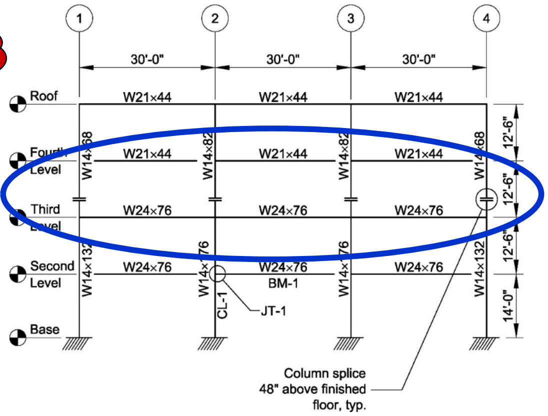
The calculated elastic drift needs to be amplified by 8.89% (say, 1.09 amplification).

Example 4.3.1 SMF Story Drift and Stability Check

Drift Check
From an elastic analysis of the structure that includes second-order effects (not shown), the maximum interstory drift occurs between the third and fourth levels. The effective elastic drift is:

Given

$$\begin{aligned}\delta_{xe} &= \delta_{4e} - \delta_{3e} \\ &= 0.482\text{ in.} \\ \delta_{xe\ RBS} &= 1.09\delta_{xe} \\ &= 1.09(0.482\text{ in.}) \\ &= 0.525\text{ in.}\end{aligned}$$



Example 4.3.1 SMF Story Drift and Stability Check

ASCE 7, Section 12.8.6, defines the design story drift, Δ, computed from δ_x, as the difference in the deflections at the center of mass at the top and bottom of the story under consideration, which in this case is the third level:

$$\Delta = \frac{C_d \delta_{xe}}{I_e} \qquad \text{(from ASCE 7, Eq. 12.8-15)}$$

$$= \frac{5\frac{1}{2}(0.525 \text{ in.})}{1.00}$$

$$= 2.89 \text{ in.}$$

From ASCE 7, Table 12.12-1, the allowable story drift at level x, Δ_a, is $0.020h_{sx}$, where h_{sx} is the story height below level x.

Example 4.3.1 SMF Story Drift and Stability Check

(Although not used in this example, Δ_a can be increased to $0.025h_{sx}$ if interior walls, partitions, ceilings, and exterior wall systems are designed to accommodate these increased story drifts.)

ASCE 7, Section 12.12.1.1, requires for seismic force-resisting systems comprised solely of moment frames in structures assigned to Seismic Design Category D, E or F, that the design story drift not exceed Δ_a/ρ for any story. Determine the allowable story drift as follows:

$$\frac{\Delta_a}{\rho} = \frac{0.020h_{sx}}{\rho}$$

$$= \frac{0.020(12.5 \text{ ft})(12 \text{ in./ft})}{1.0}$$

$$= 3.00 \text{ in.} > 2.89 \text{ in.} \quad \text{o.k.}$$

The frame satisfies the drift requirements.

If ρ = 1.3, effective allowable drift would only have been 2.31″

Example 4.3.1 SMF Story Drift and Stability Check

Frame Stability Check

ASCE 7, Section 12.8.7, provides a method for the evaluation of the *P*-Δ effects on moment frames based on a stability coefficient, θ, which should be checked for each floor.

For the purposes of illustration, this example checks the stability coefficient only for the third level. Note: $\delta_{3\text{-}2} = 0.365$ in.

The stability coefficient, θ, is determined as follows:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

(ASCE 7, Eq. 12.8-16)

Note: Δ and V_x come from same analysis

Example 4.3.1 SMF Story Drift and Stability Check

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

(ASCE 7, Eq. 12.8-16)

where:

P_x = Total vertical design load at and above Level x
Δ = Design story drift
I_e = Importance Factor = 1.0
V_x = Seismic force acting between Levels *x* and *x*-1 = 140 kips
h_{sx} = Story height below Level *x* = 12.5 ft.
C_d = Deflection amplification factor = 5 1/2

Example 4.3.1 SMF Story Drift and Stability Check

To determine P_x, ASCE 7 does not explicitly specify load factors to be used on the gravity loads for determining P_x, except that Section 12.8.7 does specify that no individual load factor need exceed 1.0.

This means that if the combinations of ASCE 7, Section 2.3, are used, a factor of 1.0 can be used for dead load rather than the usual 1.2 factor used in the LRFD load combination, for example.

This also means that the vertical component $0.2S_{DS}$ need not be considered here.

$$\begin{aligned} P_x &= 1.0D + 0.5L \\ &= 2{,}280 \text{ kips} + 540 \text{ kips} \\ &= 2{,}820 \text{ kips} \end{aligned}$$

Example 4.3.1 SMF Story Drift and Stability Check

The seismic design story drift between the second and third level, including the 9% amplification on the drift, is:

$$\Delta = \frac{C_d \delta_{xe}}{I_e} \qquad \text{(ASCE 7, Eq. 12.8-15)}$$

$$= \frac{5\frac{1}{2}(1.09)(0.365 \text{ in.})}{1.00}$$

$$= 2.19 \text{ in.}$$

Example 4.3.1 SMF Story Drift and Stability Check

Therefore, the stability coefficient is:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} \qquad \text{(ASCE 7, Eq. 12.8-16)}$$

$$= \frac{(2{,}820 \text{ kips})(2.19 \text{ in.})(1.00)}{(140 \text{ kips})(12.5 \text{ ft})(12 \text{ in./ft})\left(5\frac{1}{2}\right)}$$

$$= 0.0535$$

Because a second-order analysis was used to compute the story drift, θ is adjusted as follows to verify compliance with θ_{max}, per ASCE 7, Section 12.8.7.

$$\frac{\theta}{1+\theta} = \frac{0.0535}{1+0.0535}$$

$$= 0.0508$$

Example 4.3.1 SMF Story Drift and Stability Check

According to ASCE 7, if θ is less than or equal to 0.10, second-order effects need not be considered for computing story drift.

Note that this check illustrates that, per ASCE 7, second-order effects need not be considered for drift or member forces because θ is less than 0.10.

However, per AISC *Specification* Chapter C, second-order effects must be considered in determining design forces for member design.

Many analysis programs make it easy to include P-Δ effects

Example 4.3.1 SMF Story Drift and Stability Check

Check the maximum permitted θ

The stability coefficient may not exceed θ_{max}. In determining θ_{max}, β is the ratio of shear demand to shear capacity for the level being analyzed and may be conservatively taken as 1.0.

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad \text{(ASCE 7, Eq. 12.8-17)}$$

$$= \frac{0.5}{1.0\left(5\frac{1}{2}\right)} \leq 0.25$$

$$= 0.0909 < 0.25$$

Example 4.3.1 SMF Story Drift and Stability Check

The adjusted stability coefficient is less than the maximum:

$0.0508 < 0.0909$ o.k.

The moment frame meets the allowable story drift and stability requirements for seismic loading.

Example 4.3.1 SMF Story Drift and Stability Check

Comments:
There are a total of six bays of SMF in this example. Considering the relative expense of SMF connections and because the drift and stability limits are met, it may be more cost-effective to reduce the number of bays to four and increase member sizes to satisfy the strength and stiffness requirements.

Example 4.3.1
SMF Story Drift and Stability Check

Third Edition of AISC Seismic Design Manual

Applications of the 2016 *Seismic Provisions* – AISC 341

Smarter.
Stronger.
Steel.

SDM Part 3: Systems Not Specifically Designed for Seismic Resistance

SDM Part 2 not covered explicitly, but application illustrated through examples (e.g.,5.2.8 and 5.3.2)

In this section of the seminar, we cover:

- General Discussion of “R = 3” Systems
- Example 3.4.4: Moment Frame Connection Design

SDM Part 3: Systems Not Specifically Designed for Seismic Resistance

- Sometimes known as "R = 3" Applications
- In areas of low* and moderate seismicity, ASCE 7 permits the use of larger seismic forces in lieu of seismic detailing required by *Seismic Provisions*
- In Seismic Design Categories B and C this alternative is recognized by ASCE 7 (Section 14.1.2.2.1 and Table 12.2-1, Section H)
- When permitted, engineers may use *R* = 3 and follow design and detailing requirements of AISC *Specification* (AISC 360)

* Seismic Design Category A does not use an R-factor and *Seismic Provisions* do not apply

SDM Part 3: Systems Not Specifically Designed for Seismic Resistance

- "R = 3" approach may be cost effective if:
  - In SDC B (in almost all cases)
  - In SDC C where wind loads are significantly larger than seismic loads (i.e., member sizes are determined by wind demand not seismic demand)
  - Economic analysis shows savings from avoiding "seismic" detailing and inspection vs. added material and foundation costs due to *R* = 3

Can't completely ignore seismic; recall earlier discussion regarding load combinations with "overstrength", irregularities, etc.

Example 3.4.4

Moment Frame Beam-to-Column Connection Design

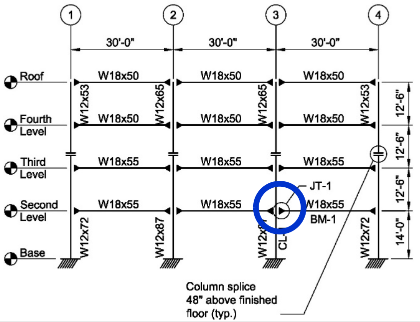
- Complete example emphasizing design of bolted flange plate connection, investigation of bolt limit states (see SDM pg. 3-13)
- Example 3.4.3 (not discussed) provides required strength of frame
- Example worked in ASD

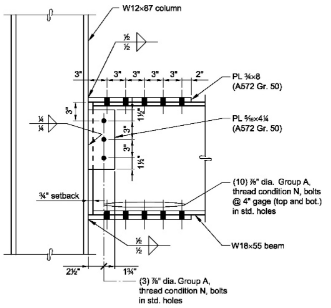
Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Given:

Refer to Joint JT-1 in Figure 3-2. Design a bolted flange-plated fully restrained (FR) moment connection between Beam BM-1 and Column CL-1.

The beam and column are ASTM A992 W-shapes, and ASTM A572 Grade 50 is used for the connecting material. Use Group A bolts with threads not excluded from the shear plane (thread condition N) and 70-ksi electrodes.





Example 3.4.4 Moment Frame Beam-to-Column Connection Design

From Example 3.4.3, the required strengths are: V_a = 23.1 kips
M_a = 212 kip-ft

ASTM A572 Grade 50	ASTM A992
F_y = 50 ksi	F_y = 50 ksi
F_u = 65 ksi	F_u = 65 ksi

From AISC *Specification* Table J3.3, for 7/8-in.-diameter bolts in standard holes:

$$d_h = \frac{15}{16} \text{ in.}$$

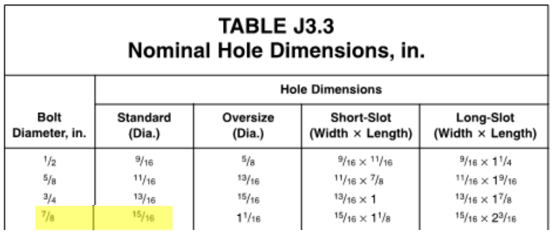


TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter, in.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16

Specifications **Table J3.3**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

From AISC *Manual* Table 1-1, the geometric properties are as follows:

W18×55
d = 18.1 in. t_w = 0.390 in. t_f = 0.630 in.
S_x = 98.3 in.^{3} b_f = 7.53 in.

Available Flexural Strength of Beam BM-1
AISC *Specification* Section F13.1 requires that tensile rupture of the tension flange be investigated if: $F_u A_{fn} < Y_t F_y A_{fg}$

Because F_y/F_u = 50 ksi / 65 ksi = 0.769 < 0.8; thus, Y_t = 1.0

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

For two rows of 7/8-in.-diameter Group A bolts in standard holes in the beam tension flange, using AISC *Specification* Section B4.3b:

$$A_{fg} = b_f t_f$$

$$= (7.53 \text{ in.})(0.630 \text{ in.})$$

$$= 4.74 \text{ in.}^2$$

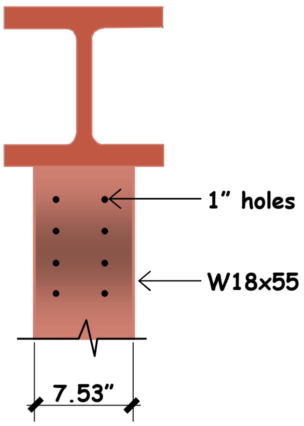
Flange gross area

$$A_{fn} = A_{fg} - 2\left(d_h + \frac{1}{16} \text{ in.}\right)t_f$$

$$= 4.74 \text{ in.}^2 - 2\left(\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.}\right)(0.630 \text{ in.})$$

$$= 3.48 \text{ in.}^2$$

Flange net area



$$F_u A_{fn} = (65 \text{ ksi})(3.48 \text{ in.}^2) = 226 \text{ kips}$$

<

$$Y_t F_y A_{fg} = 1.0(50 \text{ ksi})(4.74 \text{ in.}) = 237 \text{ kips}$$

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Since $F_u A_{fn} < Y_t F_y A_{fg}$, the limit state of tensile rupture of the flange applies.

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \qquad (\textit{Spec.} \text{ Eq. F13-1})$$

$$= \left(\frac{226 \text{ kips}}{4.74 \text{ in.}^2}\right)(98.3 \text{ in.}^3)(1 \text{ ft}/12 \text{ in.})$$

$$= 391 \text{ kip-ft}$$

The available flexural strength of the W18x55 is:

$$\frac{M_n}{\Omega_b} = \frac{391 \text{ kip-ft}}{1.67}$$

$$= 234 \text{ kip-ft} > 212 \text{ kip-ft} \quad \textbf{o.k.}$$

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Single-Plate Web Connection

As discussed in Part 12 of the AISC *Manual*, eccentricity can be neglected for the shear connection of a fully restrained moment connection; however, AISC *Manual* Table 10-10b is applied here for simplicity.

Conservatively, using AISC *Manual* Table 10-10b, select a 5/16-in.-thick ASTM A572 Grade 50 plate with three 7/8-in.-diameter Gr. A325 (Group A, thread condition N) bolts in standard holes connected to the beam web and a 1/4-in. fillet weld to the column flange. This weld is sized based on (5/8)t_p as given in AISC *Manual* Table 10-10b.

Group A bolts to be discussed

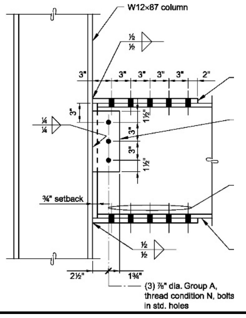
Thread condition *N* permits threads within the shear plane.

Note:(5/8)(5/16) = 0.195″ (3.12/16th, round up to $\frac{1}{4}$″ weld)

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Note, however, that the restraint provided by the flange connections will prevent fixed end moments in the single plate such that the weld need only be designed for the shear force acting at the centroid of the bolt group. The available strength of the single-plate connection is:

$$\frac{R_n}{\Omega} = 36.6 \text{ kips} > 23.1 \text{ kips} \quad \textbf{o.k.}$$



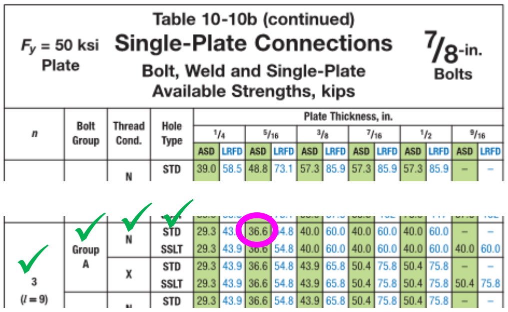


Table 10-10b (continued)

F_y = 50 ksi Plate

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
		N	STD	39.0	58.5	48.8	73.1	57.3	85.9	57.3	85.9	57.3	85.9	–	–
✓ 3 (*l* = 9)	✓ Group A	✓ N	✓ STD	29.3	43.	36.6	54.8	40.0	60.0	40.0	60.0	40.0	60.0	–	–
			SSLT	29.3	43.9	36.6	54.8	40.0	60.0	40.0	60.0	40.0	60.0	40.0	60.0
		X	STD	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	–	–
			SSLT	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	50.4	75.8
			STD	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	–	–

Steel Manual **Table 10-10b**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Because the bolt bearing and tearout limit states on the plate are included in Table 10-10b, the beam web is acceptable by inspection, as the beam web thickness of 0.390 in. is greater than the plate thickness of 5/16 in. (0.313")

Use a 5/16-in.-thick single-plate connection with three 7/8-in.-diameter Group A (thread condition N) bolts in standard holes to the beam web and a 1/4-in. fillet weld to the column flange.

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Flange-Plate Connection

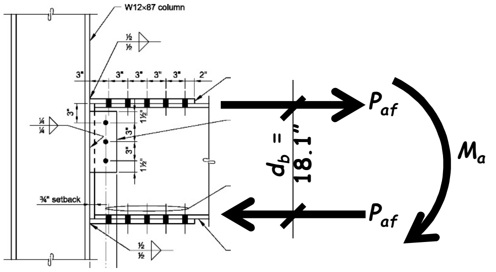
This example uses standard holes in the flange plates and beam flanges. Note that oversized holes in the flange plates may be preferable for fit-up to account for tolerances in the column flange tilt, depth, etc. Refer to AISC *Manual* Figure 12-3 for more information.

The use of oversized holes requires slip-critical bolts and reduces the net area of the flange plates.

Determine the required number of bolts in the flange plate.

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

$$\begin{aligned} P_{af} &= \frac{M_a}{d} \\ &= \frac{(212\text{ kip-ft})(12\text{ in./ft})}{18.1\text{ in.}} \\ &= 141\text{ kips} \end{aligned}$$



From AISC *Manual* Table 7-1 for available bolt shear strength, the required number of 7/8-in.-diameter Group A (thread condition N) bolts is:

$$\begin{aligned} n_{min} &= \frac{P_{af}}{r_n/\Omega} \\ &= \frac{141\text{ kips}}{16.2\text{ kips/bolt}} \\ &= 8.70\text{ bolts} \end{aligned}$$

***Steel Manual* Table 7-1**

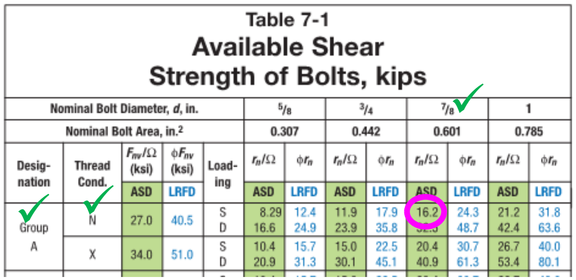


Table 7-1

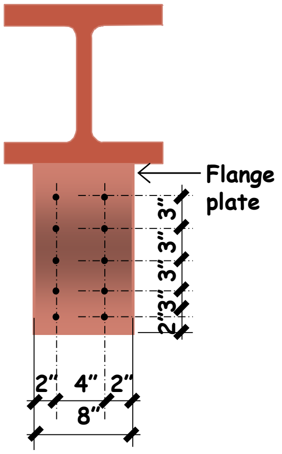
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter, *d*, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
Desig-nation	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Load-ing	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	[illegible]	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Try 10 bolts on a 4-in. gage. Using AISC *Manual* Tables 7-4 and 7-5 for bearing and tearout strength with l_e = 2 in. and s = 3 in., the available bearing and tearout strength of the beam flange is:

$$\begin{aligned} \frac{R_n}{\Omega} &= n\left(\frac{r_n}{\Omega}\right)t_f \\ &= 8(68.3\text{ kip/in.})(0.630\text{ in.}) \\ &\quad +2(59.7\text{ kip/in.})(0.630\text{ in.}) \\ &= 419\text{ kips} > 141\text{ kips} \quad \textbf{o.k.} \end{aligned}$$



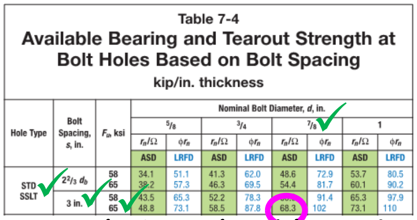


Table 7-4

Available Bearing and Tearout Strength at Bolt Holes Based on Bolt Spacing

kip/in. thickness

Hole Type	Bolt Spacing, *s*, in.	F_u, ksi	Nominal Bolt Diameter, *d*, in.							
			5/8		3/4		7/8		1	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2^2/_3\, d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	53.7	80.5
		65	38.2	57.3	46.3	69.5	54.4	81.7	60.1	90.2
	3 in.	58	43.5	65.3	52.2	78.3	[illegible]	91.4	65.3	97.9
		65	48.8	73.1	58.5	87.8	68.3	102	73.1	110

***Steel Manual* Table 7-4**

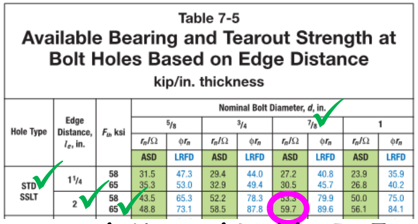


Table 7-5

Available Bearing and Tearout Strength at Bolt Holes Based on Edge Distance

kip/in. thickness

Hole Type	Edge Distance, l_e, in.	F_u, ksi	Nominal Bolt Diameter, *d*, in.							
			5/8		3/4		7/8		1	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	1¼	58	31.5	47.3	29.4	44.0	27.2	40.8	23.9	35.9
		65	35.3	53.0	32.9	49.4	30.5	45.7	26.8	40.2
	2	58	43.5	65.3	52.2	78.3	53.3	79.9	50.0	75.0
		65	48.8	73.1	58.5	87.8	59.7	89.6	56.1	84.1

***Steel Manual* Table 7-5**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Size the flange plate for the tension force

The minimum thickness of an 8-in.-wide plate for tension yielding is:

$$\begin{aligned} t_{min} &= \frac{P_{af}\Omega}{F_y b_p} \\ &= \frac{(141\text{ kips})(1.67)}{(50\text{ ksi})(8\text{ in.})} \\ &= 0.589\text{ in.} \end{aligned}$$

Although a 5/8" plate will satisfy tension yielding, need to check tensile rupture

---

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Try a 3/4-in. x 8-in. plate. The available tensile rupture strength of the plate is determined according to AISC *Specification* Section J4.1 as follows:

$$\begin{aligned} R_n &= F_u A_e \\ &= F_u A_n U \\ &= (65\text{ ksi})\left(\frac{3}{4}\text{ in.}\right)\left[8\text{ in.} - 2\left(\frac{15}{16}\text{ in.} + \frac{1}{16}\text{ in.}\right)\right](1.0) \\ &= 293\text{ kips} \end{aligned}$$

U (shear lag factor) = 1.0 for Case 1 per Spec. Table D3.1

$A_e = A_n U$

$$\begin{aligned} \frac{R_n}{\Omega} &= \frac{293\text{ kips}}{2.00} \\ &= 147\text{ kips} > 141\text{ kips} \quad \textbf{o.k.} \end{aligned}$$

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

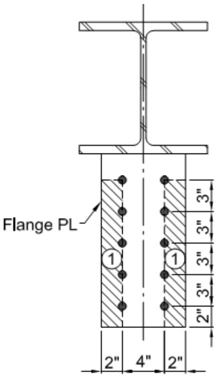
Using AISC *Manual* Tables 7-4 and 7-5 with l_e = 2 in. and s = 3 in., the bearing and tearout strength of the flange plate is:

$$\frac{R_n}{\Omega} = n\left(\frac{r_n}{\Omega}\right)t_p$$

$$= 8(68.3\text{ kip/in.})\left(\frac{3}{4}\text{ in.}\right)$$

$$+2(59.7\text{ kip/in.})\left(\frac{3}{4}\text{ in.}\right)$$

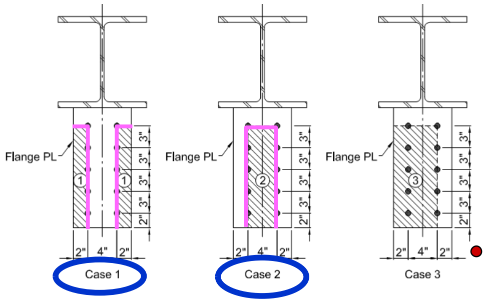
$$= 499\text{ kips} > 141\text{ kips}\quad \textbf{o.k.}$$



Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Check the flange plate and beam flange for block shear rupture
The three cases for which block shear must be considered in the **flange plate** are shown in Figure 3-4.

Case 1 involves the tearout of the two blocks outside of the two rows of bolt holes in the flange plate. For this case, the gross tension area has a width of 2(2 in.) = 4 in.

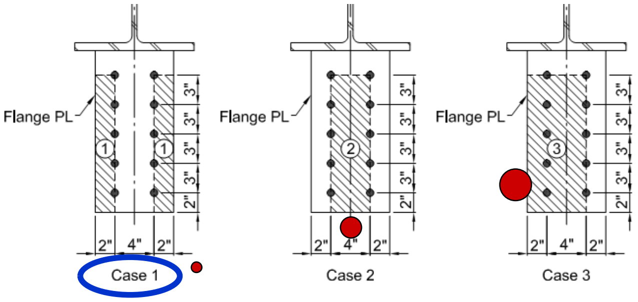


For flange plate

Case 2 is same as Case 1 because both shear and tension areas are the same.

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

The beam flange must also be checked for a failure path similar to Case 1, but need not be checked for the similar failure paths to Case 2 or Case 3 due to the presence of the web.



Only Case 1 applies to the beam due to the presence of the web.

For beam flange

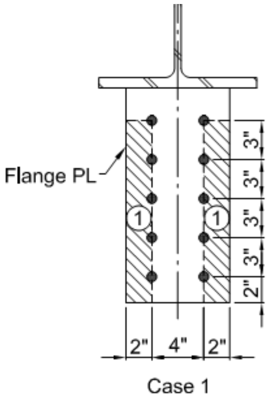
Example 3.4.4 Moment Frame Beam-to-Column Connection Design

The nominal strength for the limit state of block shear rupture is given by AISC *Specification* Equation J4-5:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$$

Check the flange plate for Case 1

The available block shear rupture strength of the flange plate is determined as follows, using AISC *Manual* Tables 9-3a, 9-3b and 9-3c and AISC *Specification* Equation J4-5, with $n = 5$, $l_{ev} = 2$ in., $l_{eh} = 2$ in., and $U_{bs} = 1.0$.

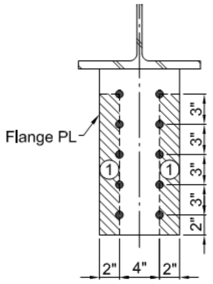


Example 3.4.4 Moment Frame Beam-to-Column Connection Design

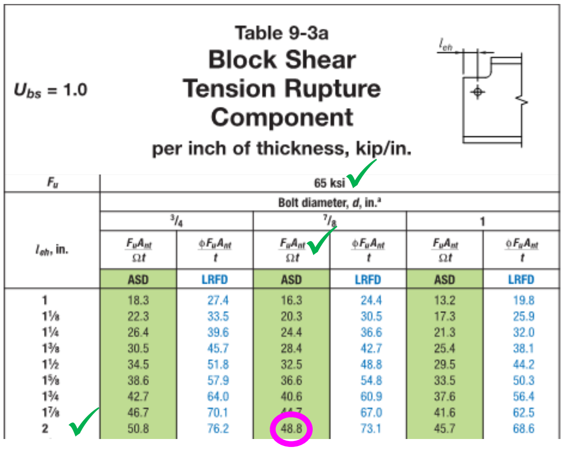
Tension rupture component from AISC *Manual* Table 9-3a:

$$2\left(\frac{F_u A_{nt}}{\Omega t}\right) = 2\left(48.8 \text{ kip/in.}\right)$$

$$= 97.6 \text{ kip/in.}$$



Case 1



$U_{bs} = 1.0$

Table 9-3a
Block Shear
Tension Rupture
Component
per inch of thickness, kip/in.

F_u	65 ksi					
	Bolt diameter, d, in.[a]					
	3/4		7/8		1	
l_{eh}, in.	$\frac{F_u A_{nt}}{\Omega t}$	$\frac{\phi F_u A_{nt}}{t}$	$\frac{F_u A_{nt}}{\Omega t}$	$\frac{\phi F_u A_{nt}}{t}$	$\frac{F_u A_{nt}}{\Omega t}$	$\frac{\phi F_u A_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	18.3	27.4	16.3	24.4	13.2	19.8
1⅛	22.3	33.5	20.3	30.5	17.3	25.9
1¼	26.4	39.6	24.4	36.6	21.3	32.0
1⅜	30.5	45.7	28.4	42.7	25.4	38.1
1½	34.5	51.8	32.5	48.8	29.5	44.2
1⅝	38.6	57.9	36.6	54.8	33.5	50.3
1¾	42.7	64.0	40.6	60.9	37.6	56.4
1⅞	46.7	70.1	44.7	67.0	41.6	62.5
2	50.8	76.2	48.8	73.1	45.7	68.6

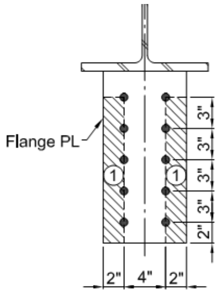
***Steel Manual* Table 9-3a**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Shear yielding component from AISC *Manual* Table 9-3b:

$$2\left(\frac{0.60 F_y A_{gv}}{\Omega t}\right) = 2\left(210 \text{ kip/in.}\right)$$

$$= 420 \text{ kip/in.}$$



Case 1

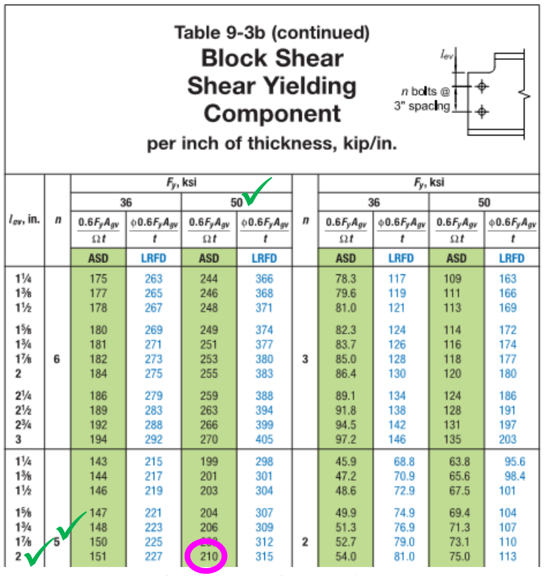


Table 9-3b (continued)
Block Shear
Shear Yielding
Component
per inch of thickness, kip/in.

		F_y, ksi					F_y, ksi			
		36		50			36		50	
l_{ev}, in.	n	$\frac{0.6F_y A_{gv}}{\Omega t}$	$\frac{\phi 0.6F_y A_{gv}}{t}$	$\frac{0.6F_y A_{gv}}{\Omega t}$	$\frac{\phi 0.6F_y A_{gv}}{t}$	n	$\frac{0.6F_y A_{gv}}{\Omega t}$	$\frac{\phi 0.6F_y A_{gv}}{t}$	$\frac{0.6F_y A_{gv}}{\Omega t}$	$\frac{\phi 0.6F_y A_{gv}}{t}$
		ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD
1¼	6	175	263	244	366	3	78.3	117	109	163
1⅜		177	265	246	368		79.6	119	111	166
1½		178	267	248	371		81.0	121	113	169
1⅝		180	269	249	374		82.3	124	114	172
1¾		181	271	251	377		83.7	126	116	174
1⅞		182	273	253	380		85.0	128	118	177
2		184	275	255	383		86.4	130	120	180
2¼		186	279	259	388		89.1	134	124	186
2½		189	283	263	394		91.8	138	128	191
2¾		192	288	266	399		94.5	142	131	197
3		194	292	270	405		97.2	146	135	203
1¼	5	143	215	199	298	2	45.9	68.8	63.8	95.6
1⅜		144	217	201	301		47.2	70.9	65.6	98.4
1½		146	219	203	304		48.6	72.9	67.5	101
1⅝		147	221	204	307		49.9	74.9	69.4	104
1¾		148	223	206	309		51.3	76.9	71.3	107
1⅞		150	225	208	312		52.7	79.0	73.1	110
2		151	227	210	315		54.0	81.0	75.0	113

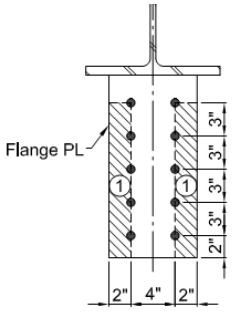
***Steel Manual* Table 9-3b**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Shear rupture component from AISC *Manual* Table 9-3c:

$$2\left(\frac{0.60F_uA_{nv}}{\Omega t}\right)=2(185\text{ kip/in.})$$

$$=370\text{ kip/in.}$$



Case 1

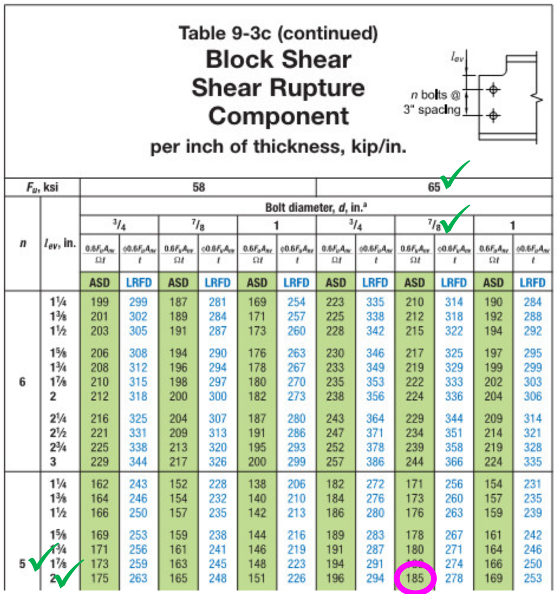


Table 9-3c (continued)
Block Shear
Shear Rupture
Component
per inch of thickness, kip/in.

F_u, ksi		58						65 ✓					
		Bolt diameter, d, in.[a]											
n	l_{ev}, in.	3/4		7/8		1		3/4		7/8 ✓		1	
		$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	1¼	199	299	187	281	169	254	223	335	210	314	190	284
	1⅜	201	302	189	284	171	257	225	338	212	318	192	288
	1½	203	305	191	287	173	260	228	342	215	322	194	292
	1⅝	206	308	194	290	176	263	230	346	217	325	197	295
	1¾	208	312	196	294	178	267	233	349	219	329	199	299
	1⅞	210	315	198	297	180	270	235	353	222	333	202	303
	2	212	318	200	300	182	273	238	356	224	336	204	306
	2¼	216	325	204	307	187	280	243	364	229	344	209	314
	2½	221	331	209	313	191	286	247	371	234	351	214	321
	2¾	225	338	213	320	195	293	252	378	239	358	219	328
	3	229	344	217	326	200	299	257	386	244	366	224	335
5 ✓	1¼	162	243	152	228	138	206	182	272	171	256	154	231
	1⅜	164	246	154	232	140	210	184	276	173	260	157	235
	1½	166	250	157	235	142	213	186	280	176	263	159	239
	1⅝	169	253	159	238	144	216	189	283	178	267	161	242
	1¾	171	256	161	241	146	219	191	287	180	271	164	246
	1⅞	173	259	163	245	148	223	194	291	[illegible]	274	166	250
	2 ✓	175	263	165	248	151	226	196	294	185	278	169	253

***Steel Manual* Table 9-3c**

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

The allowable block shear rupture strength is:

$$\frac{R_n}{\Omega}=\frac{0.60F_uA_{nv}}{\Omega}+\frac{U_{bs}F_uA_{nt}}{\Omega}\leq\frac{0.60F_yA_{gv}}{\Omega}+\frac{U_{bs}F_uA_{nt}}{\Omega}$$

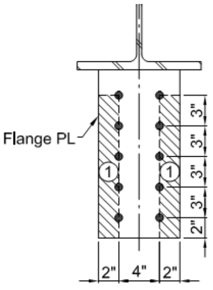
$$=\left(\frac{3}{4}\text{ in.}\right)\begin{bmatrix}370\text{ kip/in.}\\+(1.0)(97.6\text{ kip/in.})\end{bmatrix}\leq\left(\frac{3}{4}\text{ in.}\right)\begin{bmatrix}420\text{ kip/in.}\\+(1.0)(97.6\text{ kip/in.})\end{bmatrix}$$

$$=351\text{ kips}<388\text{ kips}$$

Therefore:

$$\frac{R_n}{\Omega}=351\text{ kips}>141\text{ kips}\quad\textbf{o.k.}$$

P_{af} = 141 kips



Case 1

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Check the flange plate for Case 3

The nominal strength for the limit state of block shear rupture relative to the normal force on the flange plate is:

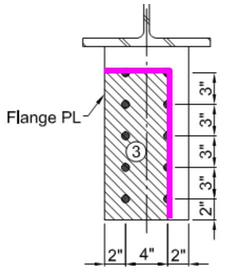
$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\text{ Eq. J4-5})$$

where

$$A_{nt} = \left(\frac{3}{4}\text{ in.}\right)\left[6\text{ in.} - (1.5)\left(\frac{15}{16}\text{in.} + \frac{1}{16}\text{ in.}\right)\right]$$

$$= 3.38\text{ in.}^2$$

$U_{bs} = 1.0$ for uniform tension stress



Case 3

For flange plate

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

The tension rupture component is:

$$U_{bs}F_uA_{nt} = 1.0(65\text{ ksi})\left(3.38\text{ in.}^2\right)$$

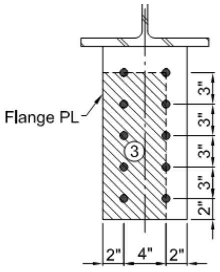
$$= 220\text{ kips}$$

Shear yielding component from AISC *Manual* Table 9-3b:

$$\frac{0.60F_yA_{gv}}{\Omega t} = 210\text{ kip/in.}$$

Shear rupture component from AISC *Manual* Table 9-3c:

$$\frac{0.60F_uA_{nv}}{\Omega t} = 185\text{ kip/in.}$$



Case 3

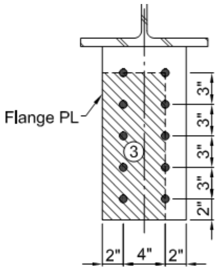
Example 3.4.4 Moment Frame Beam-to-Column Connection Design

The allowable block shear rupture strength is:

$$\begin{aligned}\frac{R_n}{\Omega} &= \frac{0.60F_uA_{nv}}{\Omega}+\frac{U_{bs}F_uA_{nt}}{\Omega} \leq \frac{0.60F_yA_{gv}}{\Omega}+\frac{U_{bs}F_uA_{nt}}{\Omega}\\ &= \left(\frac{3}{4}\text{ in.}\right)\left(185\text{ kip/in.}\right)+\frac{220\text{ kips}}{2.00} \leq \left(\frac{3}{4}\text{ in.}\right)\left(210\text{ kip/in.}\right)+\frac{220\text{ kips}}{2.00}\\ &= 249\text{ kips} < 268\text{ kips}\end{aligned}$$

Therefore:

$$\frac{R_n}{\Omega} = 249\text{ kips} > 141\text{ kips} \quad \textbf{o.k.}$$



Case 3

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Check the beam flange for block shear rupture
Based on a failure path similar to Case 1 in Figure 3-4, the available block shear rupture strength of the beam flange is determined (but not shown – see pg. 3-19 and 3-20 of the SDM):

$$\frac{R_n}{\Omega} = 284\text{ kips} > 141\text{ kips} \quad \textbf{o.k.}$$

Use five rows of 7/8-in.-diameter Group A (thread condition N) bolts in standard holes at a 4-in. gage to connect each flange plate to the beam flange. Use a 2-in. edge distance and a 3-in. spacing for the bolts.

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Check the flange plate for the compression force

The radius of gyration of the flange plate is:

$$\begin{aligned} r &= \frac{t}{\sqrt{12}} \\ &= \frac{\frac{3}{4}\text{ in.}}{\sqrt{12}} \\ &= 0.217\text{ in.} \end{aligned}$$

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

From AISC *Specification* Commentary Table C-A-7.1, use K = 1.2, and L = 3 in.:

$$\begin{aligned} \frac{L_c}{r} &= \frac{KL}{r} \\ &= \frac{1.2(3\text{ in.})}{0.217\text{ in.}} \\ &= 16.6 \end{aligned}$$

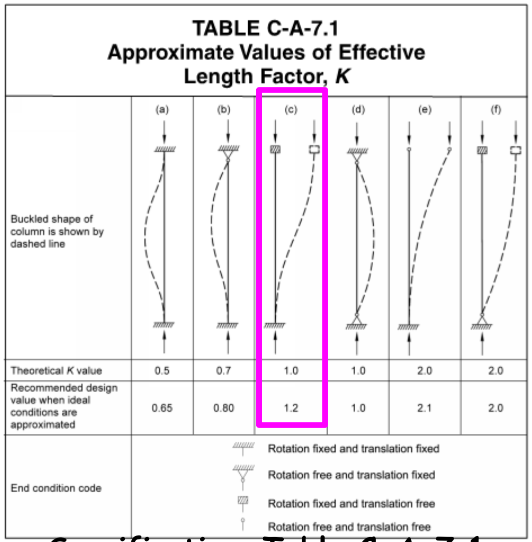


TABLE C-A-7.1
Approximate Values of Effective Length Factor, *K*

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0

End condition code

Rotation fixed and translation fixed

Rotation free and translation fixed

Rotation fixed and translation free

Rotation free and translation free

Specifications* Table *C-A-7.1

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

According to AISC *Specification* Section J4.4, because $L_c/r \le 25$, the compressive strength of the flange plate is:

$$P_n = F_y A_g \qquad (\textit{Spec.}\text{ Eq. J4-6})$$

$$= (50\text{ ksi})(8\text{ in.})\left(\frac{3}{4}\text{ in.}\right)$$

$$= 300\text{ kips}$$

$$\frac{P_n}{\Omega} = \frac{300\text{ kips}}{1.67} = 180\text{ kips} > 141\text{ kips} \quad \textbf{o.k.}$$

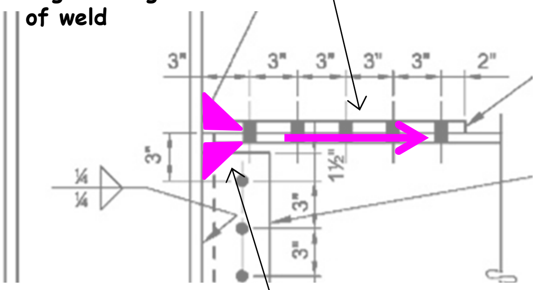
Use 3/4-in. x 8-in. ASTM A572 Grade 50 flange plates.

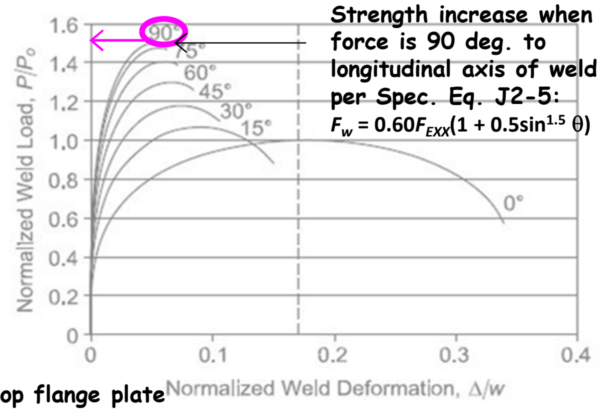
Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Design the weld between the flange plates and column flange

The directional strength increase is used in determining the required weld size. The length of the weld, l_w, is taken to be the width of the 8-in. plate.

Direction of force: 90 deg. to longitudinal axis of weld





Strength increase when force is 90 deg. to longitudinal axis of weld per Spec. Eq. J2-5:
$F_w = 0.60F_{EXX}(1 + 0.5\sin^{1.5}\theta)$

Double fillet weld to top flange plate (similar at bottom flange plate)

Detail: Beam Top Flange

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Determine the weld size

Solving for D_{min} from AISC *Manual* Equation 8-2b and applying the directional strength increase of AISC *Specification* Equation J2-5:

$$D_{min} = \frac{P_{af}}{2(1.5)(0.928\text{ kip/in.})l_w}$$

$$= \frac{141\text{ kips}}{2(1.5)(0.928\text{ kip/in.})(8\text{ in.})}$$

$$= 6.33\text{ sixteenths}$$

Strength increase of 1.5

Eq. 8-2b (ASD): $R_n = 0.928Dl_w$

Use 1/2-in. fillet welds on both sides to connect the flange plates to the column flange.

Example 3.4.4 Moment Frame Beam-to-Column Connection Design

Comment:

The column must be checked for panel zone and stiffening requirements. For further information, see AISC Design Guide No. 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

The final connection design and geometry is shown in Figure 3-5.

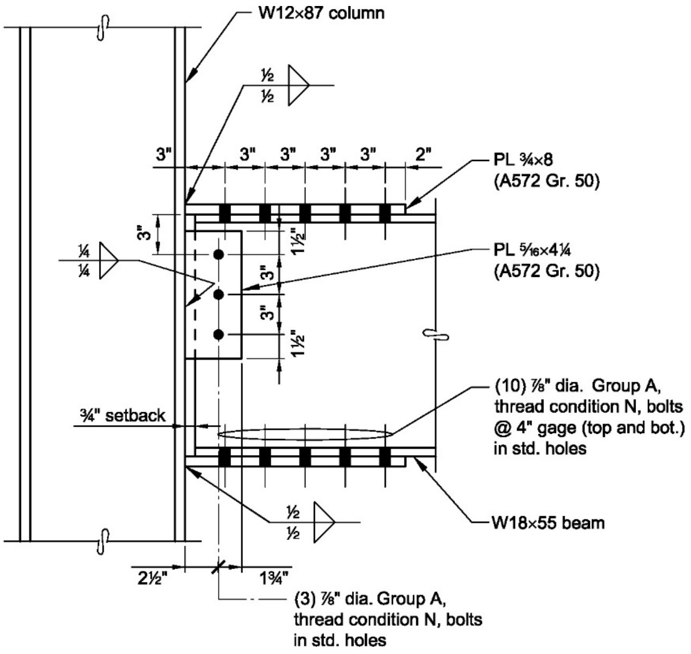


Fig. 3-5. Connection as designed in Example 3.4.4.

Example 3.4.4

Moment Frame Beam-to-Column Connection Design

Third Edition of AISC Seismic Design Manual

Applications of the 2016 *Seismic Provisions* – AISC 341

Smarter.
Stronger.
Steel.

SDM Part 9: ASIC *Seismic Provisions*

In this section of the seminar, we cover:

- *Seismic Provisions* Chapter A: Scope
- *Seismic Provisions* Chapter D: General Member and Connection Design Requirements
  - Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams
  - Example 4.3.2 SMF Column Strength Check
  - Example 4.3.6 SMF Beam-Column Connection Design – RBS
  - Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building
  - Example 4.5.2 SMF Column Splice Design in a Moment Frame Building
  - Example 4.5.3 SMF Column Base Design

Seismic Provisions Chapter A - Scope

Materials

- Unless there is supporting testing, specified minimum yield strength (F_y) for members with inelastic behavior shall not exceed 50 ksi
- Exceptions:
  - In columns for SMF, STMF, Chapter F braced frames, C-SMF, C-OBF, C-SCBF and C-EBF: $F_y \leq 70$ ksi
  - OMF, OCBF, C-OMF, C-OCBF and C-OSW: $F_y \leq 55$ ksi
  - Other permitted materials in BRBF (testing required)

Seismic Provisions Chapter A - Scope

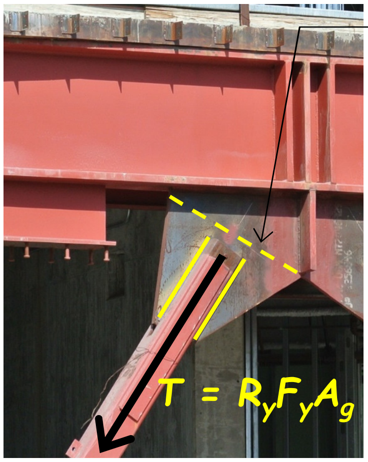
Expected Strength

- When specified in *Seismic Provisions*, required strength of one member shall be based on **expected yield stress**, R_yF_y, of connected member
  - Underlying assumption is that actual yield strength is greater than specified minimum strength – R_y corrects for this – thus, “expected” ≈ mean or average
  - In seismic design, it is not appropriate (i.e., not “conservative”) to underestimate demand on one member created by another because this can alter which elements experience inelastic deformation

Seismic Provisions Chapter A - Scope

Expected Strength

- Example: When designing gusset plate in SCBF, tension demand (required strength) on gusset plate based on $R_yF_yA_g$ of brace



Tension demand on gusset plate is based on expected strength of brace: $R_yF_yA_g$

Seismic Provisions Chapter A - Scope

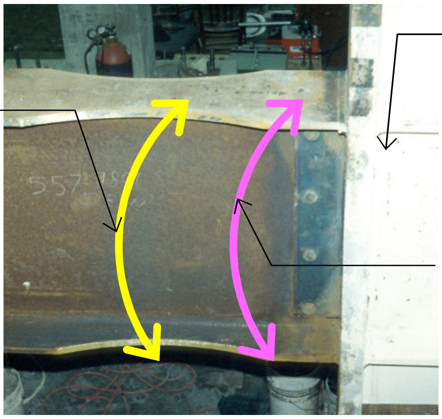
Expected Strength

- Exception: Required and available strength for the same member or connecting element may use R_tF_u (expected tensile strength) and R_yF_y when determining available strength of same member
  - Steel member generating the “demand” is same steel member providing “available strength” – thus, it is reasonable to assume that the steel has the same strength

Seismic Provisions Chapter A - Scope

Expected Strength

- Example: Flexural strength in beam at face of column loaded by the probable plastic moment at a reduced beam section may be calculated using R_yF_y – *they are both the same piece of steel*



Moment at RBS plastic hinge:
$M_{pr} = 1.15R_yF_yZ_{RBS}$
⇧

Flexural strength of column for strong column-weak beam check based on F_y of column since it is NOT the same piece of steel

Flexural strength at gross section of beam: $M_n = R_yF_yZ_x$
⇧

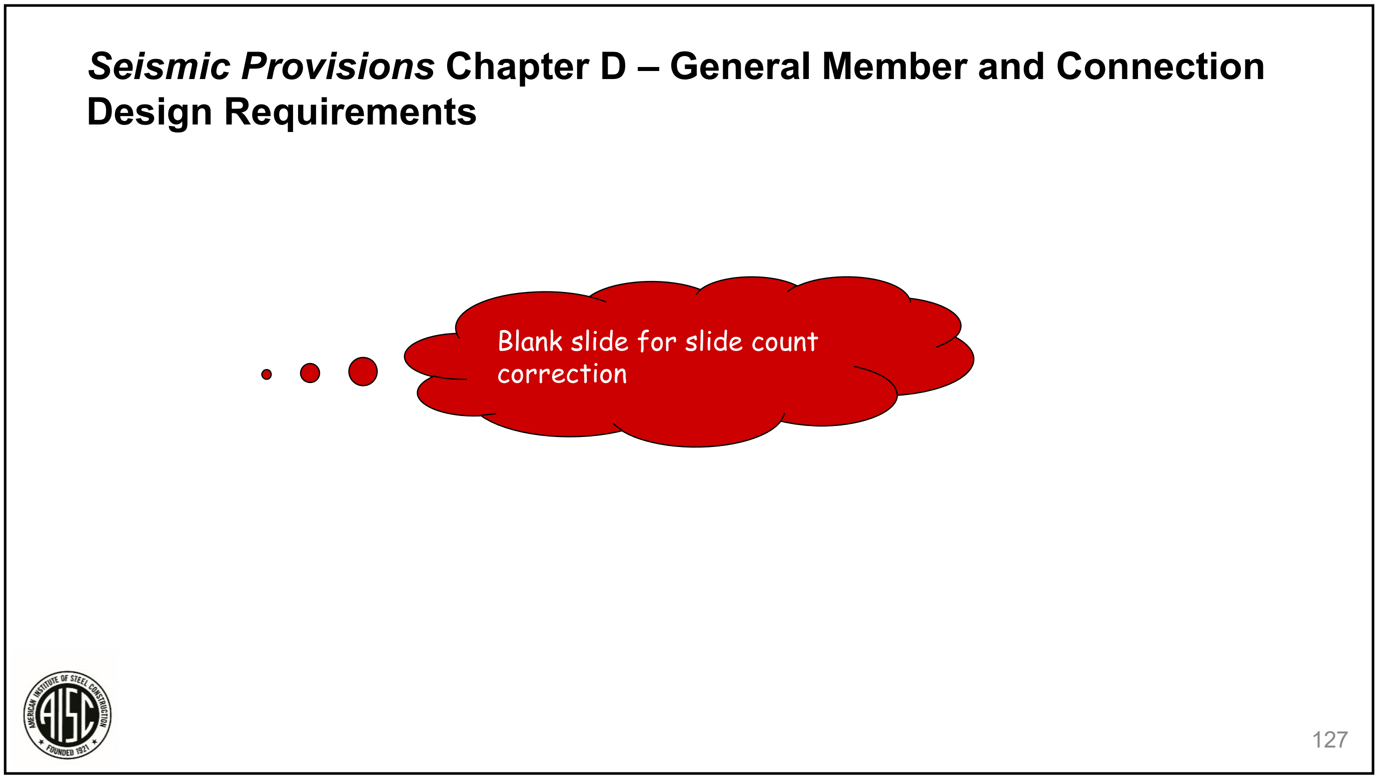
Seismic Provisions Chapter A - Scope

Expected Strength

Table A3.1(Abridged)
R_y and R_t Values for Different Material Specifications

Material Specification	R_y	R_t
ASTM A36 (shapes)	**1.5**	**1.2**
ASTM A572 Gr. 50 or 55 (shapes)	**1.1**	**1.1**
ASTM A992 (shapes)	**1.1**	**1.1**
ASTM A500 HSS, Gr. C (F_y = 50 ksi)	**1.3**	**1.2**
ASTM A1085 HSS (new spec.)	**1.25**	**1.15**
ASTM A36 (plate)	**1.3**	**1.2**

Seismic Provisions Table A3.1



Seismic Provisions Chapter D – General Member and Connection Design Requirements

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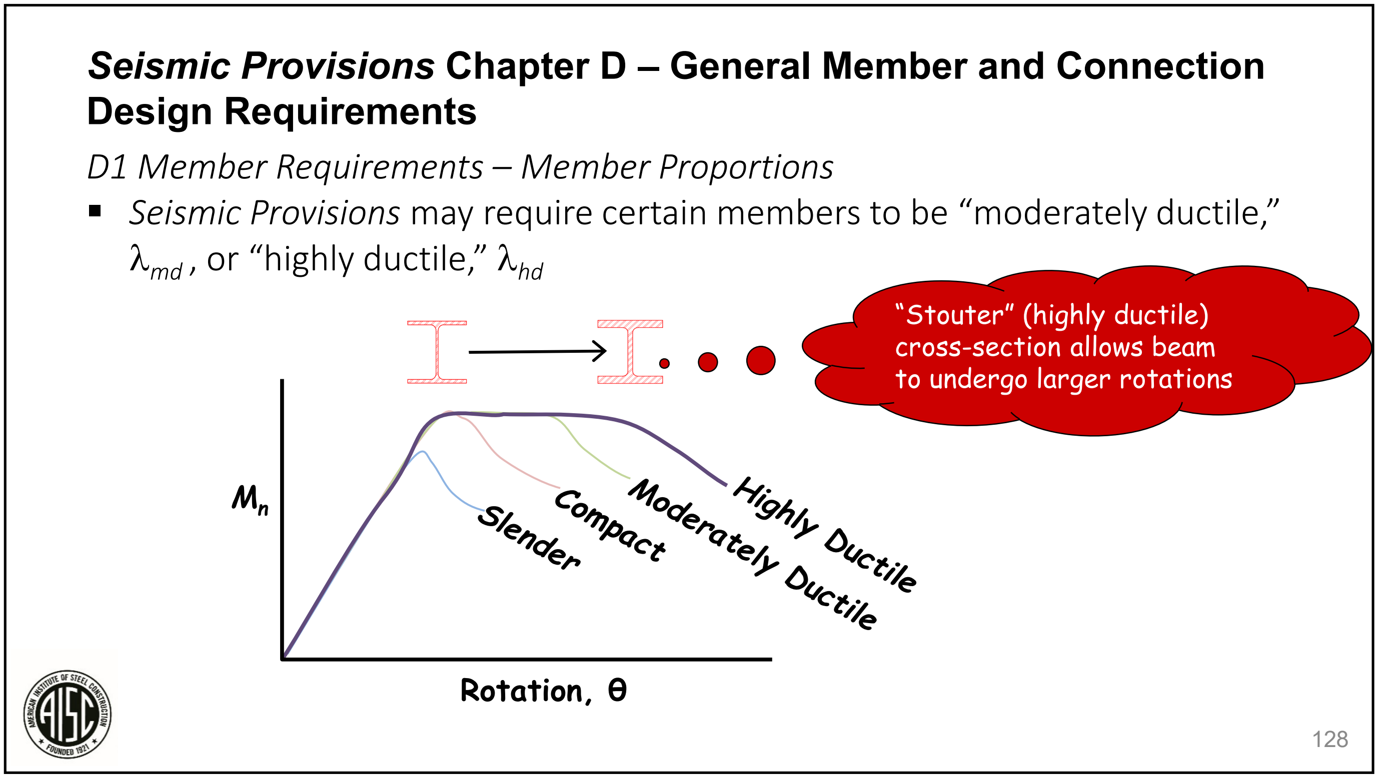

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

- *Seismic Provisions* may require certain members to be "moderately ductile," λ_{md}, or "highly ductile," λ_{hd}

"Stouter" (highly ductile) cross-section allows beam to undergo larger rotations

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

- These requirements may be more stringent than found in *Specification* Table B4.1 (e.g., Seismic Provisions use R_yF_y rather than F_y when checking limiting width-to-thickness ratios)
- These provisions are intended to limit (delay) local flange or local web buckling to help promote more robust inelastic response under cyclic loads
- For some members, these provisions reflect concerns about the consequence of failure in members expected to remain nominally elastic (e.g., columns) and are not necessarily directly related to local buckling concerns

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Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

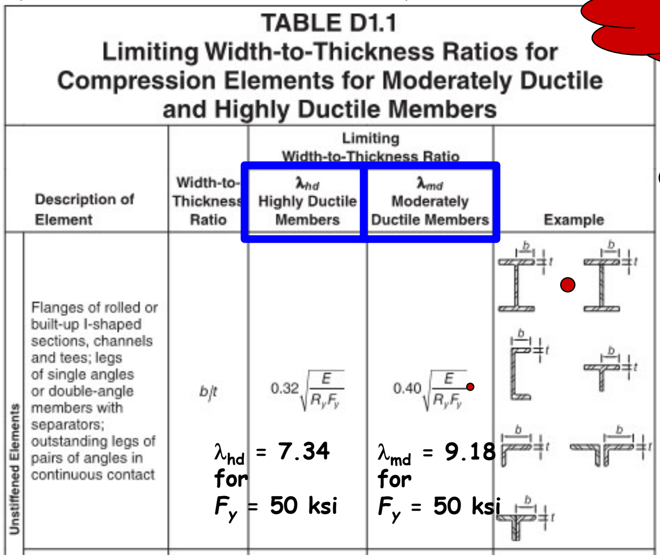


TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_{hd} Highly Ductile Members	 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	$0.32\sqrt{\frac{E}{R_yF_y}}$ λ_{hd} = 7.34 for F_y = 50 ksi	$0.40\sqrt{\frac{E}{R_yF_y}}$ λ_{md} = 9.18 for F_y = 50 ksi	

Note use of R_yF_y rather than F_y as in *Specification*

Seismic Provisions Table D1.1

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

λ_{hd} = 13.7 eliminates most HSS sections once sidewall > 9″ or 10″

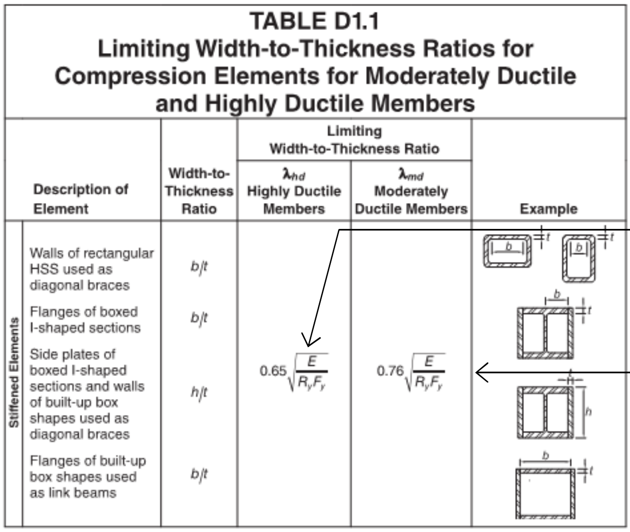


TABLE D1.1
Limiting Width-to-Thickness Ratios for Compression Elements for Moderately Ductile and Highly Ductile Members

	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio		Example
			λ_{hd} Highly Ductile Members	λ_{md} Moderately Ductile Members	
Stiffened Elements	Walls of rectangular HSS used as diagonal braces	b/t	$0.65\sqrt{\frac{E}{R_yF_y}}$	$0.76\sqrt{\frac{E}{R_yF_y}}$	
	Flanges of boxed I-shaped sections	b/t			
	Side plates of boxed I-shaped sections and walls of built-up box shapes used as diagonal braces	h/t			
	Flanges of built-up box shapes used as link beams	b/t			

λ_{hd} = 13.7
for A500 Gr.C (F_y = 50)

λ_{md} = 16.1
for A500 Gr.C (F_y = 50)

Seismic Provisions Table D1.1

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Ductility Requirements

- SDM Table 1-2 identifies member ductility requirement by SFRS

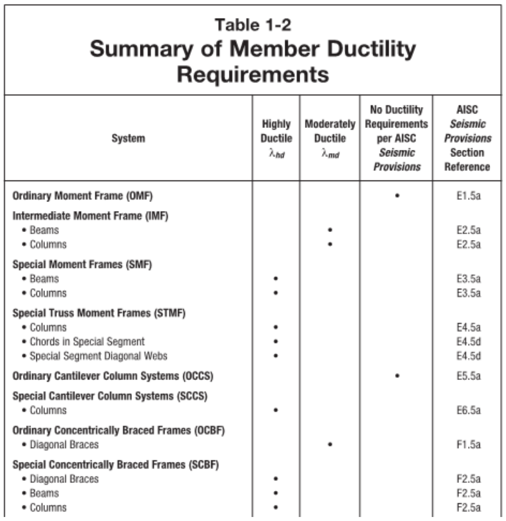


Table 1-2
Summary of Member Ductility Requirements

System	Highly Ductile λ_{hd}	Moderately Ductile λ_{md}	No Ductility Requirements per AISC *Seismic Provisions*	AISC *Seismic Provisions* Section Reference
Ordinary Moment Frame (OMF)			•	E1.5a
Intermediate Moment Frame (IMF)				
• Beams		•		E2.5a
• Columns		•		E2.5a
Special Moment Frames (SMF)				
• Beams	•			E3.5a
• Columns	•			E3.5a
Special Truss Moment Frames (STMF)				
• Columns	•			E4.5a
• Chords in Special Segment	•			E4.5d
• Special Segment Diagonal Webs	•			E4.5d
Ordinary Cantilever Column Systems (OCCS)			•	E5.5a
Special Cantilever Column Systems (SCCS)				
• Columns	•			E6.5a
Ordinary Concentrically Braced Frames (OCBF)				
• Diagonal Braces		•		F1.5a
Special Concentrically Braced Frames (SCBF)				
• Diagonal Braces	•			F2.5a
• Beams	•			F2.5a
• Columns	•			F2.5a

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

- SDM Tables 1-3 to 1-7 identify members that may be used in different SFRS
- Tables cover:
    - W-shapes
    - Angles
    - Rectangular HSS
    - Square HSS
    - Round HSS
    - Pipe

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

- A few comments about SDM Tables 1-3 to 1-7
    - A big time-saver!
    - Table entries are based on member gross sections – modifications to member geometry (e.g., RBS cuts) may allow a member to qualify because width-thickness ratio is reduced
    - Some computer programs will indicate a member doesn't qualify for the same reason (i.e., programs only check the gross member – check by hand)

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

Set up for different F_y

May be used as a F_y = 50 ksi diagonal brace

May not be used as a F_y = 50 ksi diagonal brace

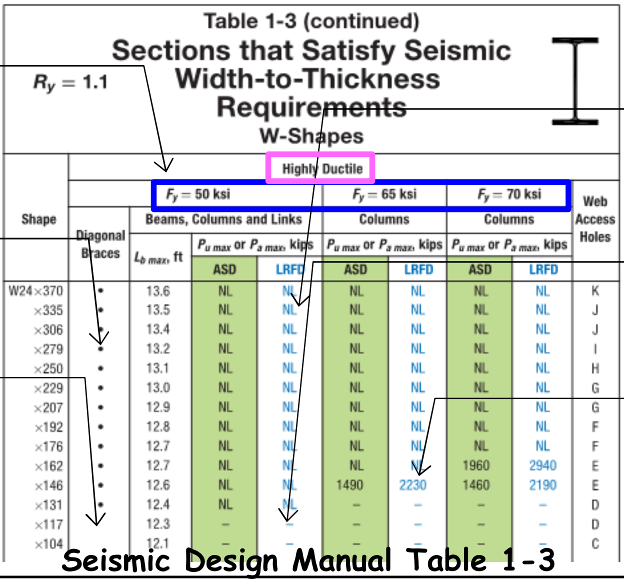


Table 1-3 (continued)

Sections that Satisfy Seismic Width-to-Thickness Requirements

W-Shapes

$R_y = 1.1$

Shape	Highly Ductile								Web Access Holes
	F_y = 50 ksi				F_y = 65 ksi		F_y = 70 ksi		
	Diagonal Braces	Beams, Columns and Links			Columns		Columns		
		$L_{b\,max}$, ft	$P_{u\,max}$ or $P_{a\,max}$, kips		$P_{u\,max}$ or $P_{a\,max}$, kips		$P_{u\,max}$ or $P_{a\,max}$, kips		
			ASD	LRFD	ASD	LRFD	ASD	LRFD	
W24×370	•	13.6	NL	NL	NL	NL	NL	NL	K
×335	•	13.5	NL	NL	NL	NL	NL	NL	J
×306	•	13.4	NL	NL	NL	NL	NL	NL	J
×279	•	13.2	NL	NL	NL	NL	NL	NL	I
×250	•	13.1	NL	NL	NL	NL	NL	NL	H
×229	•	13.0	NL	NL	NL	NL	NL	NL	G
×207	•	12.9	NL	NL	NL	NL	NL	NL	G
×192	•	12.8	NL	NL	NL	NL	NL	NL	F
×176	•	12.7	NL	NL	NL	NL	NL	NL	F
×162	•	12.7	NL	NL	NL	NL	1960	2940	E
×146	•	12.6	NL	NL	1490	2230	1460	2190	E
×131	•	12.4	NL	NL	–	–	–	–	D
×117		12.3	–	–	–	–	–	–	D
×104		12.1	–	–	–	–	–	–	C

NL = not limited by width to thickness requirements

— = may not be used due to width-to-thickness limits

$P_{u\,max}$ = limited by member available strength for F_y = 65 ksi ($P_{a\,max}$ similar)

Seismic Design Manual Table 1-3

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1 Member Requirements – Member Proportions

Set up for different uses and HSS grades

May be used as an SCBF diagonal brace (must be highly ductile)

May not be used as an SCBF diagonal brace

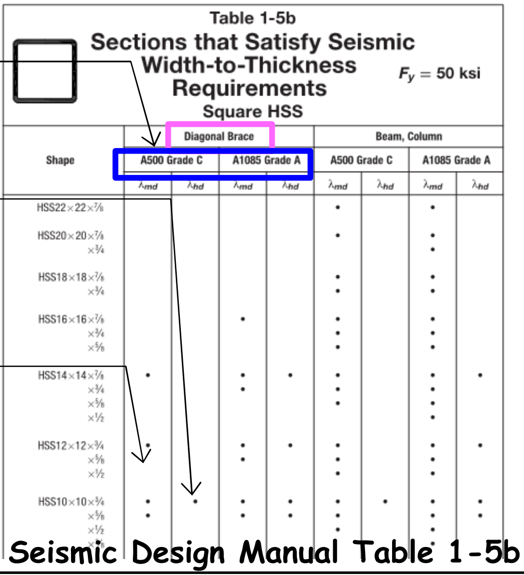


Table 1-5b

Sections that Satisfy Seismic Width-to-Thickness Requirements

Square HSS

F_y = 50 ksi

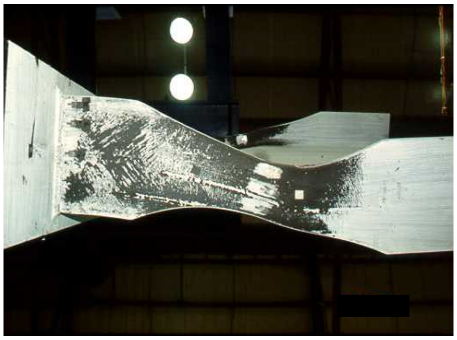
Shape	Diagonal Brace				Beam, Column			
	A500 Grade C		A1085 Grade A		A500 Grade C		A1085 Grade A	
	λ_{md}	λ_{hd}	λ_{md}	λ_{hd}	λ_{md}	λ_{hd}	λ_{md}	λ_{hd}
HSS22×22×⅞					•		•	
HSS20×20×⅞					•		•	
×¾							•	
HSS18×18×⅞					•		•	
×¾					•		•	
HSS16×16×⅞			•		•		•	
×¾					•		•	
×⅝					•		•	
HSS14×14×⅞	•		•	•	•		•	•
×¾			•		•		•	
×⅝					•		•	
×½							•	
HSS12×12×¾	•		•	•	•		•	•
×⅝			•		•		•	
×½					•		•	
HSS10×10×¾	•	•	•	•	•	•	•	•
×⅝	•		•	•	•		•	•
×½					•		•	
×⅜							•	

Seismic Design Manual Table 1-5b

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

- Stability bracing is specified for seismic systems to control lateral-torsional buckling



Lateral-torsional buckling

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

- For moderately and highly ductile members, both flanges must be braced or the section torsionally braced

Lateral bracing provided by concrete structural slab and full-height perpendicular framing

Torsional bracing provided by shallow perpendicular steel framing and stiffener – wood framing was not considered adequate

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

- Beam bracing shall meet requirements of *Specification* Appendix 6 for lateral or torsional bracing where the required strength of the member.
- When applying the required strength equations in *Specification* Appendix 6, the *Seismic Provisions* define the required flexural strength to be used as:

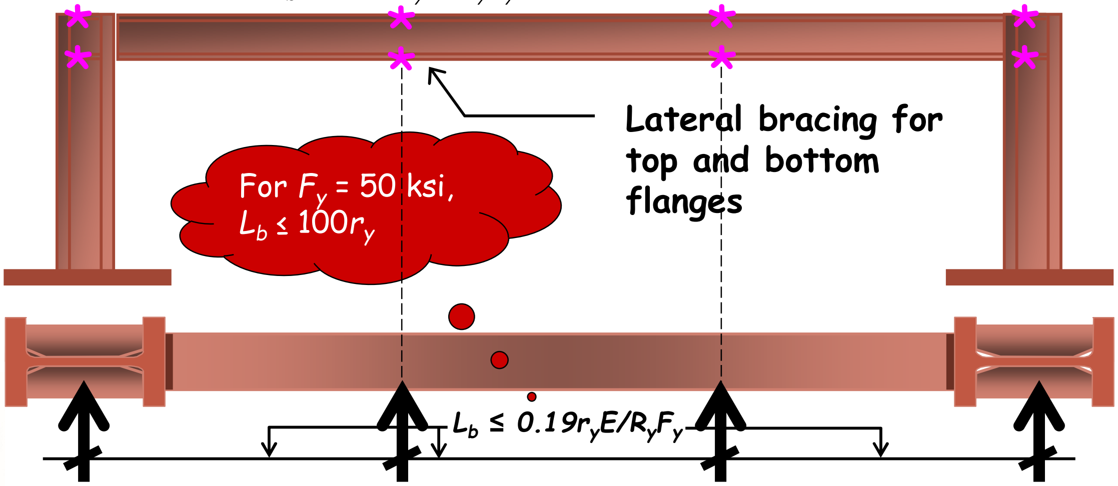
  $M_r = R_y F_y Z / \alpha_s$ (Eq. D1-1)
- When applying the required brace stiffness equations in *Specification* Appendix 6, the *Seismic Provisions* define C_d = 1.0

Not the same C_d as in ASCE 7

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

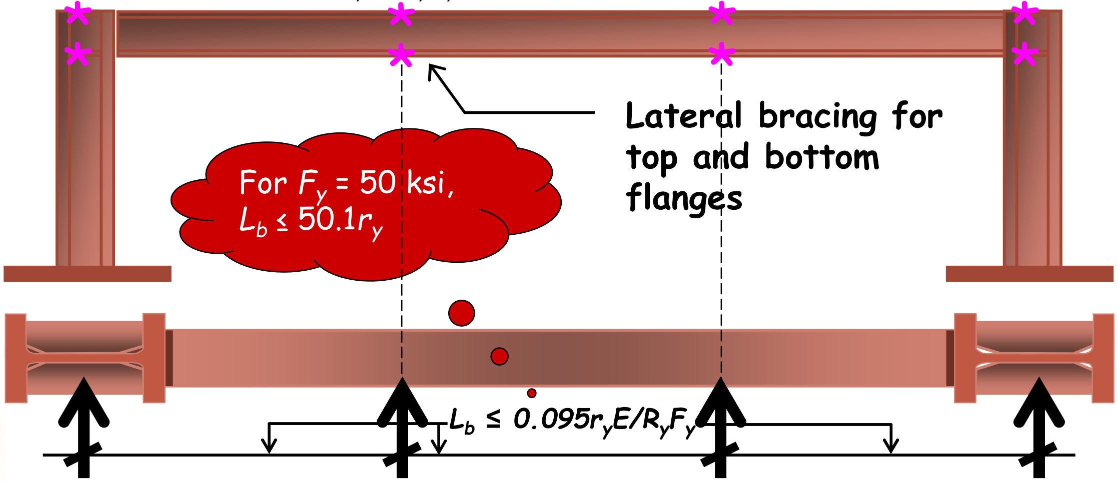
- For **moderately ductile** members, unbraced length between lateral braces shall not exceed $L_b = 0.19 r_y E / R_y F_y$



Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

- For **highly ductile** members, unbraced length between lateral braces shall not exceed $L_b = 0.095 r_y E / R_y F_y$



Seismic Provisions Chapter D – General Member and Connection Design Requirements

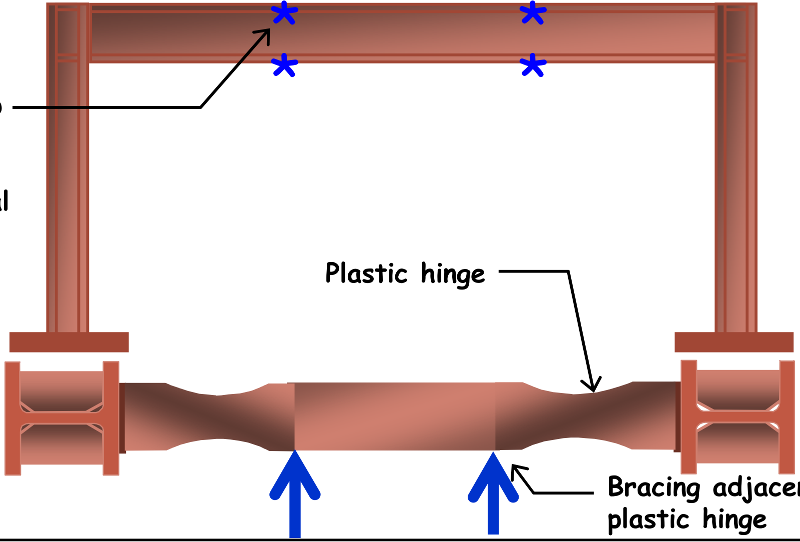
D1.2 Stability Bracing

- At plastic hinges (or directly adjacent thereto):
  - Brace top and bottom flanges or brace with point torsional bracing
  - Required strength of bracing: $P_r = 0.06 R_y F_y Z / (\alpha_s h_o)$ (at each flange for lateral bracing) or $M_r = 0.06 R_y F_y Z / \alpha_s$ (torsional bracing)
  - Bracing stiffness shall satisfy requirements of Appendix 6 of *Specification* but C_d = 1.0 and $M_r = R_y F_y Z / \alpha_s$

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.2 Stability Bracing

- At plastic hinges (or directly adjacent thereto):



Lateral bracing for top and bottom flange (not required if there is a concrete structural slab per AISC 358 for SMF and IMF)

Plastic hinge

Bracing adjacent to plastic hinge

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.3 Protected Zones

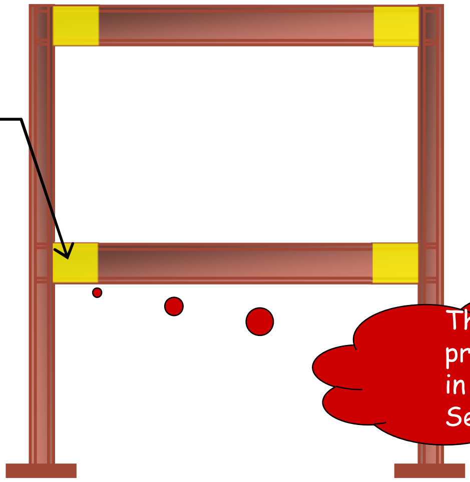
- Discontinuities (per Section I2) are prohibited in protected zones defined by *Seismic Provisions* or AISC 358
  - Examples: Shear studs, welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping, or other construction
  - There are Exceptions per *Seismic Provisions*, AISC 358 or via testing

"Protected zone" is a region expected to experience significant inelastic deformation – it is defined in the *Seismic Provisions* or AISC 358

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.3 Protected Zones

- Protected zones are defined for each system (AISC 341) or each moment frame connection (AISC 358)



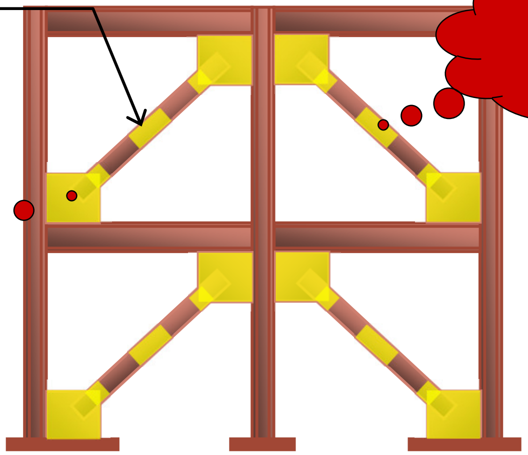
Protected zones in a moment frame – defined in *Seismic Provisions* or *Connection Prequalification Standard* per Sections E2.5c (IMF) and E3.5c (SMF)

There are no protected zones in an OMF per Section E1.5b

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.3 Protected Zones

- Protected zones are defined for each system (AISC 341) or each moment frame connection (AISC 358)



PZ in a concentrically braced frame – defined in *Seismic Provisions* Section F2.5c (SCBF)

Double stud walls (or no walls) may be required to avoid connections to PZ

There are no protected zones in an OCBF (AISC 341 is silent)

Example 4.3.4

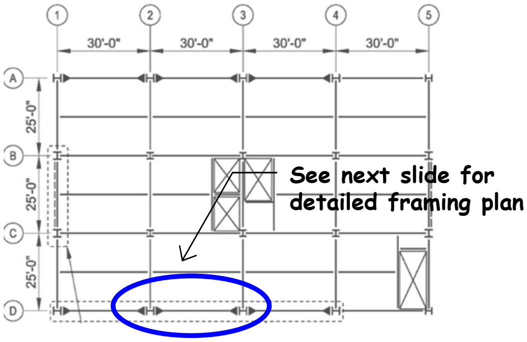
SMF Beam Stability Bracing Design – Equal Depth Beams

- Partial example emphasizing determination of brace location and required stiffness (SDM pg. 4-58)
- Example 4.3.3 (SDM pg. 4-49) confirms beam size and Example 4.3.7 (SDM pg. 4-120) designs bolted flange plate connection
- Example worked in ASD

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

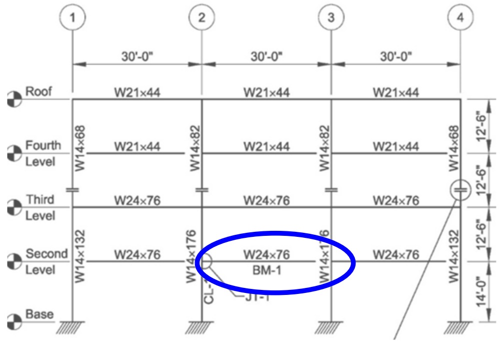
Beam Brace Spacing and Location

- Assume that lateral bracing is not provided by a structural slab. Design stability bracing for the SMF beam BM-1. Assume that the bracing is the same size as the frame beam.



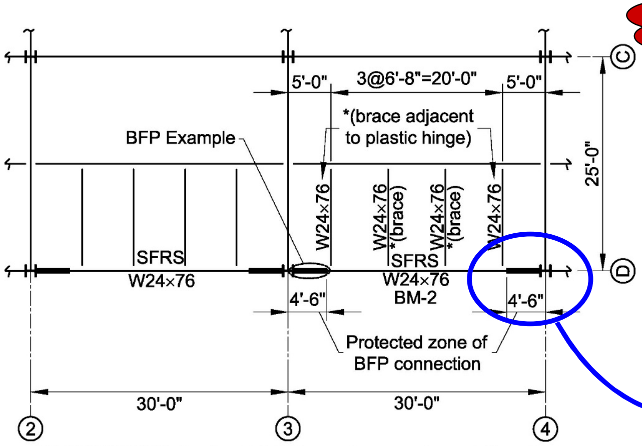
See next slide for detailed framing plan

Overall Framing Plan



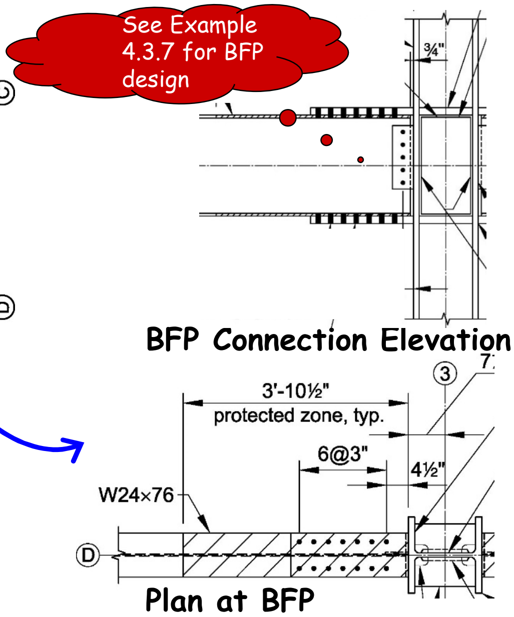
Overall SMF Elevation

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams



*ANSI/AISC 358, Section 7.3.1(7), requires supplemental bracing to be located within a distance of *d* to 1.5*d* from the bolt line farthest from face of the column. AISC *Seismic Provisions* Section D1.2b provides a maximum spacing of lateral braces. These two requirements are satisfied with the spacing shown for the lateral braces (W24×76 infill beams).

Detailed Framing Plan of Bracing



BFP Connection Elevation

Plan at BFP Beam-Column Connection

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

Assume the following material specification: ASTM A992, F_y = 50 ksi

From AISC *Manual* Table 1-1, the geometric properties of the SMF members and infill beams are as follows:

W24×76 beams

d = 23.9 in. t_w = 0.440 in. b_f = 8.99 in.
t_f = 0.680 in.
I_x = 2,100 in.^{4} Z_x = 200 in.^{3} I_y = 82.5 in.^{4}
r_y = 1.92 in.
h_o = 23.2 in.

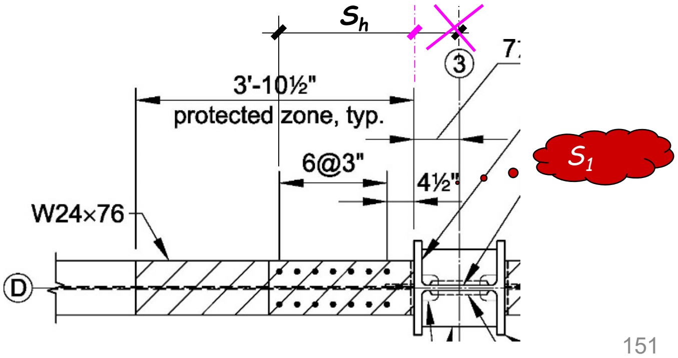
W14×176 column

d = 15.2 in.

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

Beam Brace Spacing and Location (SDM pg. 4-59)

- AISC 358, Section 7.3.1(7), requires a brace located at a distance *d* to 1.5*d* (where *d* represents the beam depth) from the bolt line farthest from the face of the column but not within the protected zone, *pz*.
- For a BFP connection, the extent of the protected zone from the face of the column is $S_h + d$,

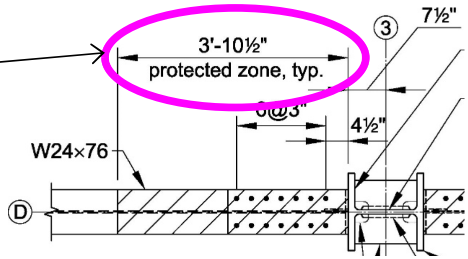


$$S_h = S_1 + (n-1)s$$

$$= 4\frac{1}{2}\text{ in.} + (7-1)(3\text{ in.})$$

$$= 22.5\text{ in.}$$

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams



$$pz = S_h + d$$

$$= 22.5\text{ in.} + 23.9\text{ in.}$$

$$= 46.4\text{ in.}$$

The total distance from the column centerline to the edge of the protected zone includes half the depth of the column, d_c, as follows:

$$pz_{total} = pz + \frac{d_c}{2}$$

$$= 46.4\text{ in.} + \frac{15.2\text{ in.}}{2}$$

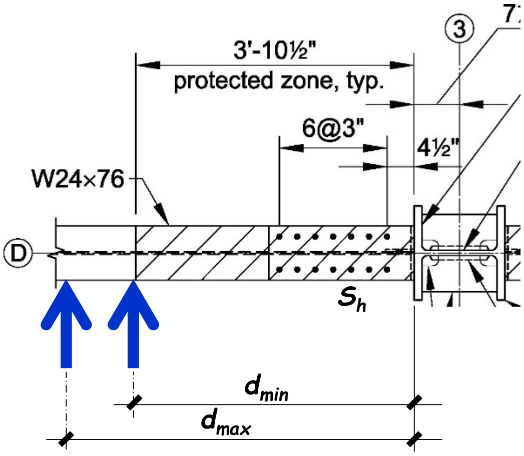
$$= 54.0\text{ in.}$$

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

Thus, the brace adjacent to the plastic hinge must be located within the distance from the face of the column equal to:

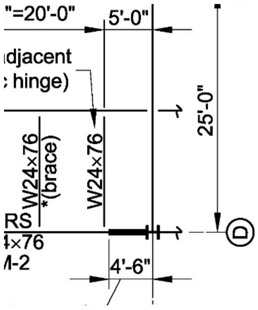
$$d_{min} = pz$$
$$= 46.4 \text{ in.}$$

$$d_{max} = S_h + 1.5d$$
$$= 22.5 \text{ in.} + 1.5(23.9 \text{ in.})$$
$$= 58.4 \text{ in.}$$



Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

The braces nearest the plastic hinges are located at a distance from the face of the column equal to:



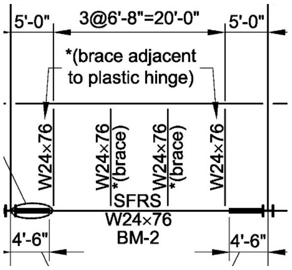
$$d_{BR} = (5 \text{ ft})(12 \text{ in./ft}) - \frac{d_c}{2}$$
$$= 60.0 \text{ in.} - \frac{15.2 \text{ in.}}{2}$$
$$= 52.4 \text{ in.}$$

Checking that d_{BR} is within d and $1.5d$ from the farthest bolt line, but not within the plastic hinge:

$$46.4 \text{ in.} < d_{BR} = 52.4 \text{ in.} < 58.4 \text{ in.} \quad \text{o.k.}$$

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

The maximum spacing of lateral braces is given in AISC *Seismic Provisions* Section D1.2b for highly ductile members as:



$$
\begin{aligned}
L_b &= 0.095 r_y E \big/ \left(R_y F_y\right) \\
&= 0.095\left(1.92 \text{ in.}\right)\left(29{,}000 \text{ ksi}\right) \big/ \left[1.1\left(50 \text{ ksi}\right)\right] \\
&= 96.2 \text{ in.}
\end{aligned}
$$

The spacing of the intermediate lateral braces is 6 ft 8 in. = 80 in. < 96.2 in.; therefore, the intermediate spacing requirement is acceptable.

(Note: Brace connection design from SDM pgs. 4-63 to 4-68 is not shown.)

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

Determine the Required Brace Stiffness (SDM pg. 4-68)

The W24x76 infill beam braces the SMF beam via torsional bracing (e.g., a full depth shear plate connection).

This part of the example examines the required stiffness of the torsional bracing.

Seismic Provisions Section D1.2c.1(c) reference *Specification* Appendix 6 (Section 6.3.2a) for the required brace stiffness. C_d =1.0 and M_r, is taken as the expected plastic flexural strength of the SMF beam.

Referring to AISC Specification Appendix 6, Section 6.3.2a, the required flexural stiffness of the beam is:

$$\beta_{br} = \frac{\beta_T}{1 - \beta_T / \beta_{sec}} \qquad \text{(}\textit{Spec}\text{. Eq. A-6-10)}$$

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

where β_T and β_{sec} are:

$$\beta_T = \Omega \frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \quad \text{(ASD)} \quad (\textit{Spec}.\ \text{Eq. A-6-11b})$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12}\right) \quad (\textit{Spec}.\ \text{Eq. A-6-12})$$

Per *Specification* User Note, β_{sec} may be taken as infinity and $\beta_{br} = \beta_T$ if full-depth stiffener is used

---

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

According to AISC *Seismic Provisions* Equation D1-6 the required flexural strength is:

$$\begin{aligned} M_r &= R_y F_y Z / \alpha_s \\ &= 1.1(50\text{ ksi})(200\text{ in.}^3)/1.5 = 7{,}330\text{ kip-in.} \end{aligned}$$

The overall brace system required stiffness, β_T, is:

$$\begin{aligned} \beta_T &= \frac{3.0(2.4)(30\text{ ft})(12\text{ in./ft})}{4(29{,}000\text{ ksi})(82.5\text{ in.}^4)}\left(\frac{7{,}330\text{ kip-in.}}{1.0}\right)^2 \\ &= 14{,}600\text{ kip-in./rad} \end{aligned}$$

Per *Spec*. Eq. A-6-11b with Ω = 3.0

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

The web distortional stiffness, β_{sec}, is:

$$\beta_{sec} = \frac{3.3(29{,}000\text{ ksi})}{23.2\text{ in.}}\left[\frac{1.5(23.2\text{ in.})(0.440\text{ in.})^3 + \left(\frac{1}{2}\text{ in.}\right)\left(8\frac{1}{4}\text{ in.}\right)^3}{12}\right]$$

$$= 97{,}500\text{ kip-in./rad}$$

Note that because the connection plate is approximately full depth, b_s is assumed as the full width of the connection plate.

Example 4.3.4 SMF Beam Stability Bracing Design – Equal Depth Beams

Therefore, the required flexural stiffness of the brace beam, β_{br}, is:

$$\beta_{br} = \frac{14{,}600\text{ kip-in./rad}}{\left(1 - \frac{14{,}600\text{ kip-in./rad}}{97{,}500\text{ kip-in./rad}}\right)} = 17{,}200\text{ kip-in./rad}$$

Given the brace is rotationally restrained at one end and simply supported at the other end, the brace will deflect in single curvature. The available flexural stiffness of the brace is:

$$\beta_b = \frac{3EI}{L} = \frac{3(29{,}000\text{ ksi})(2{,}100\text{ in.}^4)}{(12.5\text{ ft})(12\text{ in./ft})}$$

$$= 1{,}220{,}000\text{ kip-in./rad} > 17{,}200\text{ kip-in./rad} \quad \text{o.k.}$$

Example 4.3.4

SMF Beam Stability Bracing Design – Equal Depth Beams

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D1.4 Columns

- Required strength determined by requirements for each system
  - Check requirements of “analysis section” for each system
  - Compressive axial strength determined using the **overstrength seismic load**. Simultaneously applied moments may be neglected unless moment results from load applied to column between points of support.
  - For columns common to intersecting frames, consider potential for simultaneous inelasticity (i.e., traditional “100%-30%” rule may not be adequate) – see *Seismic Provisions* Section D1.4a for exceptions.

Example 4.3.2
SMF Column Strength Check

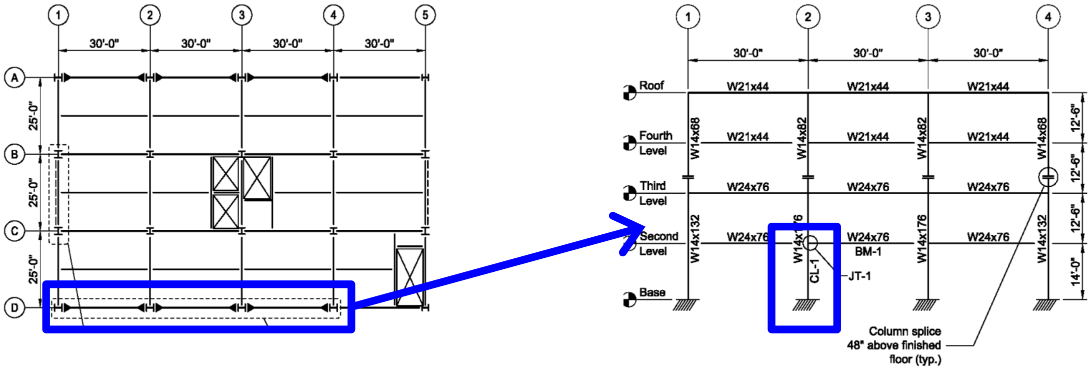
- Example emphasizing determination of frame column required strength and confirming available strength (SDM pg. 4-45)
- Example worked in LRFD

Example 4.3.2 SMF Column Strength Check

Given:

Refer to Column CL-1 on the first level in Figure 4-9. Determine the adequacy of the ASTM A992 W14×176 to resist the required loads.

There is no transverse loading between the column supports in the plane of bending.



Example 4.3.2 SMF Column Strength Check

For this example:

- Use ASCE 7 for calculation of loads.
- The required strengths are determined by a second-order analysis including the effects of P-δ and P-Δ with reduced stiffness as required by the direct analysis method.
- The governing load combination for **shear** that includes seismic load effects, with E_v and E_h incorporated from ASCE 7, Section 12.4.2, is:

Load Combination 6 from ASCE 7, Section 2.3.6 (including the permitted 0.5 factor on L):

$$\begin{aligned} V_u &= \left(1.2 + 0.2S_{DS}\right)D + \rho Q_E + 0.5L \\ &\quad + 0.2S \\ &= 32.0 \text{ kips} \end{aligned}$$

Example 4.3.2 SMF Column Strength Check

AISC *Seismic Provisions* Section D1.4a requires, with limited exceptions, that the **overstrength seismic load** (i.e., the seismic load multiplied by the overstrength factor, Ω_o) be used to calculate required column axial strength.

Moment need not be combined simultaneously with the overstrength seismic axial load in this case because there is no transverse loading between the column supports.

The redundancy factor, ρ, and the overstrength factor need not be applied simultaneously.

Example 4.3.2 SMF Column Strength Check

The governing load combination for **axial strength** that includes the overstrength seismic load, with E_v and E_{mh} incorporated from ASCE 7, Section 12.4.3, is:

Load Combination 6 from ASCE 7, Section 2.3.6 (including the permitted 0.5 factor on *L*):

$$\begin{aligned} P_u &= \left(1.2 + 0.2S_{DS}\right)D + \Omega_o Q_E + 0.5L \\ &\quad + 0.2S \\ &= 249 \text{ kips} \end{aligned}$$

Example 4.3.2 SMF Column Strength Check

The governing load combination for **axial and flexural str**ength that includes seismic load effects, with E_v and E_h incorporated from ASCE 7, Section 12.4.2, is:

Load Combination 6 from ASCE 7, Section 2.3.6:

$$\begin{aligned} P_u &= \left(1.2 + 0.2S_{DS}\right)D + \rho Q_E + 0.5L + 0.2S \\ &= 243 \text{ kips} \end{aligned}$$

$$M_u = \left(1.2 + 0.2S_{DS}\right)D + \rho Q_E + 0.5L + 0.2S$$

$$M_{u\,top} = 125 \text{ kip-ft}$$

$$M_{u\,bot} = -298 \text{ kip-ft}$$

Example 4.3.2 SMF Column Strength Check

From AISC *Manual* Table 2-4, the material properties are ASTM A992, F_y = 50 ksi.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Column

W14×176

A = 51.8 in.^{2} d = 15.2 in. t_w = 0.830 in. b_f = 15.7 in.
t_f = 1.31 in. k_{des} = 1.91 in. $b_f/2t_f$ = 5.97 h/t_w = 13.7
I_x = 2,140 in.^{4} S_x = 281 in.^{3} r_x = 6.43 in. Z_x = 320 in.^{3}
I_y = 838 in.^{4} r_y = 4.02 in.

Beam

W24×76
I_x = 2,100 in.^{4}

Example 4.3.2 SMF Column Strength Check

Column Element Slenderness
AISC *Seismic Provisions* Section E3.5a requires that the stiffened and unstiffened elements of SMF columns satisfy the requirements of Section D1.1 for highly ductile members.

From the AISC *Seismic Provisions* Table D1.1, for flanges of highly ductile members:

$$\lambda_{hd} = 0.32\sqrt{\frac{E}{R_y F_y}} = 0.32\sqrt{\frac{29{,}000\text{ ksi}}{1.1(50\text{ ksi})}} = 7.35$$

$$\begin{aligned}\lambda &= b_f / 2t_f \\ &= 55.97 < \lambda_{hd} = 7.35 \text{ OK}\end{aligned}$$

Therefore, the flanges satisfy the requirements for highly ductile elements.

Example 4.3.2 SMF Column Strength Check

The limiting width-to-thickness ratio for webs of highly ductile members is determined as follows from AISC *Seismic Provisions* Table D1.1 using the governing load case for axial load, including the overstrength seismic load, as stipulated in AISC *Seismic Provisions* Section D1.4a:

$$
\begin{aligned}
C_a &= \frac{P_u}{\phi_c P_y} \\
&= \frac{P_u}{0.90 R_y F_y A_g} \\
&= \frac{249 \text{ kips}}{0.90(1.1)(50 \text{ ksi})(51.8 \text{ in.}^2)} \\
&= 0.0971
\end{aligned}
$$

Example 4.3.2 SMF Column Strength Check

Because $C_a \leq 0.114$:

$$
\begin{aligned}
\lambda_{hd} &= 2.57\sqrt{\frac{E}{R_y F_y}}(1-1.04C_a) \\
&= 2.57\sqrt{\frac{29{,}000 \text{ ksi}}{1.1(50 \text{ ksi})}}\left[1-1.04(0.0971)\right] \\
&= 53.1
\end{aligned}
$$

Therefore, because $\lambda = h/t_w = 13.7 < \lambda_{hd}$, the web satisfies the requirements for highly ductile elements.

Example 4.3.2 SMF Column Strength Check

Alternatively, Table 1-3 in this Manual can be used to confirm that members satisfy the requirements for highly ductile members.

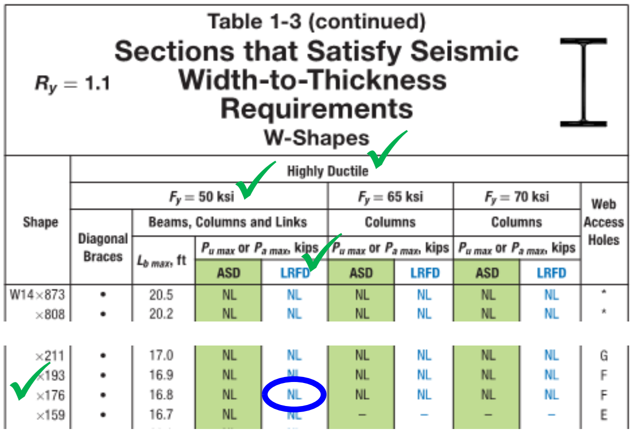


Table 1-3 (continued)
Sections that Satisfy Seismic Width-to-Thickness Requirements
W-Shapes

$R_y = 1.1$

Shape	Highly Ductile								Web Access Holes
	$F_y = 50$ ksi				$F_y = 65$ ksi		$F_y = 70$ ksi		
	Diagonal Braces	Beams, Columns and Links			Columns		Columns		
		$L_{b\,max}$, ft	$P_{u\,max}$ or $P_{a\,max}$, kips		$P_{u\,max}$ or $P_{a\,max}$, kips		$P_{u\,max}$ or $P_{a\,max}$, kips		
			ASD	LRFD	ASD	LRFD	ASD	LRFD	
W14×873	•	20.5	NL	NL	NL	NL	NL	NL	*
×808	•	20.2	NL	NL	NL	NL	NL	NL	*
×211	•	17.0	NL	NL	NL	NL	NL	NL	G
×193	•	16.9	NL	NL	NL	NL	NL	NL	F
×176	•	16.8	NL	NL	NL	NL	NL	NL	F
×159	•	16.7	NL	NL	–	–	–	–	E

Seismic Design Manual **Table 1-3**

Example 4.3.2 SMF Column Strength Check

Effective Length Factor

The direct analysis method in AISC *Specification* Section C3 states that the effective length factor, *K*, of all members is taken as unity unless a smaller value can be justified by rational analysis. Therefore,

$$K_x = 1.0 \quad K_y = 1.0$$

Available Compressive Strength

Using AISC *Manual* Table 6-2, with L_c = 14 ft, the available compressive strength of the W14×176 column is:

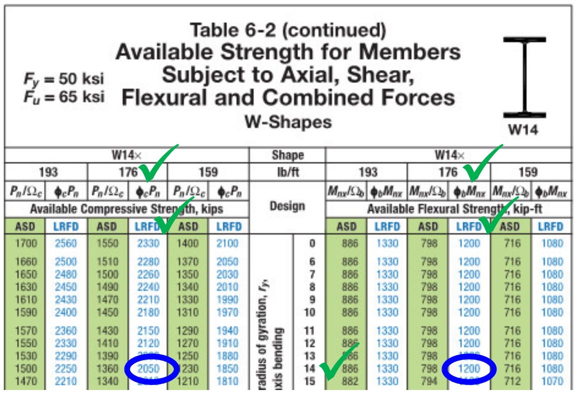
$$\phi_c P_n = 2{,}050 \text{ kips} > 249 \text{ kips} \qquad \textbf{o.k.}$$

Example 4.3.2 SMF Column Strength Check

Available Flexural Strength

Using AISC *Manual* Table 6-2, with L_b = 14 ft, the available flexural strength of the W14×176 column is:

$$\phi_b M_{nx} = 1{,}200 \text{ kip-ft} > \left|-298 \text{ kip-ft}\right| \quad \textbf{o.k.}$$



F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

W14×						Shape		W14×					
193		176 ✓		159		lb/ft		193		176 ✓		159	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD ✓	ASD	LRFD			ASD	LRFD	ASD	LRFD ✓	ASD	LRFD
1700	2560	1550	2330	1400	2100	radius of gyration, r_y, axis bending	0	886	1330	798	1200	716	1080
1660	2500	1510	2280	1370	2050		6	886	1330	798	1200	716	1080
1650	2480	1500	2260	1350	2030		7	886	1330	798	1200	716	1080
1630	2450	1490	2240	1340	2010		8	886	1330	798	1200	716	1080
1610	2430	1470	2210	1330	1990		9	886	1330	798	1200	716	1080
1590	2400	1450	2180	1310	1970		10	886	1330	798	1200	716	1080
1570	2360	1430	2150	1290	1940		11	886	1330	798	1200	716	1080
1550	2330	1410	2120	1270	1910		12	886	1330	798	1200	716	1080
1530	2290	1390	[illegible]	1250	1880		13	886	1330	798	[illegible]	716	1080
1500	2250	1360	2050	1230	1850		14 ✓	886	1330	798	1200	716	1080
1470	2210	1340	[illegible]	1210	1810		15	882	1330	794	[illegible]	712	1070

Steel Manual Table D1.1

Example 4.3.2 SMF Column Strength Check

Combined Loading

Check the interaction of compression and flexure using AISC *Specification* Section H1.1, and the governing load case for combined loading.

$$\frac{P_r}{P_c} = \frac{243 \text{ kips}}{2{,}050 \text{ kips}}$$
$$= 0.119 < 0.2$$

Therefore, use AISC *Specification* Equation H1-1b:

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

Example 4.3.2 SMF Column Strength Check

$$\frac{0.119}{2}+\left(\frac{\left|-298\text{ kip-ft}\right|}{1{,}200\text{ kip-ft}}+0\right)\leq 1.0$$

$$0.308<1.0\quad \textbf{o.k.}$$

Available Shear Strength
Using AISC *Manual* Table 6-2 for the W14×176 column:

$$\phi_v V_n = 378\text{ kips} > 32.0\text{ kips}\quad \textbf{o.k.}$$

The W14×176 is adequate to resist the loads given for Column CL-1.

Example 4.3.2 SMF Column Strength Check

Comment:
The beam and column sizes selected were based on a least-weight solution for drift control; thus, the column size is quite conservative for strength.

Example 4.3.2
SMF Column Strength Check

Seismic Provisions D – General Member and Connection Design Requirements

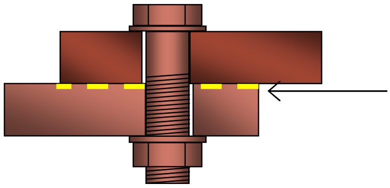
D2 Connections

- Design of connections in SFRS shall be based on:
  - Chapter J of *Specification*
  - Requirements in *Seismic Provisions*
- Column splice and base plate connections for “non-seismic” columns described in Section D2.5 and D2.6

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.2 Bolted Joints

- All bolts in SFRS shall be pretensioned high-strength bolts
- Faying surfaces shall be prepared as slip-critical with a Class A surface (this is the least intensive level of preparation)



"Faying surface" is where steel plies come into contact. Clean faying surface of oil, lose rust, dirt, paint, etc.

- Bolt pretension and faying surface preparation attempts to limit connection deformation (slip) during small and moderate seismic events

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.2 Bolted Joints

- Even though you prepare joint as if it were "slip-critical," you may use the higher bolt "bearing" values (with some exceptions)
  - Recognizes that it is not possible to prevent bolt slip during larger seismic events and bolts will go into bearing

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.2 Bolted Joints

- Bolts and welds shall not be designed to share force in a joint or same force component in a connection
  - Recognizes that stiffer connection elements will resist proportionally more of the load
  - Recognizes that welded joints have less deformation capacity than bolted joints
  - Attempts to ensure that either bolts or welds can resist load independently should stiffer connection component be overloaded

---

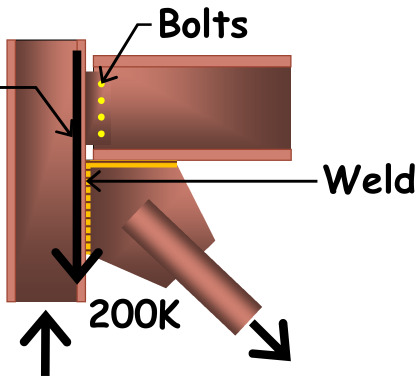
Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.2 Bolted Joints

- Example: Designer may not apportion 200K connection force so that 100K goes to bolts and 100K goes to welds.
- Welds must be designed for 200K or bolts designed for 200K (even if they actually share load)

Consider: Line of action of vertical force (ignore the horizontal force for simplicity)

In reality, vertical force from brace and beam shear may be resisted by bolts and welds, but connection must be *designed* so that *either* welds or bolts take total load



Seismic Provisions Chapter D – General Member and Connection Design Requirements

Specification Section J3.1 – Bolts and Threaded Parts

- High-Strength Bolts
  - Group A: ASTM A3125 Grade A325 replaces ASTM A325
  - Group B: ASTM A3125 Grade A490 replaces ASTM A490

Seismic Provisions Chapter D – General Member and Connection Design Requirements

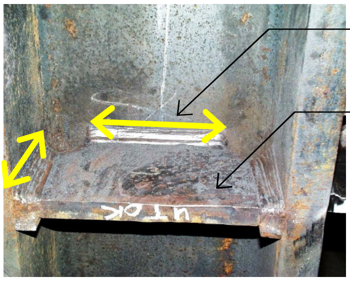
D2.3 Welded Joints

- Design welds per the requirements of Chapter J of the *Specification*
- There are other requirements related to welding in the *Seismic Provisions* and AISC 358, but they are not relevant to the examples in this seminar.

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.4 Continuity Plates and Stiffeners

- Design must account for reduced contact length to member flanges and web based on corner clips per Section I2.4 (which refers to AWS D1.8 Clause 4.1)…
  - Along web, clip extends a distance of at least 1.5” beyond “*k*” detail dimension
  - Along flange, clip shall not exceed 0.5” beyond “k_1” detail dimension

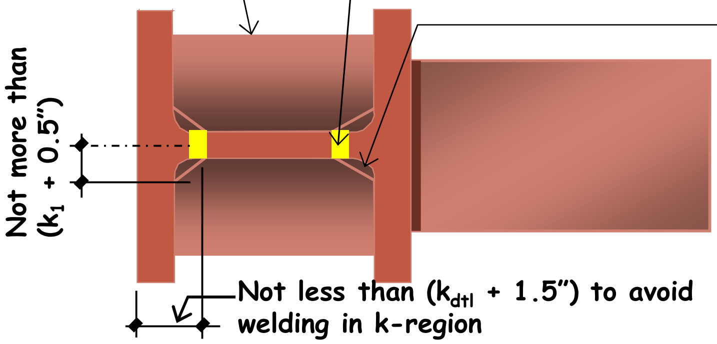


Contact length (reduced by corner clips)

Continuity plate

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.4 Continuity Plates and Stiffeners



Continuity plate

“k-region” (only in web)

“Clip” in continuity plate to avoid column fillet

Not more than (k_1 + 0.5”)

Not less than (k_{dtl} + 1.5”) to avoid welding in k-region

Detailing Requirements for Continuity Plates

Seismic Provisions Chapter D – General Member and Connection Design Requirements

We are skipping ahead to SMF continuity plates in Chapter E to illustrate design requirements

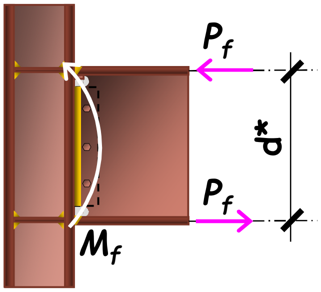
E3.6f SMF Continuity Plates

- Continuity plates shall be provided:
  - When required by testing or prequalification (e.g., AISC 358)
  - When required strength exceeds available column strength per *Specification* Section J10. When beam flange welded to column flange, applied force shall be consistent with maximum moment at column face, M_f.

Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6f Continuity Plates

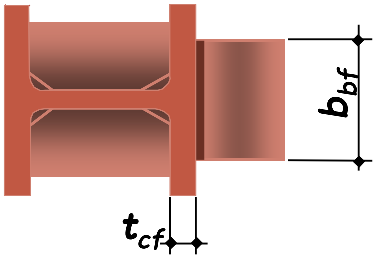
- Per User Note in Section E3.6f.1:
  - Beam flange force, P_f, determined from M_f.
  - For bolted connections: $P_f = \dfrac{M_f}{\alpha_s d^*}$
  - For welded webs: $P_f = \dfrac{0.85M_f}{\alpha_s d^*}$
  - d^* = distance between centroid of beam flanges or beam flange connections to face of column



Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6f Continuity Plates

- Continuity plates shall be provided:
  - When $t_{cf} < t_{lim}$
  - For beam flange welded to W-shape: $t_{lim} \geq \frac{b_{bf}}{6}$ (Eq. E3-8)
  - For beam flange welded to flange of I-shape in boxed W-shape:. $t_{lim} \geq \frac{b_{bf}}{12}$ (Eq. E3-9)



There are not yet continuity plate requirements for boxed sections

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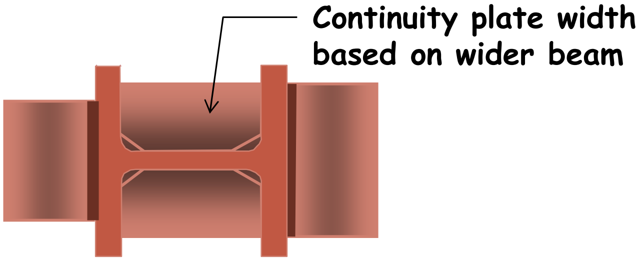
Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6f Continuity Plates

- Continuity plate width (minimum):
  - For wide-flange columns: extend from column web to a point opposite of wider beam flange
  - For boxed wide-flange columns: extend full width from column web to side plate of column

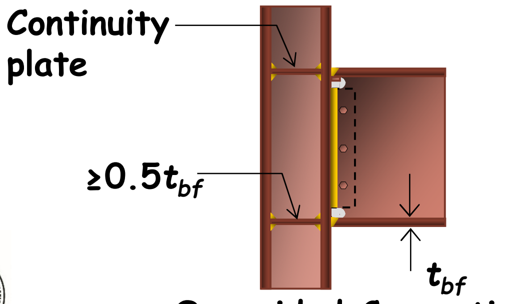


There are not yet continuity plate requirements for boxed sections

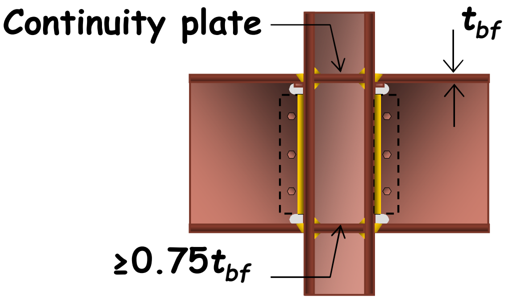
Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6f Continuity Plates

- Continuity plates thickness:
    - One-sided connections: at least one-half beam flange thickness
    - Two-sided connections: at least equal to 75% of thicker of beam flange
    - Also comply with *Specification* Section J10



One-sided Connection



Two-sided Connection

Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6f Continuity Plates

- Weld to **column flange** with CJP groove welds
- Weld to **column web** with CJP groove or fillet welds. Required strength of weld to web equal to smallest of:
    - The sum of the available tensile strengths of the contact areas of the continuity plates to column flanges that have attached beam flanges
    - The available shear strength of the contact area of the plate with the column web or extended doubler plate
    - The available shear strength of the column web when the continuity plate is welded to the column web, or the available shear strength of the doubler plate when the continuity plate is welded to an extended doubler plate

Example 4.3.6

SMF Beam-Column Connection Design – RBS

- Partial example emphasizing “continuity plate” check starting at Step 10 on SDM pg. 4-106
- Earlier part of example verified that RBS connection geometry (e.g., RBS dimensions *a, b* and *c*) are acceptable
- Example worked in LRFD

Example 4.3.6 SMF Beam-Column Connection Design – RBS

RBS Design Procedure Per ANSI/AISC 358

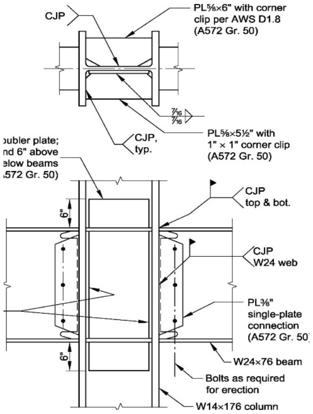
- Step 1. Choose trial values for the RBS dimensions *a*, *b* and *c*. See Example 4.3.3.

⋮

- Step 9. Design the beam web-to-column connection per AISC 358 Section 5.6
- **Step 10. Check continuity plate requirements per AISC Section 358 Chapter 2 (same as *Seismic Provisions* Section E3.6f)**
- Step 11. Check column-beam relationship limitations according to AISC 358 Section 5.4 (same a *Seismic Provisions* Section E3.4a)
- Check the column panel zone according to AISC 358 Section 5.4 (same as Seismic Provisions Section E3.6c)

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Step 10. Check Continuity Plate Requirements (SDM pg. 4-106)
AISC 358 requires that beam flange continuity plates be provided in accordance with the AISC *Seismic Provisions*. Requirements for continuity plates are specified in AISC *Seismic Provisions* Section E3.6f.



Example 4.3.6 SMF Beam-Column Connection Design – RBS

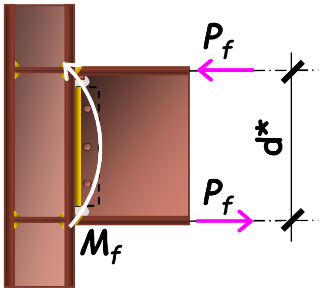
Determine the required strength at the column face as follows.

$$P_f = \frac{M_f}{\alpha_s d^*}$$

where

$$d^* = d - t_f$$
$$= 23.9 \text{ in.} - 0.680 \text{ in.}$$
$$= 23.2 \text{ in.}$$

$$P_f = \frac{9{,}720 \text{ kip-in.}}{1.0(23.2 \text{ in.})}$$
$$= 419 \text{ kips}$$



Example 4.3.6 SMF Beam-Column Connection Design – RBS

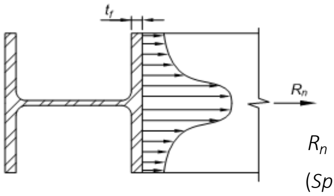
Using P_f = 419 kips, check whether continuity plates are required according to AISC *Specification* Section J10 (i.e., check flange local bending, web local yielding and web local crippling)

Flange local bending

From AISC *Manual* Table 4-1a and Equation 4-4a, the design strength for the limit state of flange local bending is:

$$\phi R_n = P_{fb} \qquad \text{(}\textit{Manual}\text{ Eq. 4-4a)}$$

$$= 483 \text{ kips} > 419 \text{ kips} \qquad \textbf{o.k.}$$



$R_n = 6.25 F_{yf} t_f^2$

(*Spec.* Eq. J-10-1)

Therefore, continuity plates are not required for this limit state.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

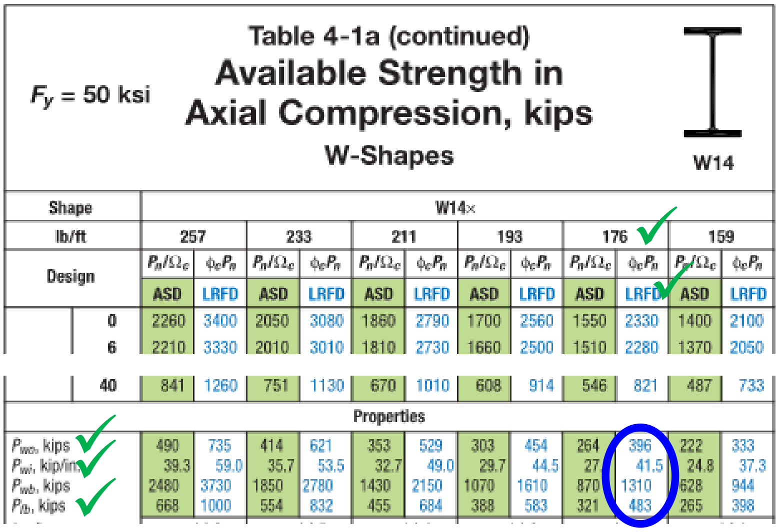


Table 4-1a (continued)

Available Strength in Axial Compression, kips

W-Shapes

F_y = 50 ksi

W14

Shape		W14×											
lb/ft		257		233		211		193		176 ✓		159	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD ✓	ASD	LRFD
	0	2260	3400	2050	3080	1860	2790	1700	2560	1550	2330	1400	2100
	6	2210	3330	2010	3010	1810	2730	1660	2500	1510	2280	1370	2050
	40	841	1260	751	1130	670	1010	608	914	546	821	487	733
Properties													
P_{wo}, kips ✓		490	735	414	621	353	529	303	454	264	396	222	333
P_{wi}, kip/in. ✓		39.3	59.0	35.7	53.5	32.7	49.0	29.7	44.5	27.	41.5	24.8	37.3
P_{wb}, kips		2480	3730	1850	2780	1430	2150	1070	1610	870	1310	628	944
P_{fb}, kips ✓		668	1000	554	832	455	684	388	583	321	483	265	398

Example 4.3.6 SMF Beam-Column Connection Design – RBS

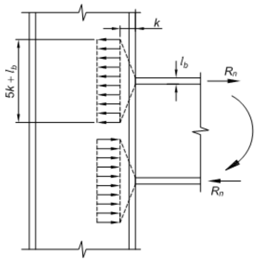
Web local yielding

Concentrated flange force is not applied near the end of the column (greater than *d* of the column); use AISC *Manual* Table 4-1a and Equation 4-2a to determine the design strength for the limit state of web local yielding.

$$\phi R_n = P_{wo} + P_{wi} l_b \qquad l_b = t_{fb} \qquad (\textit{Manual}\ \text{Eq. 4-2a})$$

$$= 396\text{ kips} + (41.5\text{ kip/in.})(0.680\text{ in.})$$

$$= 424\text{ kips} > 419\text{ kips} \quad \textbf{o.k.}$$



Therefore, continuity plates are not required for this limit state.

$R_n = F_{yw} t_w (5k + l_b)$ (*Spec.* Eq. J-10-2)

$P_{wo} = \phi 5 F_{yw} t_w k$

$P_{wi} = \phi F_{yw} t_w$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Web local crippling

Concentrated flange force is not applied near the end of the column (greater than *d*/2 of the column); use AISC *Specification* Equation J10-4 to determine the design strength for the limit state of web local crippling.

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \qquad (\textit{Spec}.\ \text{Eq. J10-4})$$

$$= 0.80(0.830\text{ in.})^2\left[1 + 3\left(\frac{0.680\text{ in.}}{15.2\text{ in.}}\right)\left(\frac{0.830\text{ in.}}{1.31\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(1.31\text{ in.})}{0.830\text{ in.}}}(1.0)$$

$$= 890\text{ kips}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

$$\begin{aligned}\phi R_n &= 0.75(890 \text{ kips}) \\ &= 668 \text{ kips} > 419 \text{ kips} \quad \text{o.k.}\end{aligned}$$

Therefore, continuity plates are not required for this limit state.

The limit states of web sidesway buckling and web compression buckling (AISC *Specification* Sections J10.4 and J10.5) are not applicable. Therefore, according to AISC *Specification* Section J10, continuity plates are not required.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

However, the AISC *Seismic Provisions* also require that the column flange thickness exceed the following to avoid using continuity plates:

$$t_{lim} = \frac{b_{bf}}{6} \qquad (\textit{Prov}. \text{ Eq. E3-8})$$

$$\begin{aligned} &= \frac{8.99 \text{ in.}}{6} \\ &= 1.50 \text{ in.}\end{aligned}$$

$$t_{cf} = 1.31 \text{ in.} < 1.50 \text{ in.} \quad \text{n.g.}$$

Because 1.31 in. < 1.50 in., continuity plates are required.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Alternatively, the W14×176 column could be upsized to a W14×211 to avoid the need for continuity plates. For the purposes of this example, the column size will not be changed, and continuity plates will be provided.

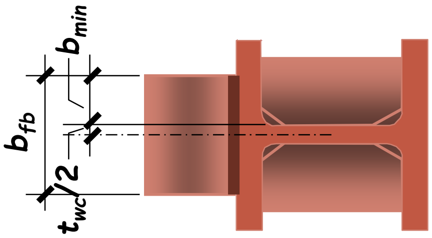
Design Continuity Plates

Determine continuity plate width
According to AISC *Seismic Provisions* Section E3.6f.2(a), continuity plates should, at a minimum, extend from the column web to a point opposite the tips of the beam flanges.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Minimum continuity plate width:

$$\begin{aligned} b_{min} &= \frac{b_{fb} - t_{wc}}{2} \\ &= \frac{8.99\text{ in.} - 0.830\text{ in.}}{2} \\ &= 4.08\text{ in.} \end{aligned}$$



Example 4.3.6 SMF Beam-Column Connection Design – RBS

Maximum continuity plate width (continuity plates extend to the edge of the column flange):

$$\begin{aligned} b_{max} &= \frac{b_{fc} - t_{wc}}{2} \\ &= \frac{15.7\text{ in.} - 0.830\text{ in.}}{2} \\ &= 7.44\text{ in.} \end{aligned}$$

Select a continuity plate width between 4.08 in. and 7.44 in.: use 6-in.-wide continuity plates.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Determine continuity plate thickness

The continuity plate thickness is determined from the requirements of AISC *Specification* Section J10 and AISC *Seismic Provisions* Section E3.6f.2(b).

Because AISC *Specification* Section J10 does not require continuity plates, only AISC *Seismic Provisions* Section E3.6f.2(b)(2) applies, which requires a minimum continuity plate thickness equal to 75% of the thicker beam flange thickness.

$$\begin{aligned} t_{st} &= 0.75 t_{fb} \\ &= 0.75(0.680\text{ in.}) \\ &= 0.510\text{ in.} \end{aligned}$$

Use 5/8-in. × 6-in. ASTM A572 Grade 50 continuity plates on both sides of the web.

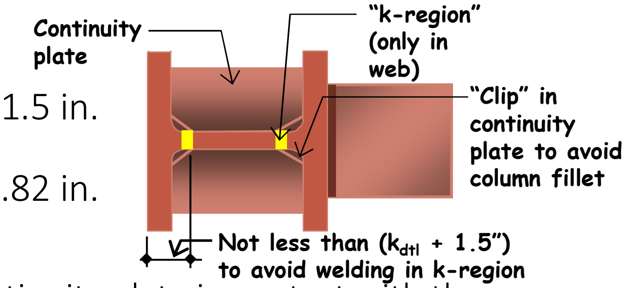
Example 4.3.6 SMF Beam-Column Connection Design – RBS

Determine size of corner clips on continuity plates

AISC *Seismic Provisions* Section I2.4 refers to AWS D1.8, clause 4.1, for corner clips on continuity plates. According to AWS D1.8, clause 4.1, the corner clip along the web is to extend a distance of at least 1.5 in. beyond the k_{det} dimension of the column.

$$k_{det} - t_{cf} + 1.5\text{ in.} = 2\frac{5}{8}\text{ in.} - 1.31\text{ in.} + 1.5\text{ in.}$$

$$= 2.82\text{ in.}$$



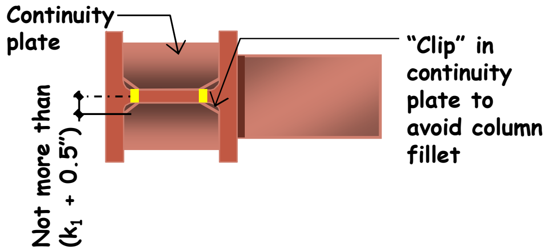
Use a 2-7/8-in. clip on the side of the continuity plate in contact with the column web.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

According to AWS D1.8, clause 4.1, the corner clip along the flange is not to exceed a distance of 1/2 in. beyond the k_1 dimension of the column.

$$k_1 - \frac{t_{cw}}{2} + \frac{1}{2}\text{ in.} = 1\frac{5}{8}\text{ in.} - \frac{0.830\text{ in.}}{2} + \frac{1}{2}\text{ in.}$$

$$= 1.71\text{ in.}$$

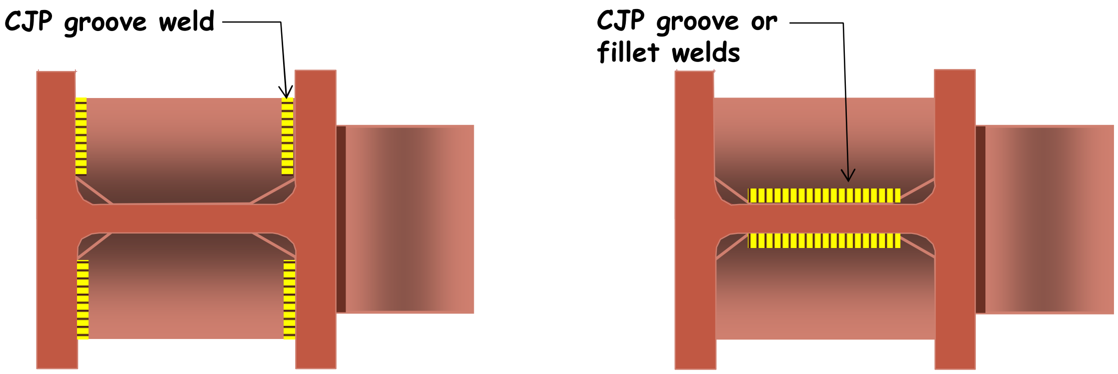


Use a 1-1/2-in. clip on the side of the continuity plate in contact with the column flange.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Continuity plate welding

- According to AISC *Seismic Provisions* Section E3.6f.2(c), continuity plates are to be welded to the column flanges using CJP groove welds.
- Welds between the continuity plates and the column web may be groove welds or fillet welds.



Example 4.3.6 SMF Beam-Column Connection Design – RBS

According to AISC *Seismic Provisions* Section E3.6f.2(c), the required strength of the continuity plate to column web weld is the lesser of:

i. The **sum of the available tensile strengths** of the contact areas of the continuity plates to column flanges that have attached beam flanges

ii. The **available shear strength** of the contact area of the plate with the column web or extended doubler plate

iii. The **available shear strength** of the column web when the continuity plate is welded to the column web, or the available shear strength of the doubler plate when the continuity plate is welded to an extended doubler plate

AISC 341-10 had 4 checks. 4th check replaced with explicit design for P_f

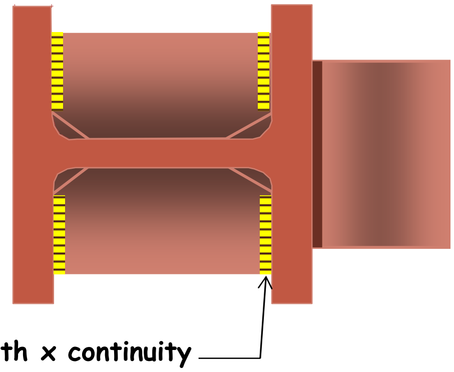
Example 4.3.6 SMF Beam-Column Connection Design – RBS

For (i):

$$\phi T_n = \phi F_y(\text{contact area})$$

$$= 0.90(50\text{ ksi})(2)\left(6\text{ in.} - 1\frac{1}{2}\text{ in.}\right)\left(\frac{5}{8}\text{in.}\right)$$

$$= 253\text{ kips}$$

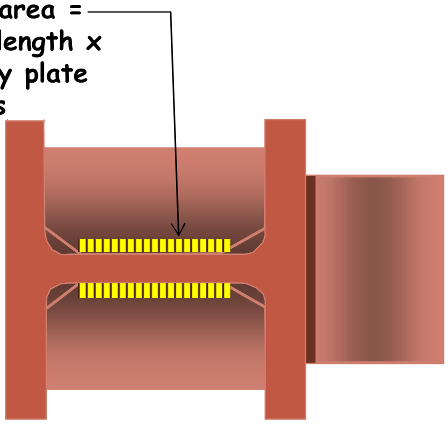


Contact length x continuity plate thickness

Example 4.3.6 SMF Beam-Column Connection Design – RBS

For (ii):

Contact area = contact length x continuity plate thickness



$$\phi V_n = \phi 0.60 F_y(\text{contact area})$$

$$\text{contact length} = d_c - 2(t_{cf} + \text{clip length})$$

$$= 15.2\text{ in.} - 2\left(1.31\text{ in.} + 2\frac{7}{8}\text{ in.}\right)$$

$$= 6.83\text{ in.}$$

$$\phi V_n = 1.00(0.60)(50\text{ ksi})(6.83\text{ in.})\left(\frac{5}{8}\text{ in.}\right)$$

$$= 128\text{ kips}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

For (iii):

$$\phi V_n = 483 \text{ kips}$$

Later in the seminar, shear strength of column web and doubler plate will be calculated.

(ii) controls since it is the smallest required strength. The required strength of the continuity plate to column web weld is 128 kips.

The required leg size of double-sided fillet welds over the contact length is:

$$D = \frac{R_u}{(1.392 \text{ kip/in.})l_w} \qquad \text{(\textit{Manual} Eq. 8-2a)}$$

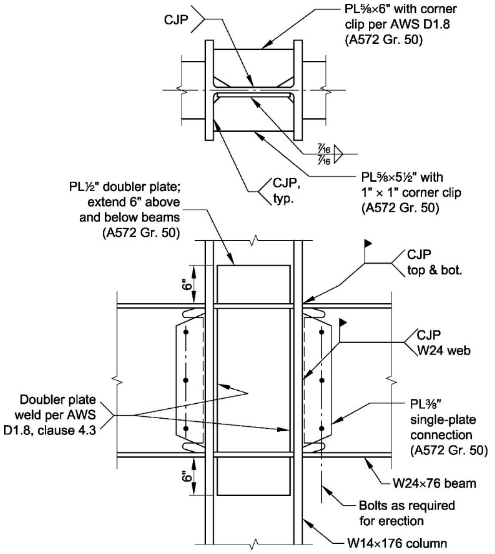
$$= \frac{128 \text{ kips}}{2(1.392 \text{ kip/in.})(6.83 \text{ in.})}$$

"2" reflects use of a double-sided fillet weld

$$= 6.73 \text{ sixteenths}$$

Here, 7/16-in. double-sided fillet welds are required; use CJP welds instead.

Example 4.3.6 SMF Beam-Column Connection Design – RBS



Note: For weld backing requirements, and treatment of weld tabs see ANSI/AISC 358, Chapter 3.

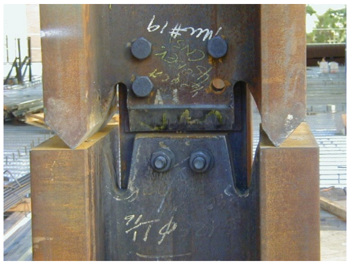
Alternative 1 - Extended Doubler Plate

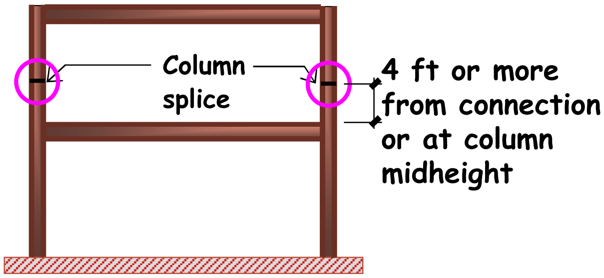
Example 4.3.6
SMF Beam-Column Connection Design – RBS

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.5 Column Splices

- The following applies to *all* columns (gravity and seismic)
    - All column splices shall be located 4 ft or more from beam-to-column connections



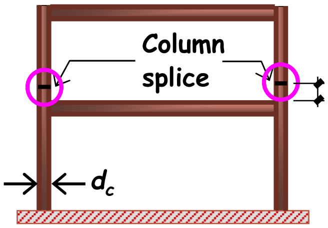


Seismic Provisions Chapter D – General Member and Connection Design Requirements

Note that OSHA regulations may overrule these exceptions

D2.5 Column Splices

- Exceptions:
    - Locate at column mid-height if clear height is less than 8 ft.
    - If flanges and webs are joined using CJP welds, splice may be located no closer to beam-column connection than depth of column
    - Composite columns



Column splice located closer than 4 ft. (but no closer than d_c) if made with CJP welds

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Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.5 Column Splices

- Required strength of splices in SFRS columns shall be greater of:
    - Required strength of column determined for each system
    - Required strength of column determined using overstrength seismic load
- Required strength of welded SFRS splices subject to net tension shall satisfy:
    - Available strength of PJP welds, if used, must be ≥ 200% of required strength
    - Available strength for each flange splice shall be at least $0.5R_yF_yb_ft_f/\alpha_s$ based on smaller connected column

See exceptions when PJP welds meet certain conditions (e.g., E2.6g, E3.6g and E4.6g)

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.5 Column Splices

- Required shear strength of *all* column splices shall not be less than $M_{pc}/(\alpha_s H)$ for each orthogonal axis
    - M_{pc} based on smaller column
    - M_{pc} based on direction under consideration (it may vary by axis)
    - H is story height

Note: Per Section E4.6g, required shear strength for SMF is $\Sigma M_{pc}/\alpha_s H$

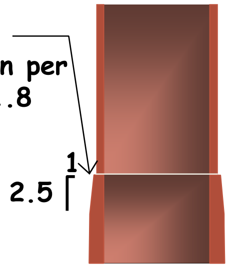
Seismic Provisions Chapter D – General Member and Connection Design Requirements

E3.6 SMF Column Splices (IMF and STMF similar)

Skipping ahead to SMF column splices

- Meet requirements of Section D2.5 unless required strength determined via appropriate nonlinear analysis
- Welds shall be CJP. PJP welds may be used if:
    - $F_y \leq 60$ ksi
    - Thicker flange is at least 5% thicker than thinner flange
    - PJP weld provides minimum effective throat of 85% of thinner flange
    - Smooth transition between thicker and thinner weld
    - Tapered transitions are used
    - Meets various detailing requirements (see E3.6g.2d and e)

Tapered transition per AWS D1.8 (4.2)



Example 4.5.1

Gravity Column Splice Design in a Moment Frame Building

- Partial example emphasizes determination of required splice strength (SDM pg. 4-187)
- Example worked in LRFD

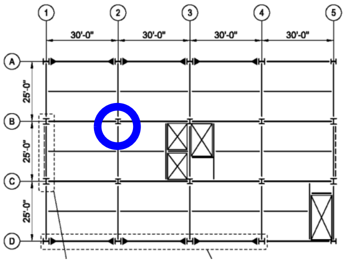
Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

Given:

Refer to the floor plan shown in Figure 4-8 and the SMF elevation shown in Figure 4-9.

Design a splice using bolted flange plates between the third and fourth levels for the gravity column located at the intersection of grids 2 and B.

Use ASTM A572 Grade 50 for all splice material. The column sizes above and below the splice are ASTM A992 W12×40 and W12×58, respectively.



Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

Solution:

From AISC *Manual* Table 2-4, the beam and column material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 2-5, the splice material properties are as follows:

ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

From AISC *Manual* Table 1-1, the column geometric properties are as follows:

Lower Shaft

W12×58
d = 12.2 in. d_{det} = 12-1/4 in. t_f = 0.640 in. b_f = 10.0 in.
Z_x = 86.4 in.^{3} Z_y = 32.5 in.^{3}

Upper Shaft

W12×40
d = 11.9 in. d_{det} = 12 in. t_f = 0.515 in. b_f = 8.01 in.
Z_x = 57.0 in.^{3} Z_y = 16.8 in.^{3}

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

AISC *Seismic Provisions* Sections D2.1, D2.5a and D2.5c have requirements for gravity column splices. Note that these gravity column splice provisions are equally applicable to gravity column splices in braced-frame buildings.

Check splice location

AISC *Seismic Provisions* Section D2.5a requires that the splice be located a minimum of 4 ft from the beam-to-column connections. The three exceptions to this requirement do not apply for this building.

Assume that the gravity column splices are at the same vertical elevation as the SMF column splices shown in Figure 4-9 (i.e., 4 ft. above finish floor). This location satisfies AISC *Seismic Provisions* Section D2.5a

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

Required Shear Strength of Splice in Minor Axis of Column

AISC *Seismic Provisions* Section D2.5c requires that, with respect to both orthogonal axes, the column splice be able to develop a required shear strength equal to:

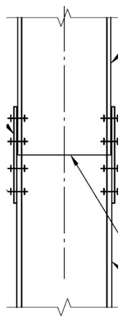
$$V_u = \frac{M_{pc}}{\alpha_s H}$$

where α_s =1.0 for LRFD.

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

In the minor axis of the column, the required shear strength of the splice is:

$$
\begin{aligned}
V_{uy} &= \frac{F_y Z_y}{\alpha_s H} \\
&= \frac{(50\text{ ksi})(16.8\text{ in.}^3)}{1.0(12.5\text{ ft})(12\text{ in./ft})} \\
&= 5.60\text{ kips}
\end{aligned}
$$



The shear force to be resisted by each flange splice plate is half of M_{pc}/H. Therefore, for one splice plate:

$$
\begin{aligned}
V_{uy} &= \frac{5.60\text{ kips}}{2} \\
&= 2.80\text{ kips}
\end{aligned}
$$

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

Note that the smaller column, the W12×40, controls the required shear strength, as is stipulated in AISC *Seismic Provisions* Section D2.5c.

Conservatively ignoring frictional resistance between the upper and lower shafts due to column dead load, this force will be resisted by the splice material.

Required Compressive Strength of Splice

With the upper shaft centered on the lower shaft, the splice will not be required to transfer any compressive loads if the upper shaft is finished to bear on the lower shaft. Because a note stating, “finish to bear,” is provided on the detail, Case I-A applies from AISC *Manual* Part 14, Table 14-3.

Example 4.5.1 Gravity Column Splice Design in a Moment Frame Building

The balance of the example showing the design of the bolted splice may be found starting on SDM Page 4-189.

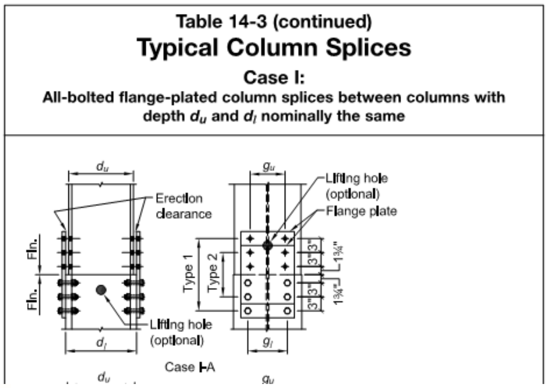
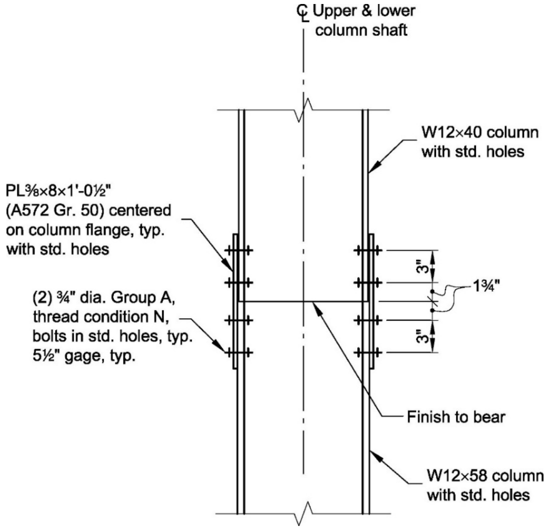


Table 14-3 (continued)

Typical Column Splices

Case I:

All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same



Example 4.5.1

Gravity Column Splice Design in a Moment Frame Building

Example 4.5.2
SMF Column Splice Design

- Example emphasizes determination of required splice strength **(SDM pg. 4-200)**
- The example demonstrates how the *Seismic Provisions* column splice requirements govern the design vs. code-based required strength
- Example worked in LRFD

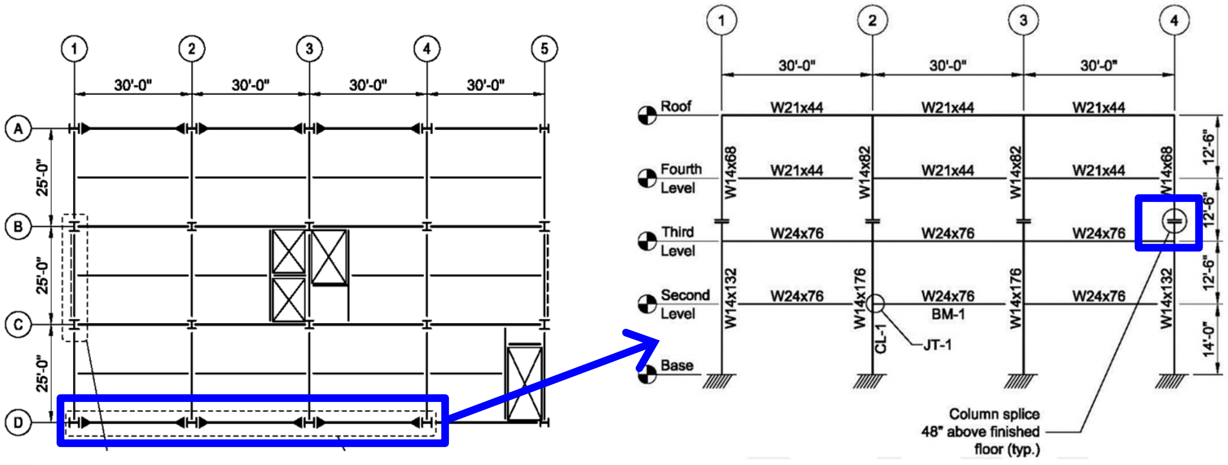
Example 4.5.2 SMF Column Splice Design

Given:
Design a splice for the SMF column located on grid 4 in Figure 4-9. The column material is ASTM A992.

For this example:

- Use ASCE 7 for calculation of loads.
- The required strengths are determined by a second-order analysis including the effects of P-δ and P-Δ with reduced stiffness as required by the direct analysis method.

Example 4.5.2 SMF Column Splice Design



Example 4.5.2 SMF Column Splice Design

The required **compressive strength** of the column is:

Load Combination 6 from ASCE 7, Section 2.3.6 (including the permitted 0.5 factor on *L*):

$$\begin{aligned} P_u &= (1.2 + 0.2S_{DS})D + \Omega_o Q_E \\ &\quad + 0.5L + 0.2S \\ &= 140 \text{ kips} \end{aligned}$$

The required **tensile strength** of the column is:

$$\begin{aligned} T_u &= (0.9 - 0.2S_{DS})D + \Omega_o Q_E \\ &= 15.3 \text{ kips} \end{aligned}$$

Example 4.5.2 SMF Column Splice Design

The required **shear strength** of the column is:

Load Combination 6 from ASCE 7, Section 2.3.6 (including the permitted 0.5 factor on *L*):

$$\begin{aligned} V_u &= \left(1.2 + 0.2S_{DS}\right)D + \Omega_o Q_E \\ &\quad + 0.5L + 0.2S \\ &= 47.2 \text{ kips} \end{aligned}$$

Note: Per E3.6g, this shear will be checked against $\Sigma M_{pc}/H$ for SMF

Example 4.5.2 SMF Column Splice Design

From ASCE 7, use Seismic Design Category D, $\Omega_o = 3$, $\rho = 1.0$, and $S_{DS} = 1.0$.

Assume that there is no transverse loading between the column supports in the plane of bending and that the connections into the column minor axis produce negligible moments on the column.

From AISC *Manual* Table 2-4, the column material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Example 4.5.2 SMF Column Splice Design

From AISC Manual Table 1-1, the column geometric properties are as follows:

Upper Shaft

W14×68

$A = 20.0$ in.^{2} $d = 14.0$ in. $b_f = 10.0$ in. $t_f = 0.720$ in.

$t_w = 0.415$ in. $Z_x = 115$ in.^{3}

Lower Shaft

W14×132

$Z_x = 234$ in.^{3}

There is no net tensile load effect on the column; therefore, the requirements of AISC *Seismic Provisions* Section D2.5b(2) do not apply.

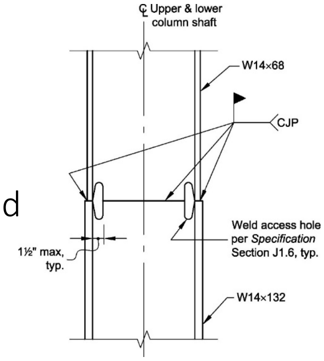
Example 4.5.2 SMF Column Splice Design

Splice Connection

CJP groove welds are used to splice the column webs and flanges directly as shown in Figure 4-42 and in accordance with the provisions of AISC *Seismic Provisions* Section E3.6g.

Required shear strength of the web splice

Per AISC *Seismic Provisions* Section D2.5c, the required shear strength of the web splice is equal to the greater of the required strength determined using AISC *Seismic Provisions* Section D2.5b(1), and the following:



$$V_u = \frac{\Sigma M_{pc}}{\alpha_s H}$$

Example 4.5.2 SMF Column Splice Design

where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the column sections above and below the splice for the direction in question. Because this requirement is for web splices, ΣM_{pc} in the major axis of the column will be considered.

$$\begin{aligned} V_u &= \frac{\Sigma M_{pc}}{\alpha_s H} \\ &= \frac{F_y\left(Z_{xtop}+Z_{xbot}\right)}{\alpha_s H} \\ &= \frac{(50\text{ ksi})\left(115\text{ in.}^3+234\text{ in.}^3\right)}{1.0(12.5\text{ ft})(12\text{ in./ft})} \\ &= 116\text{ kips} \end{aligned}$$

Example 4.5.2 SMF Column Splice Design

Using the load combinations in ASCE 7 including the overstrength seismic load, the required shear strength is given as:

$V_u = 47.2$ kips

Therefore $\frac{\sum M_{pc}}{\alpha_s H}$ governs in determining the required shear strength.

For the limit state of shear yielding on the gross section of the smaller column, according to AISC *Specification* Section G2, the available shear strength is:

$$\begin{aligned} \phi_v V_n &= \phi_v 0.6 F_y A_w C_{v1} \\ &= 1.00(0.6)(50\text{ ksi})(14.0\text{ in.})(0.415\text{ in.})(1.0) \\ &= 174\text{ kips} > 116\text{ kips} \quad \textbf{o.k.} \end{aligned}$$

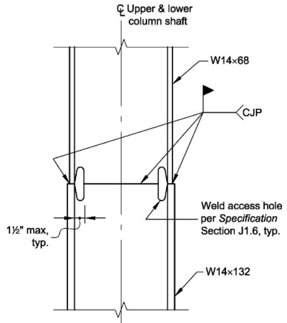
Example 4.5.2 SMF Column Splice Design

Using AISC *Specification* Equation J4-4, the minimum web depth to satisfy the limit state of shear rupture on the net section is:

$$\begin{aligned} d_w &= \frac{V_u}{\phi 0.60 F_u t_w} \\ &= \frac{116 \text{ kips}}{0.75(0.60)(65 \text{ ksi})(0.415 \text{ in.})} \\ &= 9.56 \text{ in.} \end{aligned}$$

Example 4.5.2 SMF Column Splice Design

Therefore, the maximum length of each weld access hole, l_h, permitted in the direction of the web is:



$$\begin{aligned} l_h &= \frac{1}{2}\left[d - 2t_f - d_w\right] \\ &= \frac{1}{2}\left[14.0 \text{ in.} - 2(0.720 \text{ in.}) - 9.56 \text{ in.}\right] \\ &= 1.50 \text{ in.} \end{aligned}$$

Therefore, specify that the access holes for the flange splice welds may not extend more than 1-1/2 in. measured perpendicular to the inside flange surface as shown in Figure 4-42.

Example 4.5.2 SMF Column Splice Design

Location of Splice
AISC *Seismic Provisions* Section D2.5a requires that splices be located 4 ft away from the beam-to-column flange connection. The clear distance between the beam-to-column connections is approximately 10.8 ft.

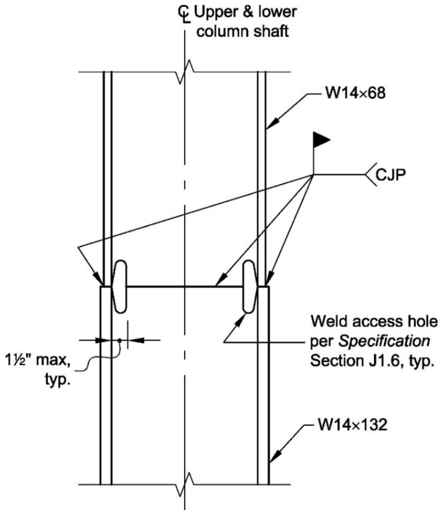
Because the webs and flanges are joined by CJP welds, AISC *Seismic Provisions* Section D2.5a(b) permits the splice to be located a minimum of the column depth (14.0 in.) from the beam-to-column flange connection.

The column splice location shown in Figure 4-9 is acceptable.

Example 4.5.2 SMF Column Splice Design

Additional Weld Requirements
Per AISC *Seismic Provisions* Section A3.4b, the filler metal used to make the splice welds must satisfy AWS D1.8/D1.8M, clauses 6.1, 6.2 and 6.3. Additionally, AISC *Seismic Provisions* Section D2.5d requires that weld tabs be removed.

AISC *Specification* Section J1.6 provides additional requirements for weld access hole geometry. The final connection design is shown in Figure 4-42.



Example 4.5.2

SMF Column Splice Design in a Moment Frame Building

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.6 Column Bases

- Required strength of all column base elements shall satisfy requirements in *Specification*
- Design concrete elements, including anchor rods, per ACI 318.
- Required axial strength of column bases and attachment to foundation is summation of vertical and horizontal components of required strength of steel elements connected to column base

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.6 Column Bases

- (a) Diagonal braces: Horizontal component determined from required strength of diagonal brace connections to SFRS
- (b) Columns: Required strength of horizontal component is lesser of:
    - $2R_yF_yZ/(\alpha_sH)$
    - Shear calculated using overstrength seismic load
- (c) Summation of required strength of horizontal components shall not be less than $0.7F_yZ/(\alpha_sH)$ of the column

---

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.6 Column Bases

- Exceptions to required shear strength:
    - Single-story columns with simple connections at each end need not comply with (b) and (c) from previous slide
    - Columns part of Ordinary systems need not comply with (c)
    - $0.7F_yZ/(\alpha_sH)$ need not exceed foundation load transfer per nonlinear analysis or inelastic behavior resulting from 0.025H story drift at *either* first or second story

Seismic Provisions Chapter D – General Member and Connection Design Requirements

D2.6 Column Bases

- Required flexural strength of SFRS columns with fixed bases shall be summation of required connection strength of steel elements connected to column base:
  - Diagonal braces: required flexural strength of column base
  - Columns:
    - $1.1R_yF_yZ/\alpha_s$
    - Based on overstrength seismic load, provided ductile limit state in either column base or foundation controls the design

Example 4.5.3

SMF Column Base Design

- Partial example emphasizes determination of required base plate strength (SDM pg. 4-204)
- The example demonstrates how the *Seismic Provisions* column base requirements govern the design vs. code-based required strength
- Example worked in ASD

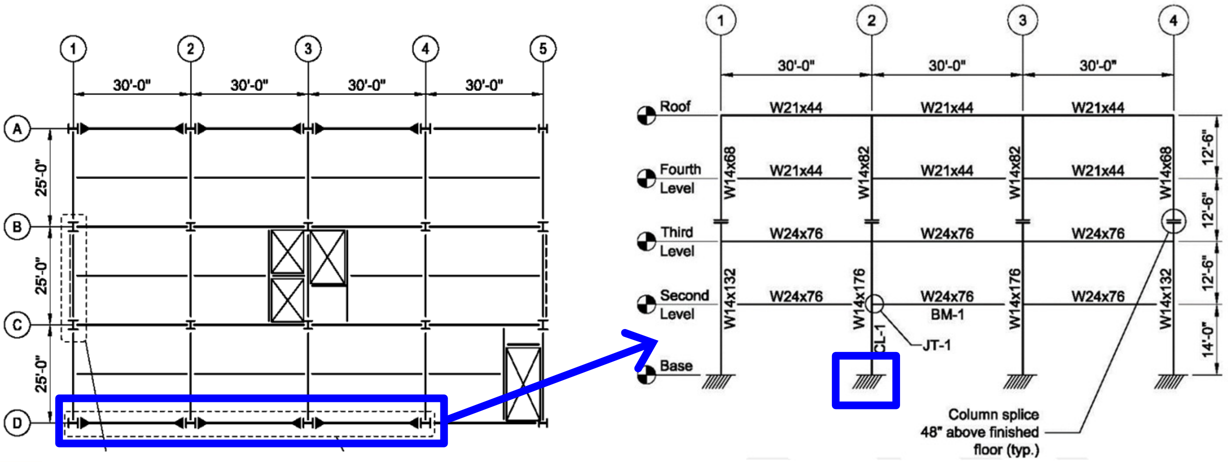
Example 4.5.3 SMF Column Base Design

Refer to Column CL-1 in Figure 4-9. Design a fixed column base plate for the ASTM A992 W-shape.

For this example:

- Use ASCE 7 for calculation of loads.
- The required strengths are determined by a second-order analysis including the effects of P-δ and P-Δ with reduced stiffness as required by the direct analysis method.

Example 4.5.3 SMF Column Base Design



Example 4.5.3 SMF Column Base Design

In this example, two of the controlling limit states are tensile yielding in the anchor rods and bending in the base plate.

For these limit states, the axial force needs to be minimized because this will increase the overturning (bending) in the base plate and increase the tensile force in the anchor rods; therefore, the required axial compressive strength is determined from:

Load Combination 10 from ASCE 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(0.6 - 0.14 S_{DS}\right) D + 0.7 \Omega_o Q_E \\ &= 64.5 \text{ kips} \end{aligned}$$

Example 4.5.3 SMF Column Base Design

The required **flexural strength** is determined from:

Load Combination 10 from ASCE 7, Section 2.4.5:

$$\begin{aligned} M_a &= \left(0.6 - 0.14 S_{DS}\right) M_D + 0.7 \Omega_o M_{Q_E} \\ &= 662 \text{ kip-ft} \end{aligned}$$

Note: this moment will be checked against requirements of D2.6c(b)

The required **shear strength** is determined from:

$$\begin{aligned} V_a &= \left(1.0 + 0.14 S_{DS}\right) D + 0.7 \Omega_o Q_E \\ &= 67.2 \text{ kips} \end{aligned}$$

Note: this shear will be checked against requirements of D2.6b(b)

Assume that the connection into the column minor axis produces negligible moments on the column.

Example 4.5.3 SMF Column Base Design

From ASCE 7, use Seismic Design Category D, $\Omega_o = 3$, $\rho = 1.0$, and $S_{DS} = 1.0$.

Solution:

From AISC *Manual* Table 2-4, the column material properties are as follows:

ASTM A992
$F_y = 50$ ksi
$F_u = 65$ ksi

Example 4.5.3 SMF Column Base Design

From AISC *Manual* Table 1-1, the column and beam geometric properties are as follows:

W14×176

$A = 51.8$ in.^{2} $d = 15.2$ in. $t_w = 0.830$ in. $b_f = 15.7$ in.
$t_f = 1.31$ in. $k_{des} = 1.91$ in. $Z_x = 320$ in.^{3}

W24×76
$d = 23.9$ in.

Example 4.5.3 SMF Column Base Design

Required Strengths at Column Base
AISC *Seismic Provisions* Section D2.6a(a) defines the required axial strength at the column base.

AISC *Seismic Provisions* Section D2.6b(b) defines the required shear strength of the column base as the lesser of the required shear strength determined from load combinations, including the overstrength seismic load or $2R_yF_yZ/(\alpha_sH)$, but not less than $0.7F_yZ/(\alpha_sH)$.

Example 4.5.3 SMF Column Base Design

$$\begin{aligned} V_a &= \frac{2R_yF_yZ}{\alpha_sH} \\ &= \frac{2(1.1)(50\text{ ksi})(320\text{ in.}^3)}{1.5(14\text{ ft})(12\text{ in./ft})} \\ &= 140\text{ kips} > 67.2\text{ kips} \qquad \text{Use 67.2 kips} \end{aligned}$$

$$\begin{aligned} V_a &> \frac{0.7F_yZ}{\alpha_sH} \\ &= \frac{0.7(50\text{ ksi})(320\text{ in.}^3)}{1.5(14\text{ ft})(12\text{ in./ft})} \\ &= 44.4\text{ kips} < 67.2\text{ kips} \qquad \text{Use 67.2 kips} \end{aligned}$$

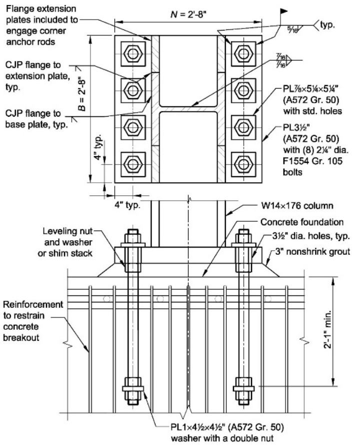
Therefore, V_a = 67.2 kips.

Example 4.5.3 SMF Column Base Design

AISC *Seismic Provisions* Section D2.6c(b) requires that the flexural strength equal or exceed the lesser of the load combination of the applicable building code, including the overstrength seismic load, or the following:

$$\begin{aligned} M_a &= \frac{1.1 R_y F_y Z_x}{\alpha_s} \\ &= \frac{1.1(1.1)(50\text{ ksi})(320\text{ in.}^3)}{1.5(12\text{ in./ft})} \\ &= 1{,}080\text{ kip-ft} > 662\text{ kip-ft} \end{aligned}$$

Therefore, M_a = 662 kip-ft.



Example 4.5.3
SMF Column Base Design

Third Edition of AISC Seismic Design Manual

Applications of the 2016 *Seismic Provisions* – AISC 341

Smarter.
Stronger.
Steel.

SDM Part 9: *Seismic Provisions* Chapter E – Moment Frames

In this section of the seminar, we cover:

- *Seismic Provisions* Chapter E: Moment Frames
  - Example 4.3.6 SMF Beam-Column Connection Design - SCWB
  - Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions
  - Example 4.3.6 SMF Beam-Column Connection Design – Panel Zone
- *Seismic Provisions* Chapter E: Ordinary and Intermediate Moment Frames (comparison with SMF requirements)

Seismic Provisions Chapter E – Moment Frames

E3.2 Basis of Design

- In *Seismic Provisions* Chapter E, the systems with the “least amount” of ductile detailing are presented first, while those requiring “more” ductile detailing are presented later (excluding STMF and SCCS)
- This presentation will reverse the order and present the SMF first and then contrast these requirements with those for OMF and IMF
- *Seismic Provisions* Section E3 addresses SMF

Seismic Provisions Chapter E – Moment Frames

E3.2 Basis of Design

- Special Moment Frames (SMF) are expected to provide significant inelastic deformation through:
    - Flexural yielding of beams (i.e., plastic hinging)
    - Limited yielding of column panel zones
- Columns generally expected to be stronger than fully yielded and strain-hardened beams
    - But flexural yielding of column bases permitted (and expected if base is fixed)

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

- Following relationship to be satisfied in beam-to-column connections

A capacity-design check: beam is demand and column is resistance

$$\frac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0 \qquad \text{(Eq. E3-1)}$$

- "Strong Column – Weak Beam" provision is intended to reduce likelihood global frame instability rather than prevent yielding of individual columns
- Similar to "proportioning" approach behind capacity-design provisions (e.g., overstrength)
- Delaying column yielding helps force beam yielding into multiple levels and provides greater overall frame stability

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

If centerlines of opposing beams do not align, use mid-line between centerlines

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

where

ΣM^*_{pc} = sum of projections of nominal flexural strength of columns above and below joint to beam centerline with reduction for axial force. May be taken as:

$$\Sigma M^*_{pc} = \Sigma Z_c (F_{yc} - \alpha_s P_r / A_g)$$

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

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

where

ΣM^*_{pb} = sum of projections of expected flexural strength of beams at plastic hinge locations to column centerline. May be taken as:

$$\Sigma M^*_{pb} = \Sigma(M_{pr} + \alpha_s M_v)$$

For example, as determined using AISC 358 requirements

Additional moment due to amplification of shear from load combinations

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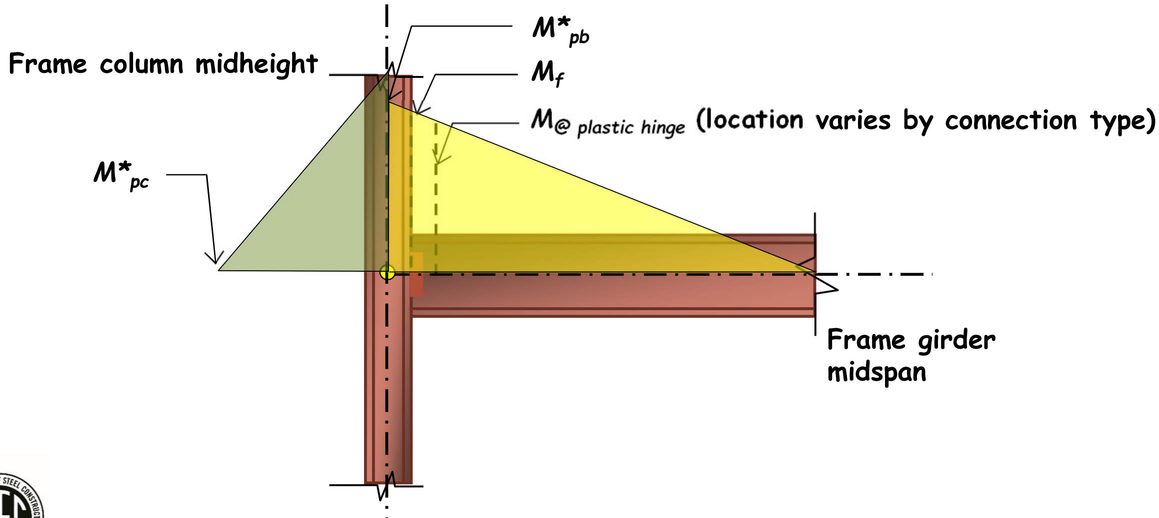
Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio



Partial Frame Moment Diagram

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

- Exceptions: Eq. E3-1 need not apply if either (a) or (b) is true:
  - (a) Columns aren't too heavily loaded and (i) they are located at the roof or in a one-story building or (ii) there aren't too many columns that don't satisfy Eq. E3-1
  - (b) Columns are sufficiently strong compared to the columns on the floor above (i.e., columns in any story that have a ratio of available shear strength to required shear strength that is 50% greater than the story above).

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

- The *Seismic Provisions* don't use the term "not too heavily loaded", so what does it mean?
  - Columns with $P_{rc} < 0.3P_c$
  - where $P_c = F_{yc}A_g/\alpha_s$

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio

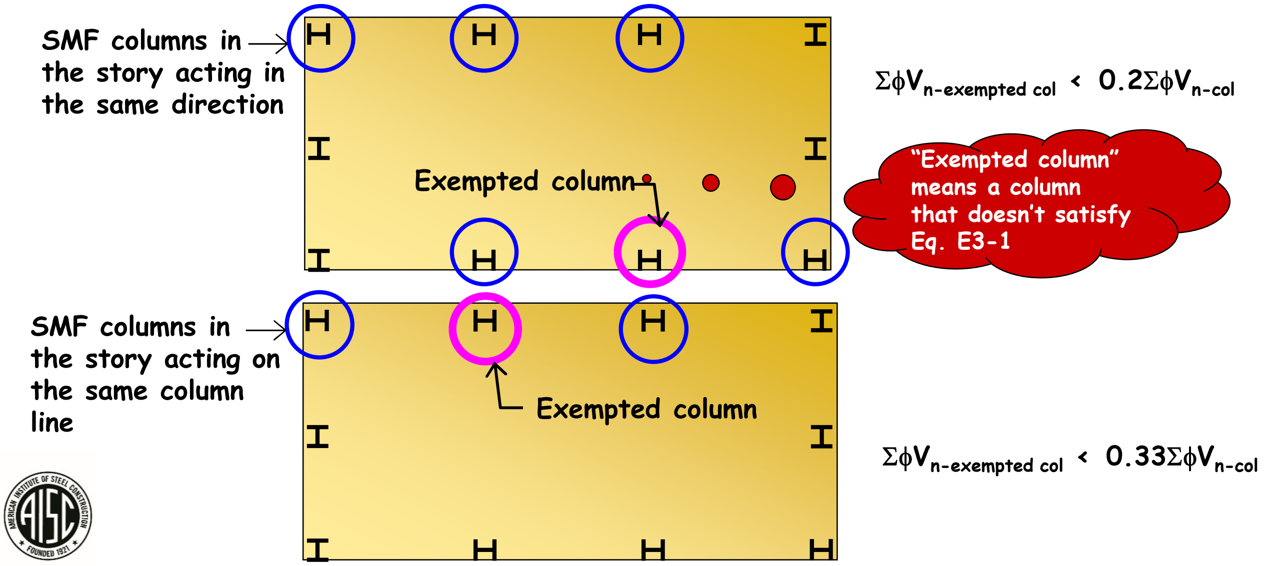
- The *Seismic Provisions* don’t use the term “not too many”, so what does it mean?
    - Columns where sum of available shear strengths of all exempted columns in the story is less than 20% of sum of available shear strength of all moment frame columns in the story acting in the same direction

…and…

    - Columns where sum of available shear strengths of all exempted columns on each moment frame column line within that story is less than 33% of available shear strength of all moment frame columns on that column line.

Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements: Moment Ratio



SMF columns in the story acting in the same direction

Exempted column

$\Sigma\phi V_{n\text{-exempted col}} < 0.2\Sigma\phi V_{n\text{-col}}$

“Exempted column” means a column that doesn’t satisfy Eq. E3-1

SMF columns in the story acting on the same column line

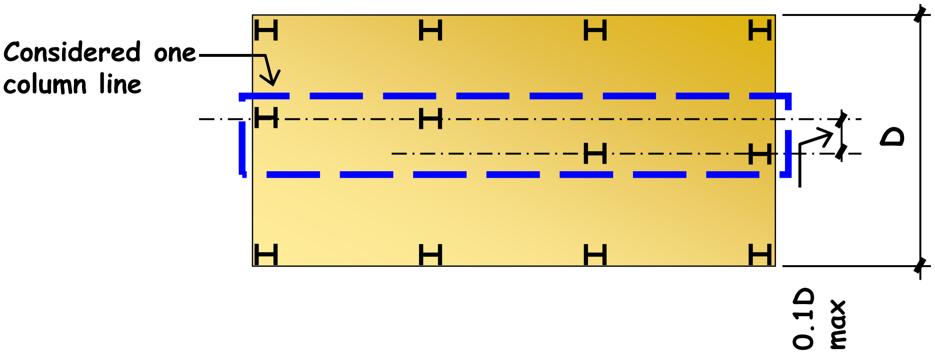
Exempted column

$\Sigma\phi V_{n\text{-exempted col}} < 0.33\Sigma\phi V_{n\text{-col}}$

Seismic Provisions E – Moment Frames

E3.4 System Requirements: Moment Ratio

- Column line
  - For the purposes of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns



Seismic Provisions Chapter E – Moment Frames

E3.4 System Requirements

- Stability Bracing
  - SMF beams: Brace along their length as for highly ductile member (see Section D1.2b)
  - SMF plastic hinges (see Section D1.2c)
  - SMF column flange required bracing strength: $0.02F_y b_f t_{bf}/\alpha_s$

Seismic Provisions Chapter E – Moment Frames

E3.5 Members

- Beam and column proportions shall satisfy “highly ductile” requirements (Section D1.1)
- Protected Zones
    - Definition: Region at each end of beam subject to inelastic straining (see Section D1.3)
    - Location of protected zone is as designated by AISC 358 or other testing (generally, from face of column to one-half of beam depth beyond plastic hinge point)

Example 4.3.6

SMF Beam-Column Connection Design

- Partial example emphasizing “strong column-weak beam” check starting at Step 11 on SDM pg. 4-110
- Example worked in LRFD

Example 4.3.6 SMF Beam-Column Connection Design – RBS

RBS Design Procedure Per ANSI/AISC 358

- Step 1. Choose trial values for the RBS dimensions *a*, *b* and *c*. See Example 4.3.3.

⋮

- Step 9. Design the beam web-to-column connection per AISC 358 Section 5.6
- Step 10. Check continuity plate requirements pre AISC Section 358 Chapter 2 (same as *Seismic Provisions* Section E3.6f)
- **Step 11. Check column-beam relationship limitations according to AISC 358 Section 5.4 (same as *Seismic Provisions* Section E3.4a)**
- Check the column panel zone according to Section AISC 358 Section 5.4 (same as *Seismic Provisions* Section E3.6c)

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Step 11. Check Column-Beam Relationship per ANSI/AISC 358, Section 5.4

AISC *Seismic Provisions* Section E3.4a requires that SMF connections satisfy the following strong-column weak-beam criterion, assuming that the exceptions stated in Section E3.4a are not met.

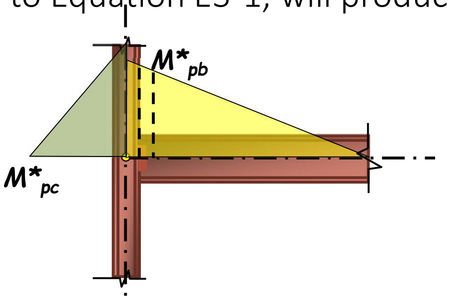
$$\frac{\Sigma M_{pc}^{*}}{\Sigma M_{pb}^{*}} > 1.0$$

(*Prov.* Eq. E3-1)

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The value of M^*_{pc} in this example is based on projecting M_{pc} to the beam centerline, assuming that the column shear, V_c, is in equilibrium with the column moment, M_{pc}. This is consistent with the definition of M^*_{pc} in AISC *Seismic Provisions* Section E3.4a.

Alternatively, the column shear could be computed to be in equilibrium with the beam moment, M_{pr}. The latter approach will result in a smaller value of M^*_{pc} and, when applied to Equation E3-1, will produce a slightly more conservative result.



Example 4.3.6 SMF Beam-Column Connection Design – RBS

The axial load on the column must also be considered when determining the flexural strength of the column at the beam centerline. (For simplicity, the same axial load will be used above and below the joint, although this is not quite accurate.)

Using P_{uc} = 249 kips as given in Example 4.3.2, and the height of the column to its assumed points of inflection above [h_t = (12.5 ft/2)(12 in./ft) = 75.0 in.] and below [h_b = (14 ft/2)(12 in./ft) = 84.0 in.] the beam centerline, ΣM^*_{pc} is determined as follows:

Example 4.3.6 SMF Beam-Column Connection Design – RBS

$$\sum M_{pc}^{*} = \sum Z_c \left(F_{yc} - \frac{\alpha_s P_r}{A_g} \right) \qquad (\textit{Prov.}\ \text{Eq. E3-2})$$

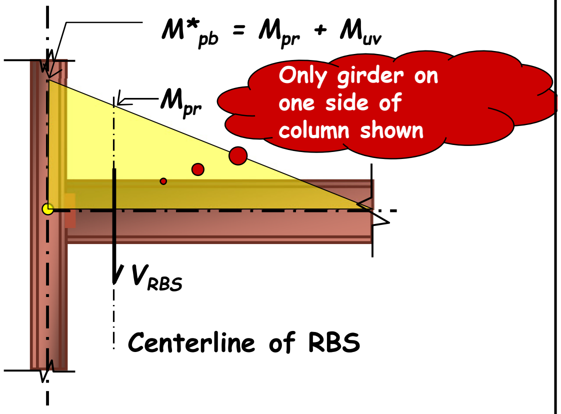
$$= Z_{xt}\left(F_y - \frac{\alpha_s P_{uc}}{A_g}\right)\left(\frac{h_t}{h_t - d_b/2}\right) + Z_{xb}\left(F_y - \frac{\alpha_s P_{uc}}{A_g}\right)\left(\frac{h_b}{h_b - d_b/2}\right)$$

$$= \left(320 \text{ in.}^3\right)\left[50 \text{ ksi} - \frac{1.0(249 \text{ kips})}{51.8 \text{ in.}^2}\right]\left(\frac{75.0 \text{ in.}}{75.0 \text{ in.} - 23.9 \text{ in.}/2} + \frac{84.0 \text{ in.}}{84.0 \text{ in.} - 23.9 \text{ in.}/2}\right)$$

$$= 34{,}100 \text{ kip-in.}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The expected flexural demand of the beam at the column centerline is defined in ANSI/AISC 358, Section 5.4, as:



$$\sum M_{pb}^{*} = \sum \left(M_{pr} + M_{uv}\right)$$
$$= \sum M_{pr} + \sum M_{uv}$$

where

$\sum M_{pr}$ = summation of the probable maximum moment at the center of each RBS determined previously

$$\sum M_{uv} = \sum \left[V_{RBS} \left(a + \frac{b}{2} + \frac{d_c}{2} \right) \right]$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The term ΣM_{uv} is the sum of the moments produced at the column centerline by the shear at the plastic hinges. Recalling the values of V_{RBS} and V'_{RBS} computed in Step 4 of this example and the values of the RBS cut confirmed in Step 1, ΣM_{uv} is:

$$\begin{aligned}\sum M_{uv} &= \left(V_{RBS} + V'_{RBS}\right)\left(a + \frac{b}{2} + \frac{d_c}{2}\right)\\ &= \left(72.1\text{ kips} + 37.6\text{ kips}\right)\left(52\text{ in.} + \frac{18\text{ in.}}{2} + \frac{15.2\text{ in.}}{2}\right)\\ &= 2{,}420\text{ kip-in.}\end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Therefore, the expected flexural demand of the beam at the column centerline is:

$$\begin{aligned}\sum M^*_{pb} &= 2M_{pr} + \sum M_{uv}\\ &= 2\left(8{,}670\text{ kip-in.}\right) + 2{,}420\text{ kip-in.}\\ &= 19{,}800\text{ kip-in.}\end{aligned}$$

$$\begin{aligned}\frac{\sum M^*_{pc}}{\sum M^*_{pb}} &= \frac{34{,}100\text{ kip-in.}}{19{,}800\text{ kip-in.}}\\ &= 1.72 > 1.0 \quad \textbf{o.k.}\end{aligned}$$

Therefore, the strong-column weak-beam check is satisfied.

Example 4.3.6
SMF Beam-Column Connection Design

Example 4.3.8
SMF Strong-Column Weak Beam Exceptions

- Example to illustrate to how to determine if columns may be exempted from SCWB requirements for SMF (SDM pg. 4-138)
- Example worked in LRFD

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

AISC *Seismic Provisions* Section E3.4a includes the following three exceptions when AISC *Seismic Provisions* Equation E3-1 (referred to as the strong-column weak-beam requirement) need not be applied.

1. ~~Columns with low axial loads ($P_{rc} < 0.3P_c$) used in one-story buildings or in the top story of a multi-story building [Section E3.4a(a)(1)]~~
2. (i) Columns with low axial loads ($P_{rc} < 0.3P_c$) in which the available shear strength of the exempted columns represents a relatively small portion of the available shear strength of the story and the moment frame column line [Section E3.4a(a)(2)]…and…

   (ii)Columns in levels that are significantly stronger than the level above (as computed relative to their respective required shear strengths) [Section E3.4a(b)]

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

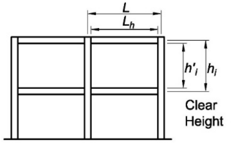
As part of the exception, it is necessary to calculate the available shear strength of the exempted moment frame columns and the non-exempted moment frame columns.

There are several approaches that may be used to calculate these quantities. The User Note in AISC *Seismic Provisions* Section E3.4a provides guidance on two options, and these options, along with a third, are as follows:

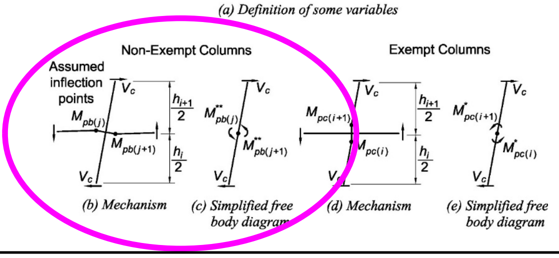
Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

A. The User Note states that the available shear strengths of the columns can be calculated considering the **flexure at each end of the column as limited by the flexural strength of the attached beams**. Columns that **satisfy the strong-column weak-beam requirement** [see Figure 4-30(b) and Figure 4-30(c)] would have a shear strength, V_c, equal to:

$$V_c = \frac{\sum_j M^{**}_{pb(j)}}{\sum_i \left(h_i/2\right)}$$



(a) Definition of some variables



(b) Mechanism (c) Simplified free body diagram (d) Mechanism (e) Simplified free body diagram

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Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Where

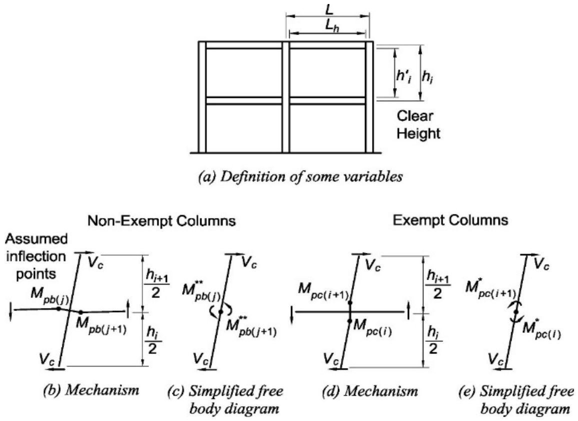
$M^{**}_{pb(j)}$ = projection of the nominal flexural strength of the beam to the centerline of the column as calculated according to AISC *Seismic Provisions* Section E3.4a.

The calculation is made for each beam, *j*, rigidly framing into the joint. To be consistent with the way the column flexural strength is calculated, the beam nominal flexural strength should be used (neglecting the $1.1R_y$ factor).

Similar to the strong-column weak-beam check, the moment capacities of all beams framing into the joint (either one or two) are summed.

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

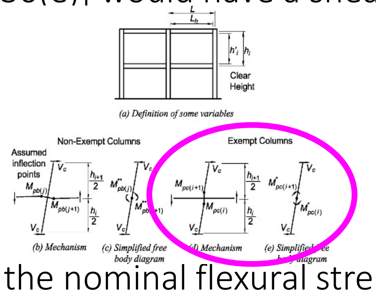
h_i = story height from centerline of beam to centerline of beam. The sum of the distances half way to the adjacent floor lines results in the height between approximate inflection points where the shear, V_c, is assumed to act (see Figure 4-30). If investigating a joint at the roof level, the denominator consists of only one term that is half the height of the top story.



Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Columns that **don't satisfy the strong-column weak-beam requirement** [see Figure 4-30(d) and Figure 4-30(e)] would have a shear strength, V_c, equal to:

$$V_c = \frac{\sum_i M^*_{pc(i)}}{\sum_i \left(h_i/2\right)}$$



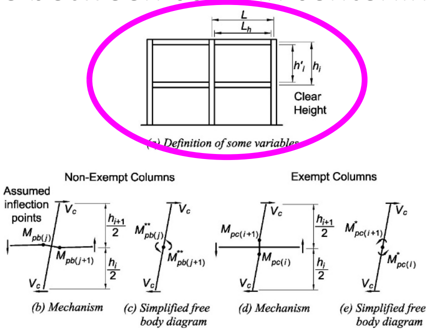
where

$M^*_{pc(i)}$ = projection of the nominal flexural strength of the column to the centerline of the beam as calculated according to AISC *Seismic Provisions* Section E3.4a. The calculation is made for each column, i, which includes two columns if the column extends above the joint, and one column otherwise.

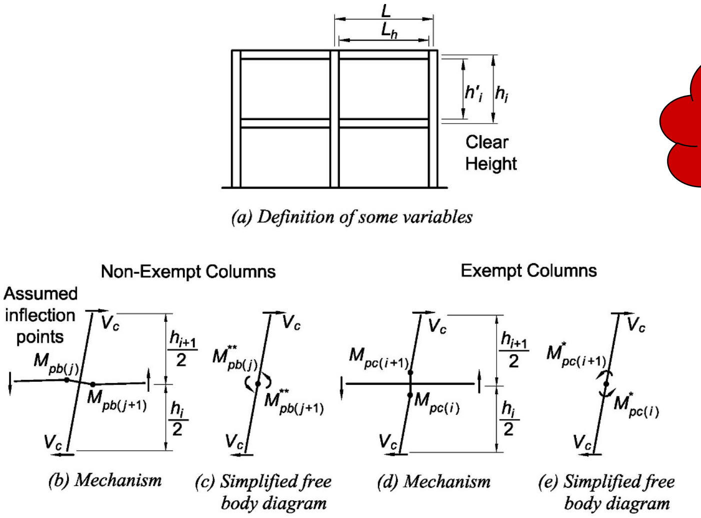
Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Projecting the nominal flexural strength of the beam, M_{pb}, from face of column to centerline of column or the column flexural moment, M_{pc}, from face of beam to beam centerline can be done by multiplying the moments by L/L_h and h_i/h_i', respectively.

The lengths and heights are shown in Figure 4-30(a) as the distance between beam plastic hinges, L_h, the distance between column centerlines, L, and the clear height between beams, h_i'.



Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions



This option also may be used for ASCE 7 weak-story check

Fig. 4-30. Diagram showing calculation of column shear for Option A.

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

B. The User Note in AISC *Seismic Provisions* Section E3.4a states that the available shear strengths of the columns can alternatively be calculated based on the **flexural strength of the columns**. This is similar to the equation presented under Option A for columns not satisfying strong-column weak-beam requirements, but in this case, it is applied to all columns.

Compared to Option A, this method increases the shear strength for the non-exempt columns, thus making the contribution of exempt columns to story shear strength seem smaller than it is. Option A provides a more accurate assessment of story shear strength than this method.

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

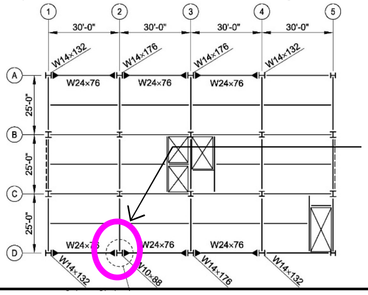
C. A **nonlinear pushover analysis** could be conducted on the individual story to calculate available shear strength of the story and the contribution of the exempt columns to the available shear strength.

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Given:

Refer to floor plan in Figure 4-31. Column CL-1 is a W10 due to architectural reasons. The story height is 14 ft below this floor and 12 ft 6 in. above this floor. The clear height between beams is h_i' = 12 ft and h_i' = 10 ft 6 in. above and below this floor, respectively.

The horizontal distance between plastic hinges is L_h = 26 ft 4 in. Verify that Column CL-1 can be exempt from the strong-column weak-beam requirements.



W10x88 column being investigated

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

The governing load combination for **axial and flexural strength** including seismic effects from ASCE/SEI 7, Section 2.3.6 (for LRFD) and Section 2.4.5 (for ASD), including E_v and E_h as defined in Section 12.4.2, is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on L):

$$\begin{aligned} P_u &= \left(1.2 + 0.2S_{DS}\right)D + \rho Q_E \\ &\quad + 0.5L + 0.2S \\ &= 243 \text{ kips} \end{aligned}$$

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

From AISC *Manual* Table 2-4, the W-shape material properties are as follows:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Column	Beam
W10×88	W24×76
A = 26.0 in.^{2}	Z_x = 200 in.^{3}
Z_x = 113 in.^{3}	

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Solution:

Because Column CL-1 is not part of a one-story building or at the top story of the building, Exception 1 described in the preceding discussion does not apply.

To satisfy Exception 2, the column required axial strength has to be less than 30% of the nominal compressive strength ($P_{rc} < 0.3P_r$), and the shear strength of the exempted column must be a small portion of the story available shear strength as specified in AISC *Seismic Provisions* Section E3.4a(a)(2).

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Start by checking whether the column axial force is less than 30% of the nominal compressive strength. The nominal compressive strength is determined from AISC *Seismic Provisions* Equation E3-5:

$$\begin{aligned} P_c &= F_{yc}A_g/\alpha_s \\ &= (50\text{ ksi})(26.0\text{ in.}^2)/1.0 \\ &= 1{,}300\text{ kips} \end{aligned}$$

Note, the column nominal compressive strength in this application is not determined based on F_{cre}.

$$\begin{aligned} \frac{P_u}{P_c} &= \frac{243\text{ kips}}{1{,}300\text{ kips}} \\ &= 0.187 < 0.3 \quad \textbf{o.k.} \end{aligned}$$

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Next, check AISC *Seismic Provisions* Section E3.4a(a)(2)(i), which states that the sum of the available shear strengths of all exempted columns in the story be less than 20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction.

The first option, Option A, discussed at the beginning of the example, is selected for calculating the available shear strength of the columns based on the flexural strength of the columns as limited by the flexural strength of the attached beams.

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

Because the phi or omega factors will cancel out, the nominal shear strengths are calculated. The nominal shear strength of the exempt column is:

$$V_{exempt} = \frac{\sum_i M^*_{pc(i)}}{\sum_i \left(h_i/2\right)}$$

$$= \frac{F_{yc}Z_x\left(h_1/h_1'\right) + F_{yc}Z_x\left(h_2/h_2'\right)}{h_1/2 + h_2/2}$$

$$= \frac{\left(50\text{ ksi}\right)\left(113\text{ in}^3\right)\left[\left(\frac{14\text{ ft}}{12\text{ ft}}\right) + \left(\frac{12.5\text{ ft}}{10.5\text{ ft}}\right)\right]}{\left(\frac{14\text{ ft}}{2} + \frac{12.5\text{ ft}}{2}\right)\left(12\text{ in./ft}\right)}$$

$$= 83.8\text{ kips}$$

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

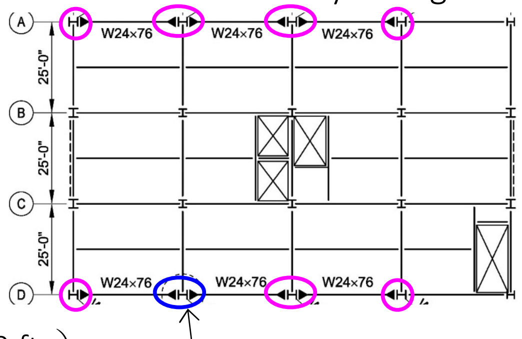
The nominal shear strength of all moment frame columns in the story acting in the same direction is:

$$V_{direction} = \frac{\sum_j M^{**}_{pb(j)}}{\sum_i \left(h_i/2\right)} + V_{exempt}$$

$$= \frac{\left(10\text{ connections}\right)F_{yb}Z_{xb}\left(L/L_h\right)}{\left(h_1/2\right) + \left(h_2/2\right)} + V_{exempt}$$

$$= \frac{\left(10\text{ connections}\right)\left(50\text{ ksi}\right)\left(200\text{ in.}^3\right)\left(\frac{30\text{ ft}}{26.3\text{ ft}}\right)}{\left(\frac{14\text{ ft}}{2} + \frac{12.5\text{ ft}}{2}\right)\left(12\text{ in./ft}\right)} + 83.8\text{ kips}$$

$$= 801\text{ kips}$$



W10x88 column being investigated

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

The ratio of the nominal shear strengths is:

$$\frac{V_{exempt}}{V_{direction}} = \frac{83.8\text{ kips}}{801\text{ kips}}$$

$$= 0.105 < 0.2 \quad \textbf{o.k.}$$

Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

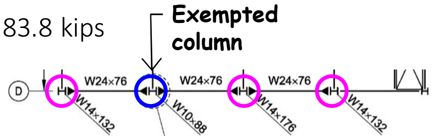
Next, check AISC *Seismic Provisions* Section E3.4a(a)(2)(ii), which states that the sum of the available shear strengths of all exempted columns in a moment frame column line be less than 33% of the sum of the available shear strengths of all moment frame columns in that column line.

The shear strength of all moment frame columns in the moment frame column line is:

$$V_{line} = \frac{\sum_j M^{**}_{pb(j)}}{\sum_i (h_i/2)} + V_{exempt} = \frac{(4\text{ connections})F_{yb}Z_{xb}(L/L_h)}{(h_1/2)+(h_2/2)} + V_{exempt}$$

$$= \frac{(4\text{ connections})(50\text{ ksi})(200\text{ in.}^3)\left(\dfrac{30\text{ ft}}{26.3\text{ ft}}\right)}{\left(\dfrac{14\text{ ft}}{2}+\dfrac{12.5\text{ ft}}{2}\right)(12\text{ in./ft})} + 83.8\text{ kips}$$

$$= 371\text{ kips}$$



Example 4.3.8 SMF Strong-Column Weak-Beam Exceptions

The ratio of the available shear strengths is:

$$\frac{V_{exempt}}{V_{line}} = \frac{83.8\text{ kips}}{371\text{ kips}}$$
$$= 0.226 < 0.33 \quad \textbf{o.k.}$$

Because the conditions of AISC *Seismic Provisions* Section E3.4a(a)(2) are satisfied, the W10×88 column is exempted from the strong-column weak-beam requirements of AISC *Seismic Provisions* Section E3.4a.

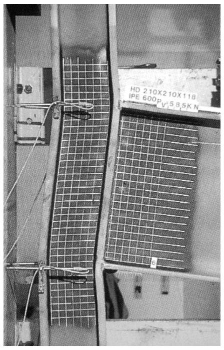
Example 4.3.8

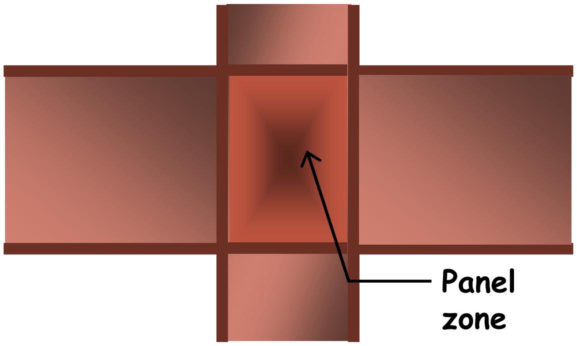
SMF Strong-Column Weak Beam Exceptions

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Panel zone must be strong enough to resist demand from connecting beam without excessive deformation; if it is not, reinforce the panel zone (e.g., using a doubler plate or increase the column size)
- Yielding of panel zone recognized as an efficient method of providing ductility

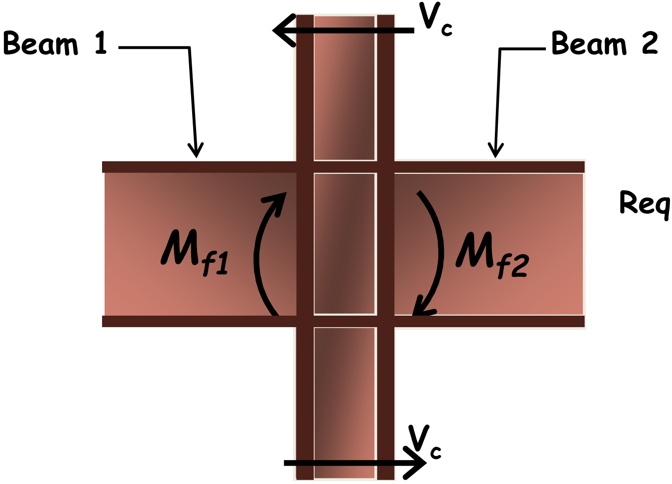




Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Required shear strength based on summation of moments at column face from expected moments at plastic hinge.



Required Panel Zone Shear Strength

$$R_u = \frac{\sum M_f}{(d_b - t_f)} - V_c$$

What is V_c?

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- What is V_c?
  - It is not the code-based column shear from the elastic analysis of the frame.
  - It is the shear produced in the column by the flexure in the frame beams assuming that the beams have yielded

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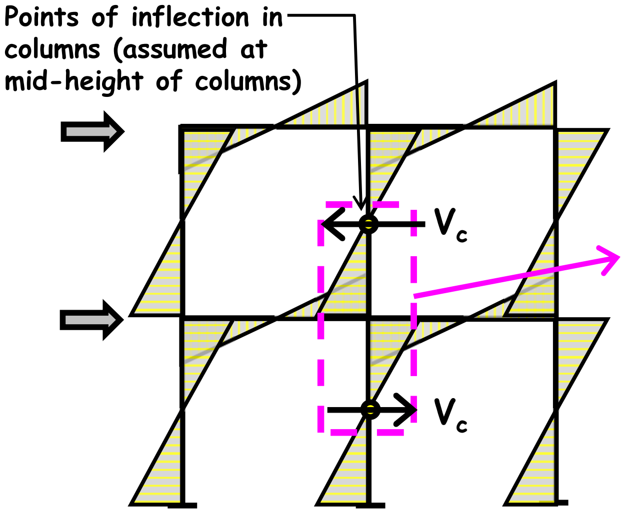
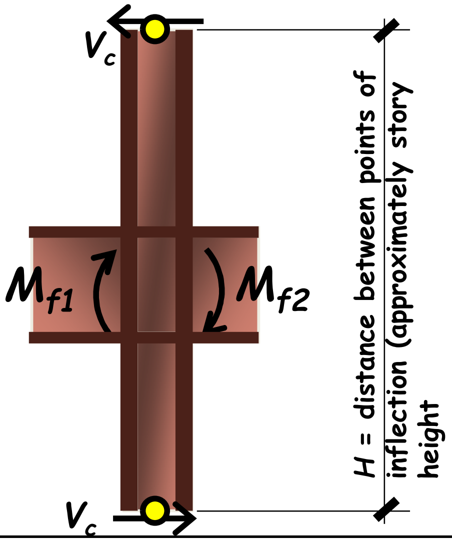
Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Approximate analysis of panel zone shear, V_c

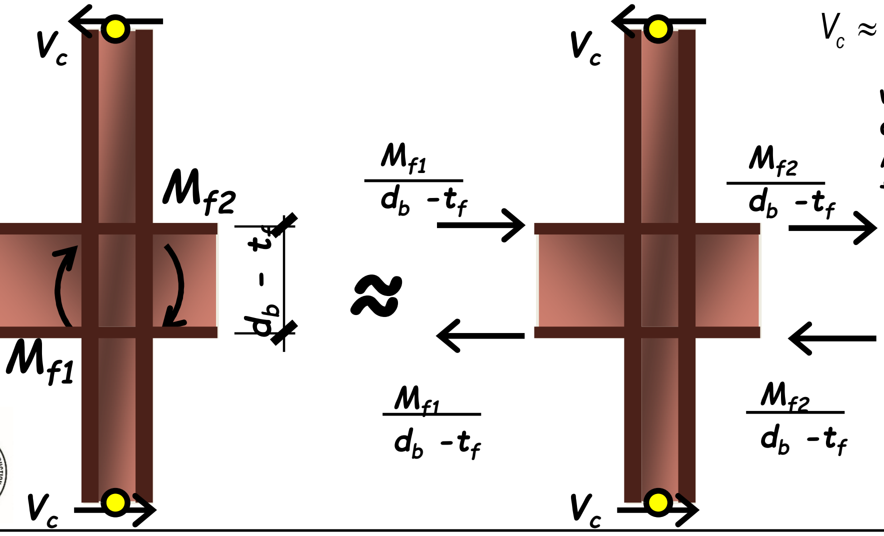




Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Approximate analysis of panel zone shear, V_c



$$V_c \approx \frac{M_{f1} + M_{f2}}{H} = \frac{\sum M_f}{H}$$

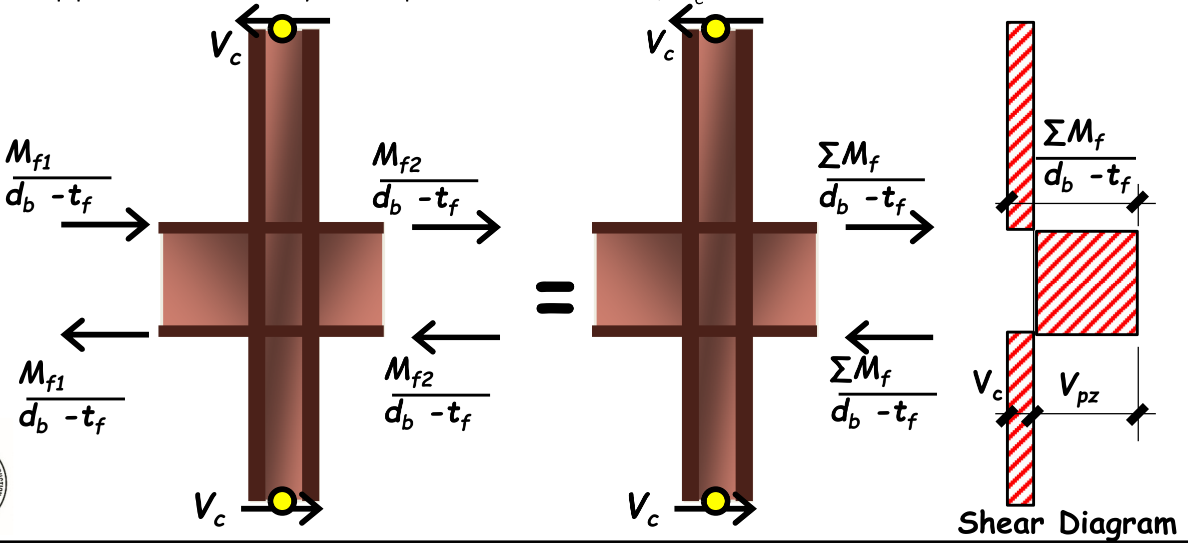
where M_f can be estimated as $1.1R_yM_p$ of the beam (or M_{pr} amplified to the face of the column if an RBS)

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Approximate analysis of panel zone shear, V_c

$$V_{pz} = \frac{\sum M_f}{d_b - t_f} - V_c \cong \frac{\sum M_f}{d_b - t_f} - \frac{\sum M_f}{H}$$



Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Available shear strength is determined from *Specification* Section J10.6 using $\phi = 1.00$
- Section J10.6 lists two equations to establish available shear strength, depending upon whether the effect of panel zone deterioration on frame stability is considered. But, the implications behind the choice of equations reflects different assumptions about the frame behavior

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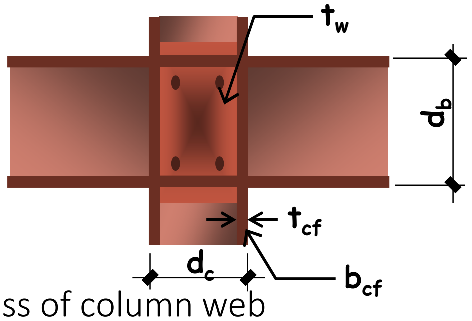
Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- When impact of panel zone deformation on frame stability **is not** considered, shear strength of panel zone is:



For $P_u \le 0.4P_y$

$$R_n = 0.6F_y d_c t_w$$ (AISC Spec EQ J10-9)

Where: d_c = column depth

F_y = specified minimum yield stress of column web

t_w = thickness of column web including doubler plate

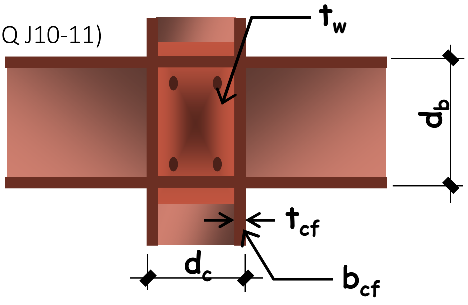
Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- When impact of panel zone deformation on frame stability is considered, shear strength of panel zone is:

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

(AISC Spec EQ J10-11)



Where:
d_c = column depth
d_b = beam depth
b_{cf} = column flange width
t_{cf} = column flange thickness
F_y = minimum specified yield stress of column web
t_w = thickness of column web including doubler plate

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- What are the differences between the equations?

$$R_n = 0.6F_y d_c t_w$$

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

This "second term" accounts for additional shear strength in columns with thicker flanges and inelastic panel zone deformation (at least three times the deformation at panel zone yielding)

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- What are the differences between the equations?
  - When centerline modeling is used in combination with no rigid offsets, effect of panel zone deformation is reasonably considered (based on SAC research)
  - Excessively flexible panel zones will introduce local bending into beam flange welds at column face and can lead to weld fracture.
  - Contribution of "second term" is relatively less in thinner flanged members and in deeper column sections due to the deeper web
- Practically speaking, Eq. J10-11 is the most frequently used equation.

Panel zone flexibility important in wind and stability analyses. Cannot simply neglect flange contribution to justify ignoring PZ flexibility.

Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone Stability

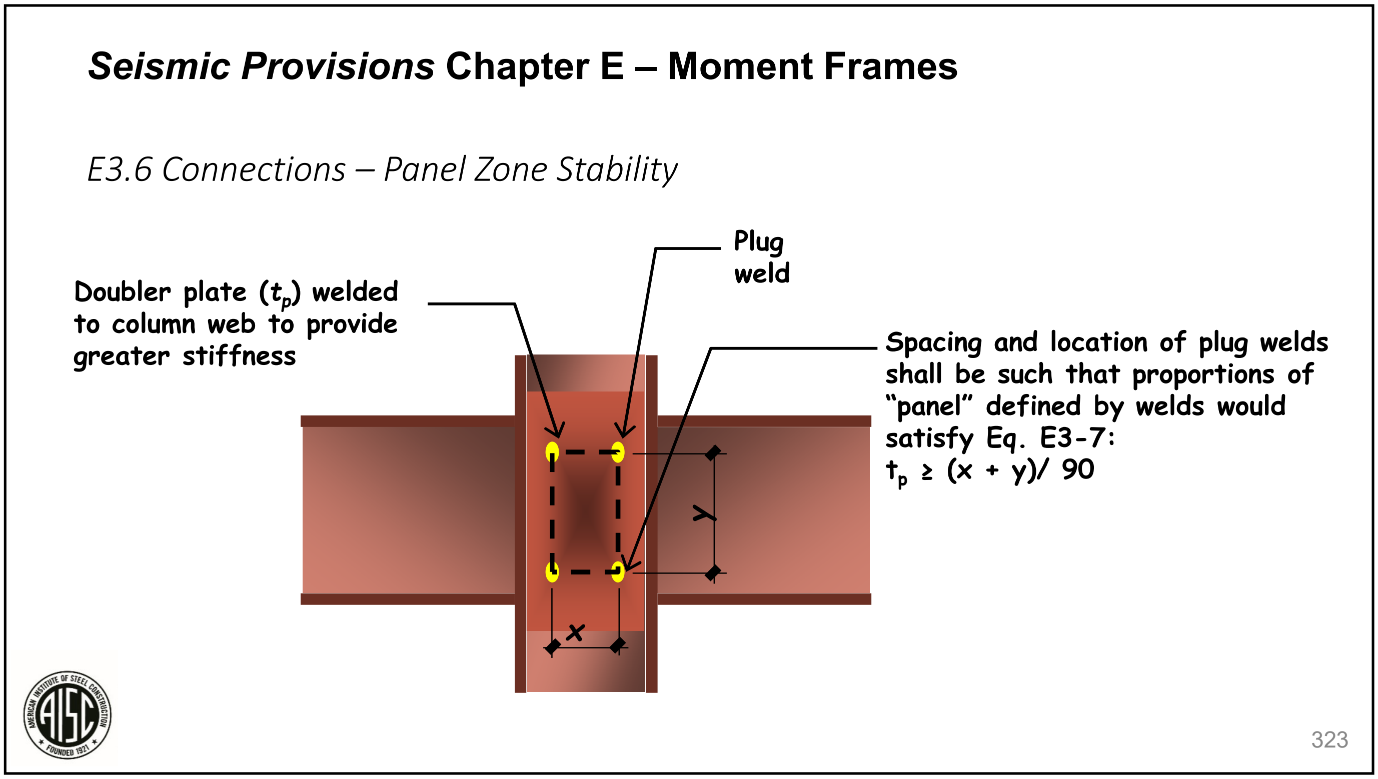
- Column web (and doubler plate, if used) shall satisfy *individually*:

$t \geq (d_z + w_z)/ 90$ (Eq. E3-7)

where:

t = thickness of column web or doubler plate (in.)
d_z = Panel zone depth between continuity plates: $(d - 2t_f)$ (in.)
w_z = Panel zone width between column flanges (in.)

- … or …when local buckling of column web and doubler plate is prevented by plug welds, *total panel zone thickness* shall satisfy Eq. E3-7



Seismic Provisions Chapter E – Moment Frames

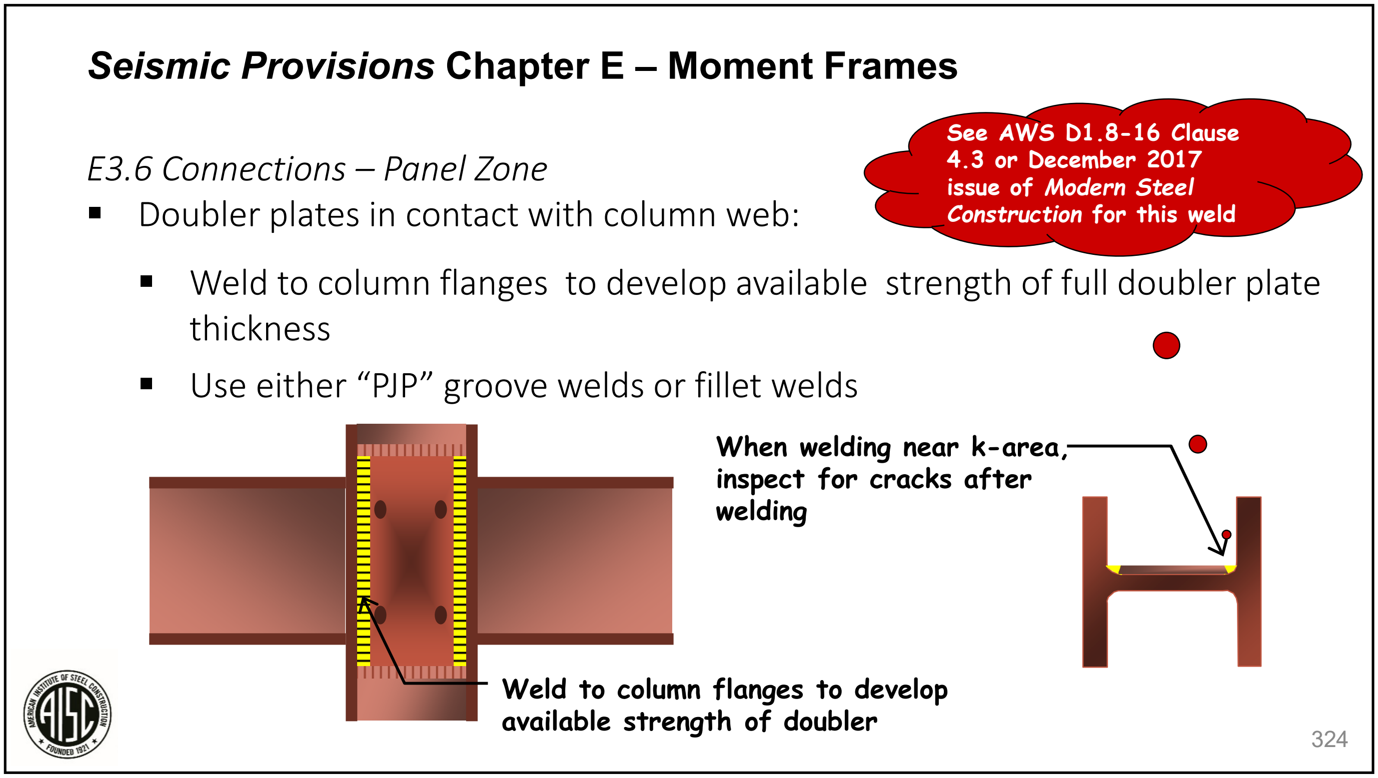
E3.6 Connections – Panel Zone Stability

Doubler plate (t_p) welded to column web to provide greater stiffness

Plug weld

Spacing and location of plug welds shall be such that proportions of "panel" defined by welds would satisfy Eq. E3-7:

$t_p \geq (x + y)/90$



Seismic Provisions Chapter E – Moment Frames

E3.6 Connections – Panel Zone

- Doubler plates in contact with column web:
  - Weld to column flanges to develop available strength of full doubler plate thickness
  - Use either "PJP" groove welds or fillet welds

See AWS D1.8-16 Clause 4.3 or December 2017 issue of *Modern Steel Construction* for this weld

When welding near k-area, inspect for cracks after welding

Weld to column flanges to develop available strength of doubler

Example 4.3.6
SMF Beam-Column Connection Design

- Partial example to illustrate to how to apply column panel zone requirements for SMF starting on SDM page 4-112.
- Example worked in LRFD

Example 4.3.6 SMF Beam-Column Connection Design – RBS

RBS Design Procedure Per ANSI/AISC 358

- Step 1. Choose trial values for the RBS dimensions *a*, *b* and *c*. See Example 4.3.3.

⋮

- Step 9. Design the beam web-to-column connection per AISC 358 Section 5.6
- Step 10. Check continuity plate requirements pre AISC Section 358 Chapter 2 (same as *Seismic Provisions* Section E3.6f)
- Step 11. Check column-beam relationship limitations according to AISC 358 Section 5.4 (same a *Seismic Provisions* Section E3.4a)
- Check the column panel zone according to Section AISC 358 Section 5.4 (same as *Seismic Provisions* Section E3.6c)

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Check Panel Zone

AISC *Seismic Provisions* Section E3.6e.1 specifies that the required shear strength of the panel zone be calculated by summing the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces; in this example, M_f and M_f'.

Thus, the required shear strength of the panel zone can be computed as follows:

$$R_u = \frac{\sum M_f}{d_b - t_{bf}} - V_c$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

In this equation, V_c is the shear force in the column outside of the panel zone. Assuming points of inflection at mid-height of the columns above and below the joint, V_c can be estimated from statics as follows:

$$\begin{aligned} V_c &= \frac{\sum M_{pb}^*}{h_b + h_t} \\ &= \frac{19{,}800 \text{ kip-in.}}{84 \text{ in.} + 75 \text{ in.}} \\ &= 125 \text{ kips} \end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Then:

$$\begin{aligned}R_u &= \frac{\sum M_f}{d_b - t_{bf}} - V_c \\ &= \frac{9{,}720 \text{ kip-in.} + 9{,}220 \text{ kip-in.}}{23.9 \text{ in.} - 0.680 \text{ in.}} - 125 \text{ kips} \\ &= 691 \text{ kips}\end{aligned}$$

According to AISC *Seismic Provisions* Section E3.6e.1, the available shear strength of the panel zone is calculated per AISC *Specification* Section J10.6, but with ϕ_v = 1.00.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

AISC *Specification* Section J10.6 provides different equations for computing the nominal panel-zone shear strength, depending on whether or not the effect of panel-zone deformation on frame stability is included in the analysis.

In this example, analysis of the building frame, including analysis of interstory drift, was based on a centerline model of the frame, without rigid end offsets at the joints. This is considered to satisfy the requirement that the effect of panel-zone deformation on frame stability was included in the analysis.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Therefore, use AISC *Specification* Section J10.6(b) to compute the nominal shear strength of the panel zone.

$$
\begin{aligned}
P_r &= 243 \text{ kips from Example 4.3.2} \\
P_r &< 0.75P_c \\
&< 0.75F_yA_g \\
&< 0.75(50 \text{ ksi})(51.8 \text{ in.}^2) \\
&= 1{,}940 \text{ kips} > 243 \text{ kips} \quad \textbf{o.k.}
\end{aligned}
$$

For $\alpha_s P_r \geq 0.75P_y$, use Eq. J10-12

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Therefore, the shear strength of the panel zone is given by AISC *Specification* Equation J10-11:

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \qquad \text{(\textit{Spec.} Eq. J10-11)}$$

$$\phi R_n = 1.00(0.60)(50 \text{ ksi})(15.2 \text{ in.})(0.830 \text{ in.})\left[1 + \frac{3(15.7 \text{ in.})(1.31 \text{ in.})^2}{(23.9 \text{ in.})(15.2 \text{ in.})(0.830 \text{ in.})}\right]$$

$= 480$ kips < 691 kips. Therefore, a doubler plate is required.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Alternatively, using Table 4-2 of this Manual for the W14×176 column:

$0.75P_y = 1{,}940 \text{ kips}$

$\phi R_{v1} = 378 \text{ kips}$

$\phi R_{v2} = 2{,}420 \text{ kip-in.}$

$$\phi R_n = \phi R_{v1} + \frac{\phi R_{v2}}{d_b}$$

$$= 378 \text{ kips} + \frac{2{,}420 \text{ kip-in.}}{23.9 \text{ in.}}$$

$$= 479 \text{ kips} < 691 \text{ kips} \quad \textbf{n.g.}$$

Therefore, a doubler plate is required.

See SDM pg. 4-229 for explanation

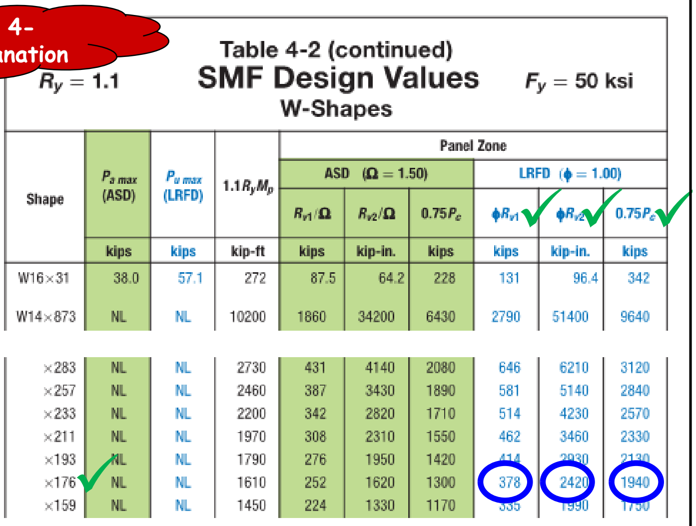


Table 4-2 (continued)
SMF Design Values
W-Shapes

$R_y = 1.1$ $F_y = 50$ ksi

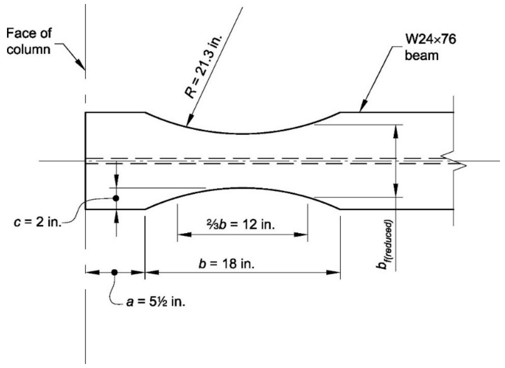
Shape	$P_{a\,max}$ (ASD)	$P_{u\,max}$ (LRFD)	$1.1R_yM_p$	Panel Zone					
				ASD ($\Omega = 1.50$)			LRFD ($\phi = 1.00$)		
				R_{v1}/Ω	R_{v2}/Ω	$0.75P_c$	ϕR_{v1} ✓	ϕR_{v2} ✓	$0.75P_c$ ✓
	kips	kips	kip-ft	kips	kip-in.	kips	kips	kip-in.	kips
W16×31	38.0	57.1	272	87.5	64.2	228	131	96.4	342
W14×873	NL	NL	10200	1860	34200	6430	2790	51400	9640
×283	NL	NL	2730	431	4140	2080	646	6210	3120
×257	NL	NL	2460	387	3430	1890	581	5140	2840
×233	NL	NL	2200	342	2820	1710	514	4230	2570
×211	NL	NL	1970	308	2310	1550	462	3460	2330
×193	NL	NL	1790	276	1950	1420	414	2930	2130
×176 ✓	NL	NL	1610	252	1620	1300	378	2420	1940
×159	NL	NL	1450	224	1330	1170	335	1990	1750

Seismic Design Manual **Table 4-2**

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Reducing the RBS cut (in other words, reducing dimension *c*) will bring M_f closer to $\phi_d M_{pe}$ and reduce the impact of the RBS on frame stiffness.

On the other hand, increasing the RBS cut (in other words, increasing dimension *c*) will reduce the required shear strength of the panel zone and, in some cases, eliminate the need for doubler plates.



Example 4.3.6 SMF Beam-Column Connection Design – RBS

Size web doubler plate

The minimum thickness of each component of the panel zone, without the aid of intermediate plug welds between the column web and the doubler is:

$$t \geq \frac{\left(d_z + w_z\right)}{90}$$ (*Prov.* Eq. E3-7)

From Table 4-2 of this Manual, for the W24×76 beam:

$$\frac{d_z}{90} = 0.250 \text{ in.}$$

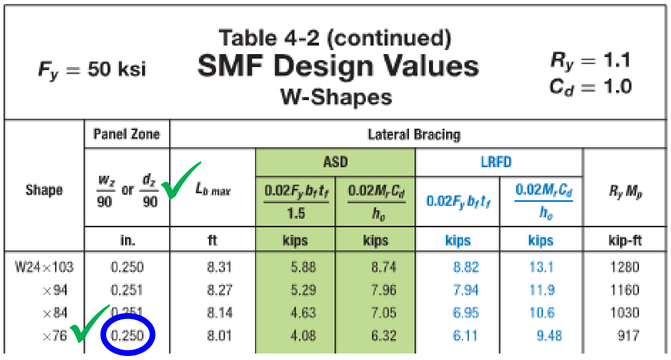


Table 4-2 (continued)
SMF Design Values
W-Shapes

$F_y = 50$ ksi
$R_y = 1.1$
$C_d = 1.0$

Shape	Panel Zone	Lateral Bracing					
			ASD		LRFD		
	$\frac{w_z}{90}$ or $\frac{d_z}{90}$ ✓	$L_{b\,max}$	$\frac{0.02F_y b_f t_f}{1.5}$	$\frac{0.02M_r C_d}{h_o}$	$0.02F_y b_f t_f$	$\frac{0.02M_r C_d}{h_o}$	$R_y M_p$
	in.	ft	kips	kips	kips	kips	kip-ft
W24×103	0.250	8.31	5.88	8.74	8.82	13.1	1280
×94	0.251	8.27	5.29	7.96	7.94	11.9	1160
×84	0.251	8.14	4.63	7.05	6.95	10.6	1030
✓ ×76	0.250	8.01	4.08	6.32	6.11	9.48	917

Seismic Design Manual Table 4-2

Example 4.3.6 SMF Beam-Column Connection Design – RBS

From Table 4-2 of this Manual, for the W14×176 column:

$$\frac{w_z}{90} = 0.140 \text{ in.}$$

$$t \geq 0.250 \text{ in.} + 0.140 \text{ in.}$$

$$= 0.390 \text{ in.}$$

The column web satisfies this requirement:

t_w = 0.830 in. > 0.390 in. o.k.

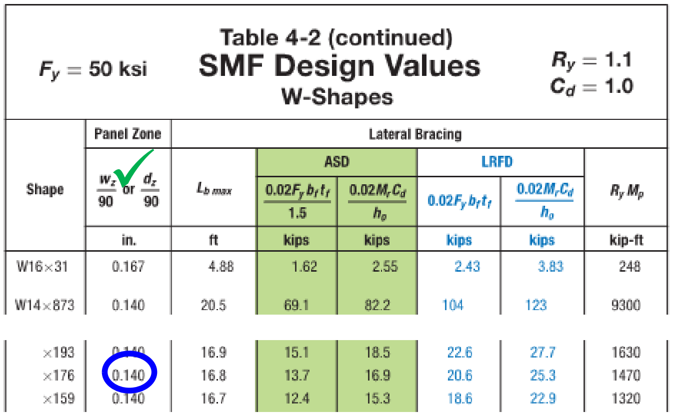


Table 4-2 (continued)
SMF Design Values
W-Shapes

$F_y = 50$ ksi
$R_y = 1.1$
$C_d = 1.0$

Shape	Panel Zone	Lateral Bracing					
			ASD		LRFD		
	$\frac{w_z}{90}$ ✓ or $\frac{d_z}{90}$	$L_{b\,max}$	$\frac{0.02F_y b_f t_f}{1.5}$	$\frac{0.02M_r C_d}{h_o}$	$0.02F_y b_f t_f$	$\frac{0.02M_r C_d}{h_o}$	$R_y M_p$
	in.	ft	kips	kips	kips	kips	kip-ft
W16×31	0.167	4.88	1.62	2.55	2.43	3.83	248
W14×873	0.140	20.5	69.1	82.2	104	123	9300
×193	0.140	16.9	15.1	18.5	22.6	27.7	1630
×176	0.140	16.8	13.7	16.9	20.6	25.3	1470
×159	0.140	16.7	12.4	15.3	18.6	22.9	1320

Seismic Design Manual Table 4-2

Example 4.3.6 SMF Beam-Column Connection Design – RBS

If the doubler plate satisfies this minimum thickness, it is permitted to be applied directly to the column web or spaced away from the web, without the use of plug welds.

The available shear strength of the panel zone is checked using AISC *Specification* Equation J10-11 with the thickness, t_w, taken as the combined thickness of the column web and doubler plate.

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^{\;2}}{d_b d_c t_w}\right) \qquad (\textit{Spec}.\ \text{Eq. J10-11})$$

where t_w used in two places is replaced by $t_w + t_p$.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Rearranging to solve for t_p:

$$t_w + t_p \ge \left[R_u - \frac{0.60F_y\left(3b_{cf} t_{cf}^{\;2}\right)}{d_b}\right]\left[\frac{1}{0.60F_y d_c}\right]$$

$$t_p \ge \left\{691\text{ kips} - \frac{0.60(50\text{ ksi})\left[3(15.7\text{ in.})(1.31\text{ in.})^2\right]}{(23.9\text{ in.})}\right\}$$

$$\times\left[\frac{1}{0.60(50\text{ ksi})(15.2\text{ in.})}\right] - 0.830\text{ in.}$$

$$\ge 0.463\text{ in.}$$

Use an ASTM A572 Grade 50, 1/2-in.-thick doubler plate.

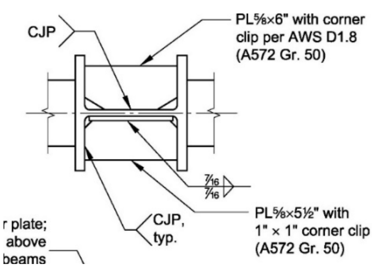
Example 4.3.6 SMF Beam-Column Connection Design – RBS

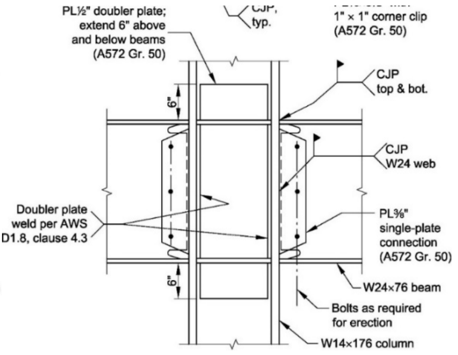
Because the doubler plate meets the minimum thickness required by AISC *Seismic Provisions* Equation E3-7 (0.390 in.), plug welds between the doubler and the column web are not required.

For doubler plates in contact with the web, AISC *Seismic Provisions* Section E3.6e.3 permits doubler plates to be extended above and below the beam or, alternatively, permits the doubler plate to be fit between the continuity plates. Both alternatives will be illustrated in this example.

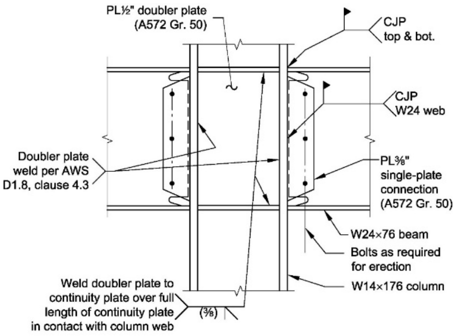
Example 4.3.6 SMF Beam-Column Connection Design – RBS

Figure 4-24 shows the final configuration of the panel zone using the two alternatives presented in the following.





Alt. 1



Alt. 2

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Alternative 1—Extended Doubler Plate
According to the requirements of AISC *Seismic Provisions* Section E3.6e.3, the doubler plate must be extended at least 6 in. above and below the beam. Because continuity plates are present, no weld is required along the top and bottom edges of the doubler plate.

The vertical edges of the doubler plate will be welded to the column flanges using web doubler plate welds in accordance with AWS D1.8, clause 4. No ultrasonic testing is required for these welds.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

On the side of the column with the 1/2-in.-thick doubler plate, the continuity plate will be welded to the doubler plate. Because this continuity plate is not welded directly to the column web, the clip size is not required to meet AWS D1.8, clause 4.1. Use a 1-in. x 1-in. clip.

AISC *Seismic Provisions* Section E3.6e.3 states that the required strength of the weld between the continuity plate and the doubler plate need not exceed the available shear yield strength of the doubler plate.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The available shear yield strength of the doubler plate is determined from AISC *Specification* Section J4.2:

$$\begin{aligned}\phi_v R_n &= \phi_v 0.60 F_y A_{gv}\\ &= 1.00(0.60)(50\text{ ksi})\left(\frac{1}{2}\text{ in.}\right)\left[15.2\text{ in.} - 2(1.31\text{ in.})\right]\\ &= 189\text{ kips}\end{aligned}$$

The contact length between the continuity plate and the doubler plate is:

$$\begin{aligned}\text{contact length} &= 15.2\text{ in.} - 2(1.31\text{ in.} + 1\text{ in.})\\ &= 10.6\text{ in.}\end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The required leg size of double-sided fillet welds over the contact length is:

"2" reflects use of double-sided fillet weld

$$D = \frac{R_u}{(1.392\text{ kip/in.})l} \quad (\textit{Manual}\text{ Eq. 8-2a})$$

$$= \frac{189\text{ kips}}{2(1.392\text{ kip/in.})(10.6\text{ in.})}$$

$$= 6.40\text{ sixteenths}$$

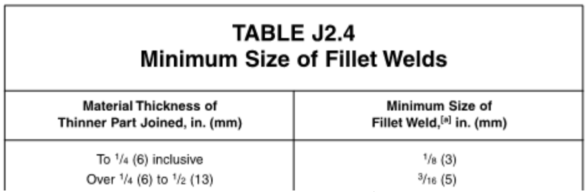


TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To $^1/_4$ (6) inclusive	$^1/_8$ (3)
Over $^1/_4$ (6) to $^1/_2$ (13)	$^3/_{16}$ (5)

***Specifications* Table J2.4**

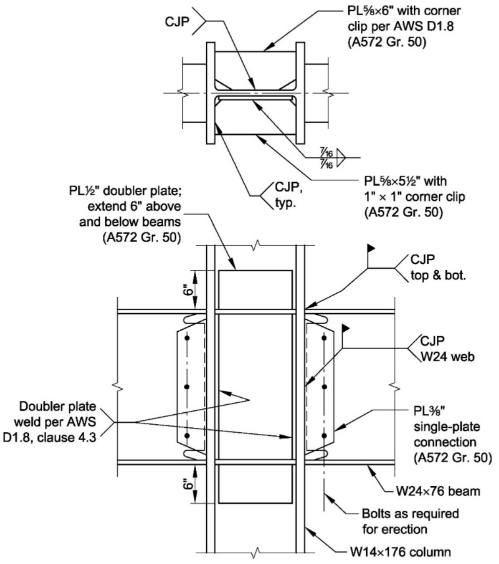
Use 7/16-in. double-sided fillet welds over the full contact length. Note the minimum fillet weld is 3/16 in. from AISC *Specification* Table J2.4.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

As explained in AISC *Seismic Provisions* Commentary Section E3.6e.3, welding a continuity plate to a doubler plate does not substantially change the shear force in the doubler plate or in the doubler plate-to-column connections.

Consequently, no special consideration is needed in the design of the doubler plate or doubler plate-to-column connections when a continuity plate is present.

Example 4.3.6 SMF Beam-Column Connection Design – RBS



Note: For weld backing requirements, and treatment of weld tabs see ANSI/AISC 358, Chapter 3.

Alternative 1 - Extended Doubler Plate

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Alternative 2—Fitted Doubler Plate

For this alternative, both continuity plates will be welded directly to the column web and the doubler plate will be placed between the continuity plates.

The vertical edges of the doubler plate will be welded to the column flanges using web doubler plate welds in accordance with AWS D1.8, clause 4. These welds will begin and end 1 in. from the continuity plates to avoid interference with the continuity plate. No ultrasonic testing is required for these welds.

Example 4.3.6 SMF Beam-Column Connection Design – RBS

AISC *Seismic Provisions* Section E3.6e.3 requires that the top and bottom of the doubler plate be welded to the continuity plate over the full contact length between the continuity plate and the column web. The required strength of this weld is 75% of the available shear yield strength of the doubler plate over the contact length with the continuity plate.

The contact length between the continuity plate and the column web is determined using the clip dimension required by AWS D1.8.

$$\begin{aligned}\text{contact length} &= 15.2\text{ in.} - 2\left(1.31\text{ in.} + 2\frac{7}{8}\text{ in.}\right)\\ &= 6.83\text{ in.}\end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

In accordance with AISC *Seismic Provisions* Section E3.6e.3, the required shear strength of the doubler plate-to-continuity plate weld is:

$$\begin{aligned} R_u &= 0.75\left(\phi_v 0.60F_y\right)\left(\text{contact length}\right)t \\ &= 0.75\left(1.00\right)\left(0.60\right)\left(50\text{ ksi}\right)\left(6.83\text{ in.}\right)\left(\frac{1}{2}\text{ in.}\right) \\ &= 76.8\text{ kips} \end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

The design strength of the PJP groove weld is, from AISC *Specification* Equation J2-3:

$$\begin{aligned} \phi R_n &= \phi F_{nw} A_{we} \\ &= 0.75\left(0.60\right)\left(70\text{ ksi}\right)\left(6.83\text{ in.}\right)t_e \\ &= \left(215\text{ kip/in.}\right)t_e \end{aligned}$$

where t_e is the effective throat.

Solve for t_e:

$$\begin{aligned} t_e &\geq \frac{76.8\text{ kips}}{215\text{ kip/in.}} \\ &\geq 0.357\text{ in.} \end{aligned}$$

Example 4.3.6 SMF Beam-Column Connection Design – RBS

Or, determine the required fillet weld size.

$$D = \frac{R_u}{(1.392\text{ kip/in.})l} \qquad (\textit{Manual}\text{ Eq. 8-2a})$$

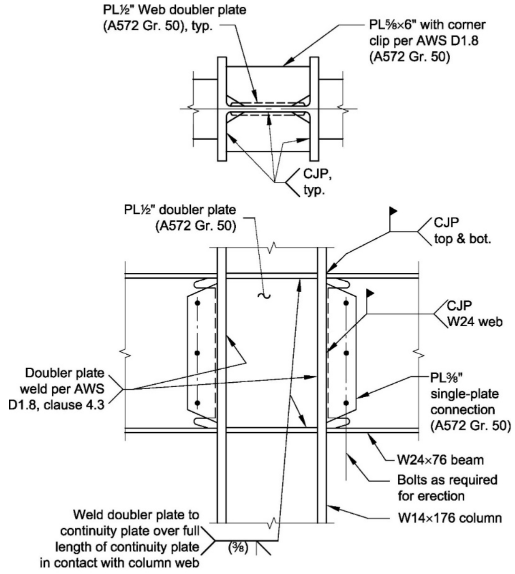
$$= \frac{76.8\text{ kips}}{(1.392\text{ kip/in.})(6.83\text{ in.})}$$

$$= 8.08\text{ sixteenths}$$

Use a PJP groove weld with a 3/8-in. effective throat (3/8 in. > 0.357 in.).

Because this weld is classified as PJP, no ultrasonic testing is required.

Example 4.3.6 SMF Beam-Column Connection Design – RBS



Note: For weld backing requirements, and treatment of weld tabs see ANSI/AISC 358, Chapter 3.

Alternative 2 - Fitted Doubler Plate

Example 4.3.6
SMF Beam-Column Connection Design

Seismic Provisions Chapter E – Moment Frames - OMF

E1 Ordinary Moment Frames

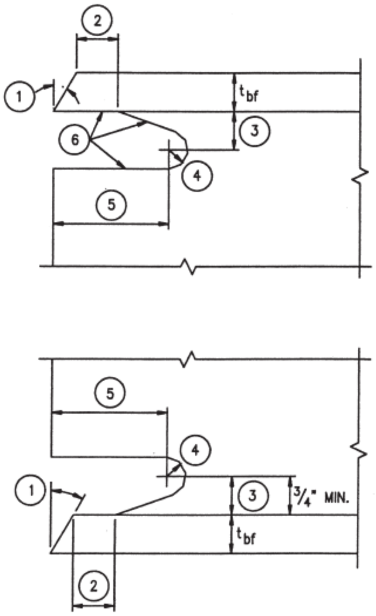
- OMF are expected to withstand minimal inelastic deformations (R = 3.5) in their members and connections when subjected to design earthquake.
- Nearly all the OMF requirements are less stringent than SMF and may simply refer the designer back to the *Specification*
- Building codes place significant limits on where OMF may be used

Seismic Provisions Chapter E – Moment Frames - OMF

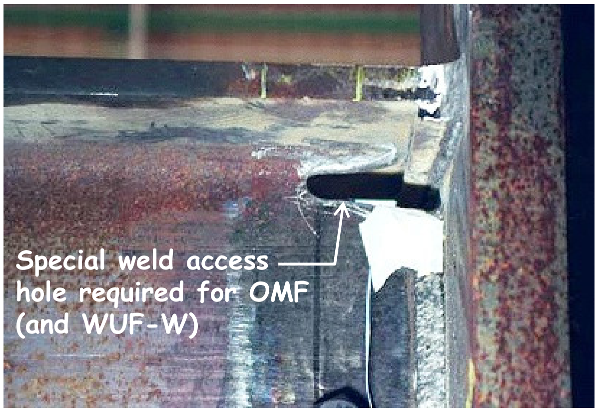
Maximum Building Height of OMF by Seismic Design Category per ASCE 7 Table 12.2-1 and Section 12.2.5.6					
Seismic Design Category	A or B	C	D	E	F
Maximum Height	No limit	No Limit	Not permitted[1,2,3]	Not permitted[1,2,3]	Not permitted[4]

1. OMF may be used in a single story building ≤ 65 ft. tall with roof dead load ≤ 20 psf and dead load of any wall tributary to the OMF > 35 ft. is ≤ 20 psf.
2. Exception: Unoccupied, single-story maintenance related structures may be of unlimited height with roof dead load ≤ 20 psf and dead load of any wall tributary to the OMF > 35 ft. is ≤ 20 psf.
3. OMF may be used in light-framed buildings of unlimited height with roof or floor dead load ≤ 35 psf and dead load of any wall tributary to the OMF is ≤ 20 psf.
4. Similar to Note 1 but exterior wall weight limit applies without respect to height.

Seismic Provisions Chapter E – Moment Frames - OMF



Notes: 1. Bevel as required for selected groove weld.
2. Larger of t_{bf} or 2 in. (13 mm) (plus 2 t_{bf}, or minus 4 t_{bf})
3. w t_{bf} to t_{bf}, w in. (19 mm) minimum (± 4 in.) (± 6 mm)
4. a in. (10 mm) minimum radius (plus not limited, minus 0)
5. 3 t_{bf} (± 2 in.) (±13 mm)



Special weld access hole required for OMF (and WUF-W)

Seismic Provisions Chapter E – Moment Frames - IMF

E2 Intermediate Moment Frames

- IMF are expected to withstand limited inelastic deformations (R = 4.5) when subjected to design earthquake.
- Most of the IMF requirements are less stringent than SMF and may simply refer the designer back to the *Specification*
- Building codes place limits on where IMF may be used

Seismic Provisions Chapter E – Moment Frames - IMF

Maximum Building Height of IMF by Seismic Design Category per ASCE 7 Table 12.2-1 and Section 12.2.5.7

Seismic Design Category	A or B	C	D	E	F
Maximum Height	No limit	No Limit	35[1,2,3]	Not permitted[1,2,4]	Not permitted[4,5]

1. IMF may be used in a single story building ≤ 65 ft. tall with roof dead load ≤ 20 psf and dead load of any wall tributary to the IMF > 35 ft. is ≤ 20 psf
2. Exception: Unoccupied, single-story maintenance related structures may be of unlimited height with roof dead load ≤ 20 psf and dead load of any wall tributary to the IMF > 35 ft. is ≤ 20 psf.
3. IMF may be used in a multi-story building ≤ 35 ft. tall with weight restrictions similar to Note 1.
4. Similar to Note 1 but with weight restrictions similar to Note 1 without regard to height
5. IMF may be used in light-framed buildings ≤ 65 ft. tall with roof dead load ≤ 20 psf and dead load of any wall tributary to the IMF > 35 ft. is ≤ 20 psf

Seismic Provisions Chapter E – SMF vs. IMF vs. OMF

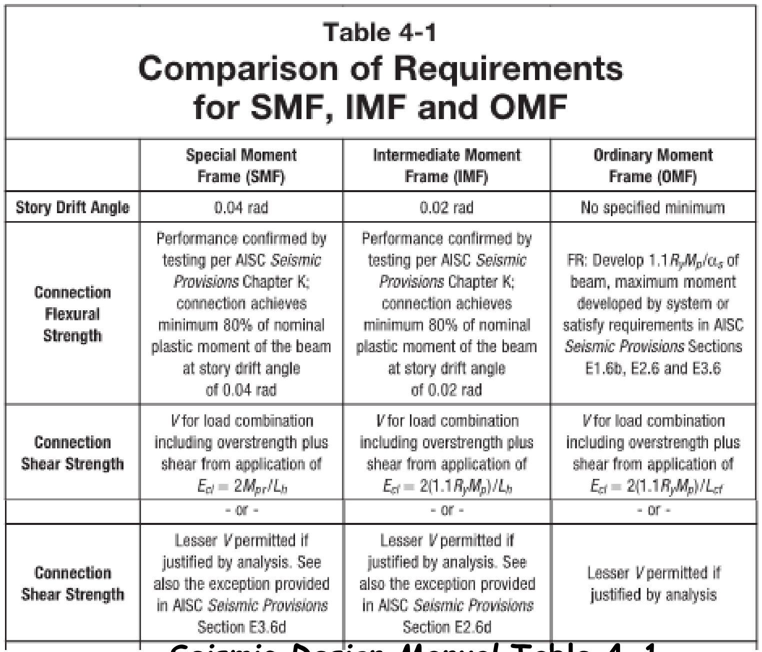


Table 4-1
Comparison of Requirements for SMF, IMF and OMF

	Special Moment Frame (SMF)	Intermediate Moment Frame (IMF)	Ordinary Moment Frame (OMF)
Story Drift Angle	0.04 rad	0.02 rad	No specified minimum
Connection Flexural Strength	Performance confirmed by testing per AISC *Seismic Provisions* Chapter K; connection achieves minimum 80% of nominal plastic moment of the beam at story drift angle of 0.04 rad	Performance confirmed by testing per AISC *Seismic Provisions* Chapter K; connection achieves minimum 80% of nominal plastic moment of the beam at story drift angle of 0.02 rad	FR: Develop $1.1R_yM_p/\alpha_s$ of beam, maximum moment developed by system or satisfy requirements in AISC *Seismic Provisions* Sections E1.6b, E2.6 and E3.6
Connection Shear Strength	V for load combination including overstrength plus shear from application of $E_{cl} = 2M_{pr}/L_h$	V for load combination including overstrength plus shear from application of $E_{cl} = 2(1.1R_yM_p)/L_h$	V for load combination including overstrength plus shear from application of $E_{cl} = 2(1.1R_yM_p)/L_{cf}$
	- or -	- or -	- or -
Connection Shear Strength	Lesser V permitted if justified by analysis. See also the exception provided in AISC *Seismic Provisions* Section E3.6d	Lesser V permitted if justified by analysis. See also the exception provided in AISC *Seismic Provisions* Section E2.6d	Lesser V permitted if justified by analysis

Seismic Design Manual **Table 4-1**

Seismic Provisions Chapter E – SMF vs. IMF vs. OMF

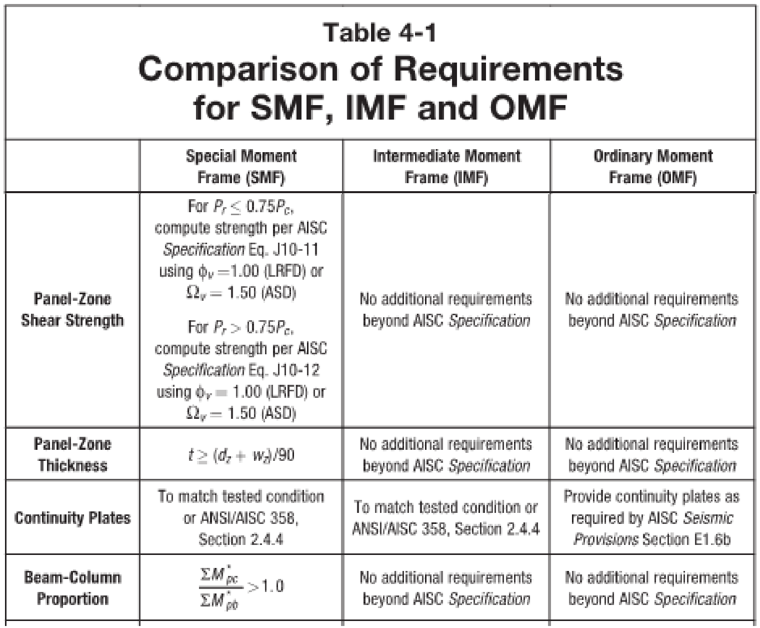


Table 4-1
Comparison of Requirements for SMF, IMF and OMF

	Special Moment Frame (SMF)	Intermediate Moment Frame (IMF)	Ordinary Moment Frame (OMF)
Panel-Zone Shear Strength	For $P_r \leq 0.75P_c$, compute strength per AISC *Specification* Eq. J10-11 using $\phi_v = 1.00$ (LRFD) or $\Omega_v = 1.50$ (ASD) For $P_r > 0.75P_c$, compute strength per AISC *Specification* Eq. J10-12 using $\phi_v = 1.00$ (LRFD) or $\Omega_v = 1.50$ (ASD)	No additional requirements beyond AISC *Specification*	No additional requirements beyond AISC *Specification*
Panel-Zone Thickness	$t \geq (d_z + w_z)/90$	No additional requirements beyond AISC *Specification*	No additional requirements beyond AISC *Specification*
Continuity Plates	To match tested condition or ANSI/AISC 358, Section 2.4.4	To match tested condition or ANSI/AISC 358, Section 2.4.4	Provide continuity plates as required by AISC *Seismic Provisions* Section E1.6b
Beam-Column Proportion	$\dfrac{\Sigma M^*_{pc}}{\Sigma M^*_{pb}} > 1.0$	No additional requirements beyond AISC *Specification*	No additional requirements beyond AISC *Specification*

Seismic Design Manual **Table 4-1**

Seismic Provisions Chapter E – SMF vs. IMF vs. OMF

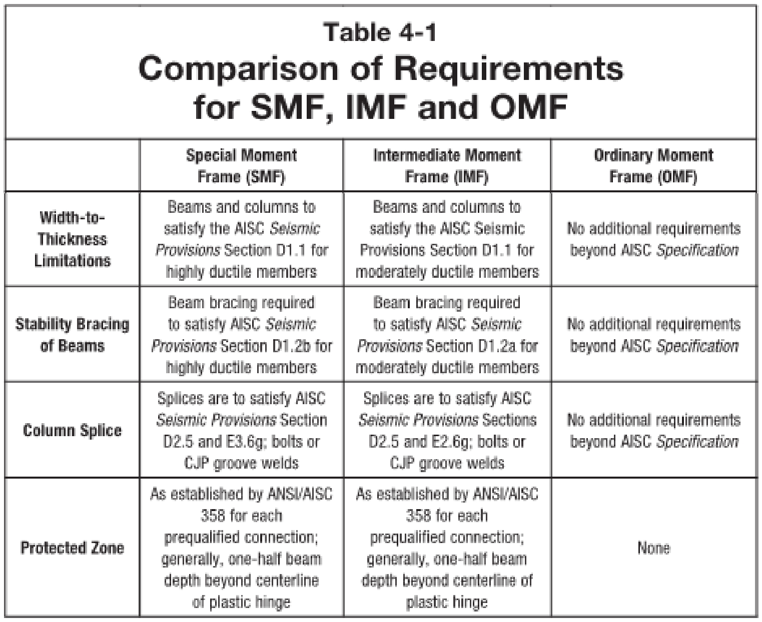


Table 4-1
Comparison of Requirements for SMF, IMF and OMF

	Special Moment Frame (SMF)	Intermediate Moment Frame (IMF)	Ordinary Moment Frame (OMF)
Width-to-Thickness Limitations	Beams and columns to satisfy the AISC *Seismic Provisions* Section D1.1 for highly ductile members	Beams and columns to satisfy the AISC Seismic Provisions Section D1.1 for moderately ductile members	No additional requirements beyond AISC *Specification*
Stability Bracing of Beams	Beam bracing required to satisfy AISC *Seismic Provisions* Section D1.2b for highly ductile members	Beam bracing required to satisfy AISC *Seismic Provisions* Section D1.2a for moderately ductile members	No additional requirements beyond AISC *Specification*
Column Splice	Splices are to satisfy AISC *Seismic Provisions* Section D2.5 and E3.6g; bolts or CJP groove welds	Splices are to satisfy AISC *Seismic Provisions* Sections D2.5 and E2.6g; bolts or CJP groove welds	No additional requirements beyond AISC *Specification*
Protected Zone	As established by ANSI/AISC 358 for each prequalified connection; generally, one-half beam depth beyond centerline of plastic hinge	As established by ANSI/AISC 358 for each prequalified connection; generally, one-half beam depth beyond centerline of plastic hinge	None

Seismic Design Manual **Table 4-1**

End of Part V

AISC

Smarter.
Stronger.
Steel.

Third Edition of AISC Seismic Design Manual

Applications of the 2016 *Seismic Provisions* – AISC 341

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SDM Part 9: *Seismic Provisions* Chapter F – Braced Frames

In this section of the seminar, we cover:

- *Seismic Provisions* Chapter F: OCBF and SCBF
    - Example 5.2.7 MT-OCBF Column Design
    - Example 5.2.8 MT-OCBF Column Design - Tension-Only Bracing Exception
    - Example 5.3.1 SCBF Brace Design
    - Example 5.3.2 SCBF Analysis
    - Example 5.3.4 SCBF Beam Design
    - Example 5.3.7 SCBF Brace-to-Brace Connection Design
    - Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

SDM Part 9: *Seismic Provisions* Chapter F – Braced Frames

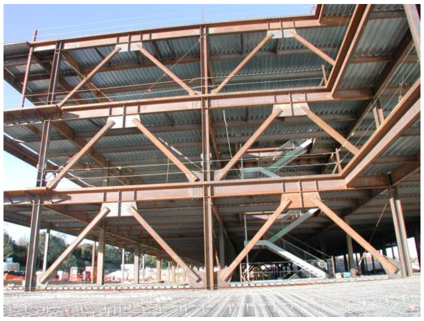
In this section of the seminar, we cover:

- *Seismic Provisions* Chapter F: BRBF
  - Example 5.5.1 BRBF Brace Design

Seismic Provisions Chapter F – Braced Frames

Design Basis and Anticipated Behavior

- Concentrically braced frames (CBF) resist lateral loads primarily through axial strength and stiffness of braces
- CBF arranged such that beam, column and brace centerlines coincide (or nearly so) to minimize flexural behavior



Seismic Provisions Chapter F – Braced Frames

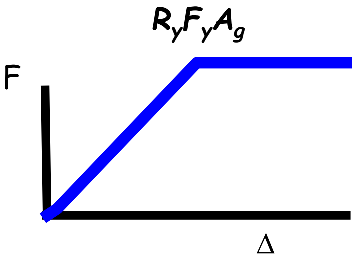
Design Basis and Anticipated Behavior

- CBF may be cost-effective relative to material, fabrication and erection costs vs. moment frames
- CBF generally less flexible in floor-plan layout and mechanical system routing
- If CBF are located in core areas to minimize floor-plan conflicts, torsion may be a significant design issue

Seismic Provisions Chapter F – Braced Frames

Design Basis and Anticipated Behavior

- Preferred mode of behavior: tension brace yielding

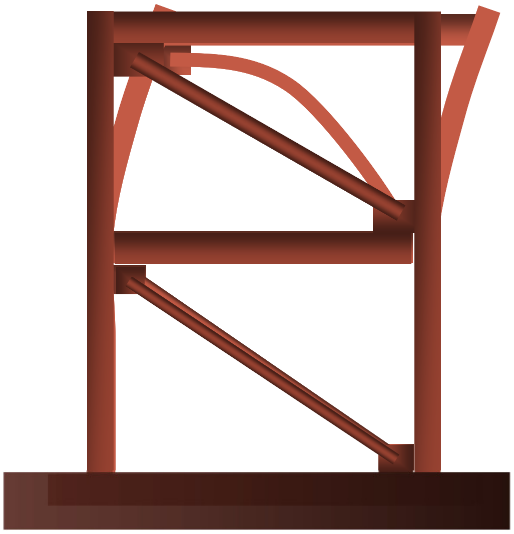


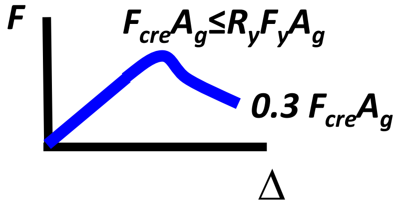
Consider maximum effects due to brace force ($R_yF_yA_g$)

Seismic Provisions Chapter F – Braced Frames

Design Basis and Anticipated Behavior

- Preferred mode of behavior: compression brace buckling



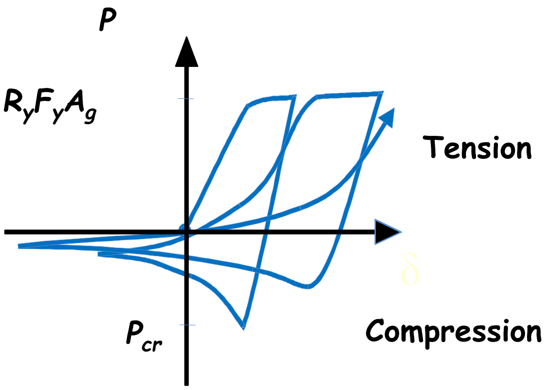


Consider maximum effect due to brace force

Seismic Provisions Chapter F – Braced Frames

Design Basis and Anticipated Behavior

- Resulting hysteretic behavior under reversing tension and compression loads

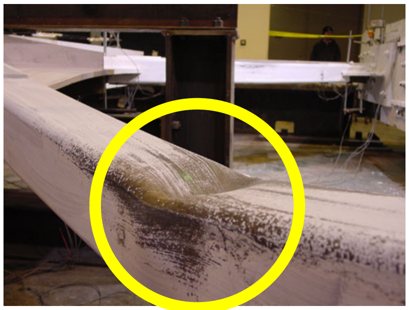


Typical brace behavior is asymmetric with respect to tension and compression and is subject to strength and stiffness degradation

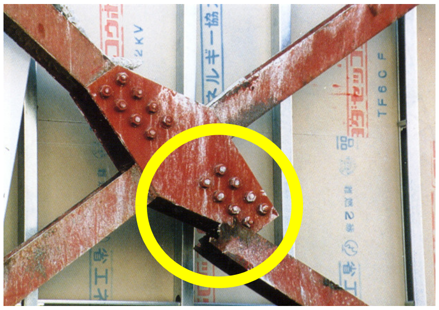
Seismic Provisions Chapter F – Braced Frames

Design Basis and Anticipated Behavior

- Undesirable modes of behavior
    - Connection fracture
    - Brace wall buckling
    - Beam failure



Brace wall buckling

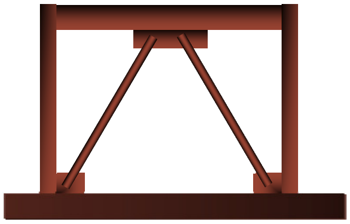


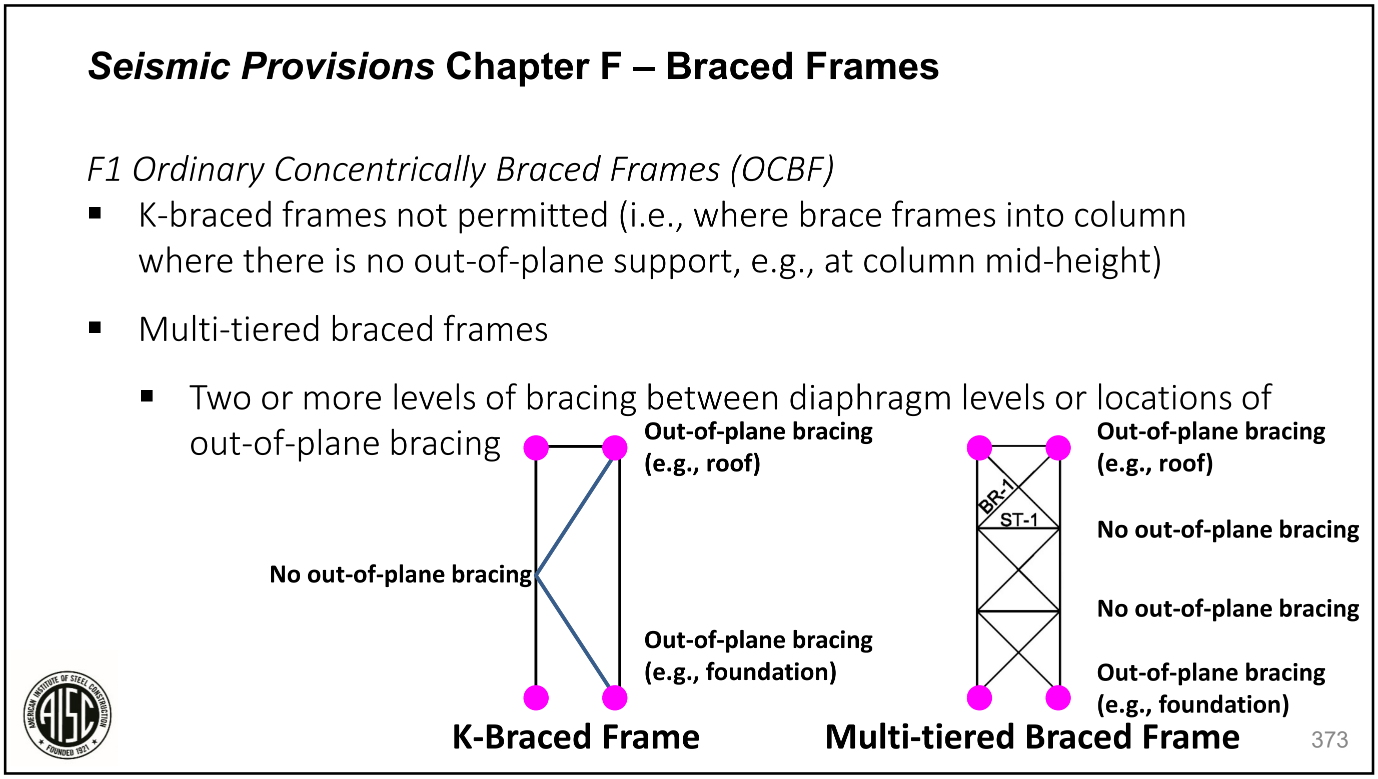
Connection fracture

Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Limited inelastic deformation capability
- Requires use of a larger seismic force for design than SCBF (i.e., $R = 3.25$)
- V- or Inverted-V configurations require horizontal beam to resist
    - Tension braces apply a force the least of load effect using overstrength or maximum force developed by the system
    - Compression braces apply a force at least equal to $0.3P_n$



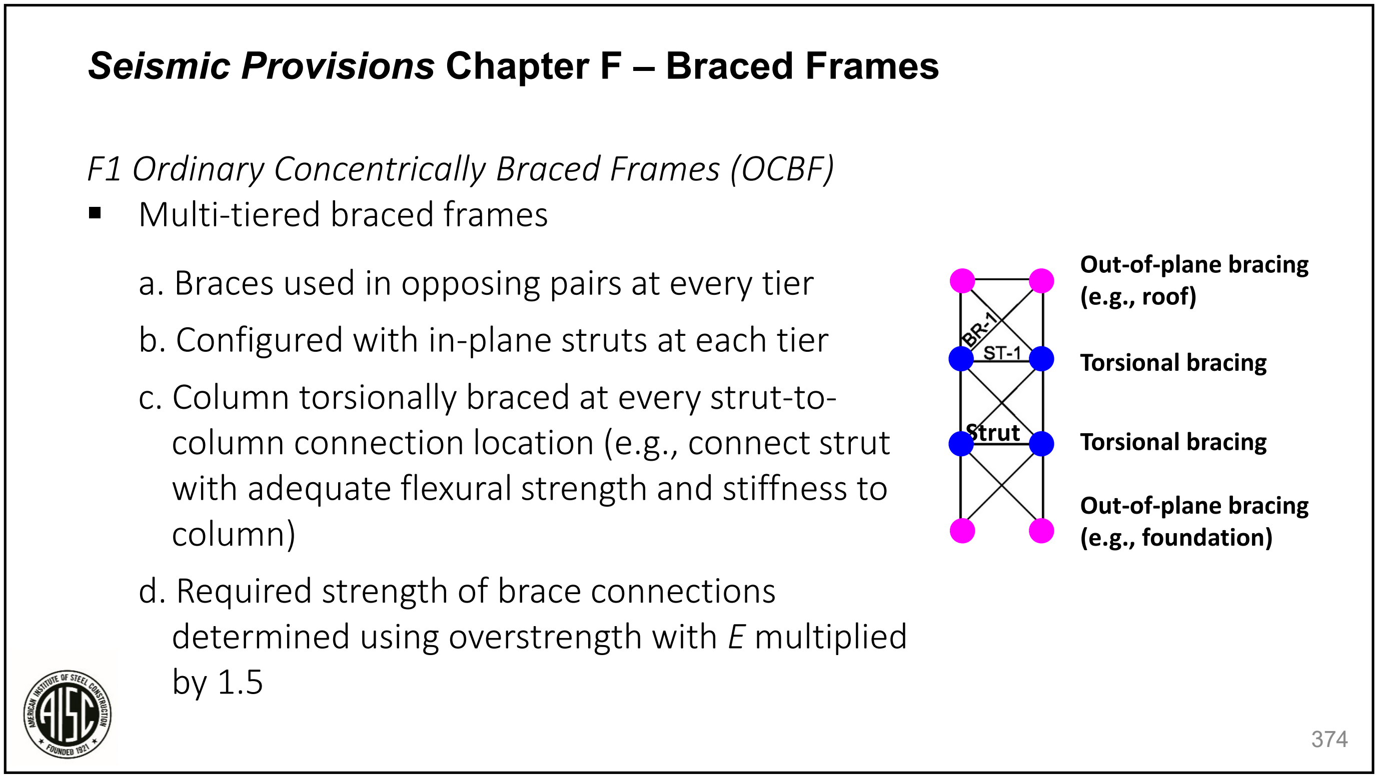


Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- K-braced frames not permitted (i.e., where brace frames into column where there is no out-of-plane support, e.g., at column mid-height)
- Multi-tiered braced frames
    - Two or more levels of bracing between diaphragm levels or locations of out-of-plane bracing

K-Braced Frame **Multi-tiered Braced Frame**



Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Multi-tiered braced frames

    a. Braces used in opposing pairs at every tier

    b. Configured with in-plane struts at each tier

    c. Column torsionally braced at every strut-to-column connection location (e.g., connect strut with adequate flexural strength and stiffness to column)

    d. Required strength of brace connections determined using overstrength with E multiplied by 1.5

Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Multi-tiered braced frames (continued)

  e. Required axial strength of struts determined using overstrength with *E* multiplied by 1.5. In tension-compression X-bracing, forces shall be determined in the absence of compression braces.

  f. Required axial strength of columns determined using overstrength with *E* multiplied by 1.5.

  g. For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects (e.g., notional load applied at every tier equal to 0.006 x (vertical load contributed by compression brace connecting column at the tier)

Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Multi-tiered braced frames (continued)
  - When tension-only bracing is used, (d), (e) and (f) need not be satisfied if:
    - Braces have controlling slenderness of 200 or more.
    - Columns designed to resist additional in-plane bending moments due to unbalanced lateral forces at each tier using capacity-limited seismic load based on expected brace strength (i.e., $R_yF_yA_g$).
    - Unbalanced lateral force at any tier shall not be less than 5% of larger horizontal brace component resisted by braced above and below tier.

Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Members
  - Braces shall be moderately ductile
  - In V- and inverted V- configurations, braces shall have

$$\frac{L_c}{r} \leq 4\sqrt{\frac{E}{F_y}}$$

where L_c = effective length of brace

Seismic Provisions Chapter F – Braced Frames

F1 Ordinary Concentrically Braced Frames (OCBF)

- Connections
  - Required strength determined using overstrength
  - Exception: Required strength need not exceed:
    - Tension: expected strength divided by α_s (i.e., $R_yF_yA_g/\alpha_s$)
    - Compression: expected strength divided by α_s (i.e., lesser of $R_yF_yA_g/\alpha_s$ and $1.1F_{cre}A_g/\alpha_s$). Use R_yF_y in lieu of F_y when using *Specification* equations for F_{cre}. Brace length to determine F_{cre} shall not exceed distance from brace end to brace end.

Example 5.2.7
MT-OCBF Column Design

- Example illustrating application of MTBF requirements (SDM pg. 5-101)
- Example worked in LRFD

Example 5.2.7 MT-OCBF Column Design

Given:
Refer to Figure 5-11 to select an ASTM A992 W-shape for Column CL-1.

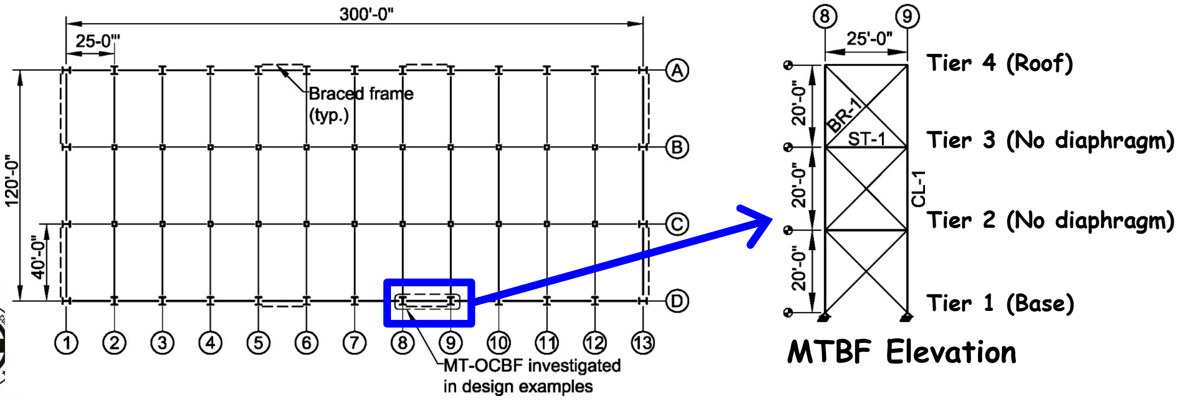
From ASCE 7, the following parameters apply: Seismic Design Category D, R = 3-1/4, $\Omega_o = 2$, $\rho = 1.0$, $I_e = 1.0$.

For this example:

- Note that the ends of the columns are assumed as pinned and braced against translation for both the *x-x* and *y-y* axes, and against rotation.
- Loading for the columns is determined from a first-order analysis.
- Use the approximate method given in AISC *Specification* Appendix 8 to account for second-order effects.

Example 5.2.7 MT-OCBF Column Design

Also note that many columns in the building (the sidewalls and the interior) rely on the four OCBF for stability. The code requires frame stability analysis and the consideration of leaning columns; however, an exhaustive stability analysis is not presented in this example in order to focus on the seismic requirements.



Example 5.2.7 MT-OCBF Column Design

Solution:

From AISC *Manual* Table 2-4, the material properties are:

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Example 5.2.7 MT-OCBF Column Design

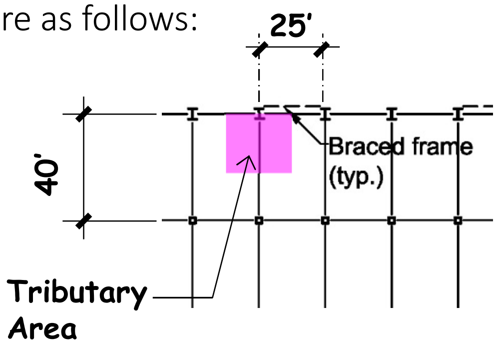
Required Axial Strength

From the plan in Figure 5-10, the tributary area for Column CL-1 is:

$$(25 \text{ ft})(20 \text{ ft}) = 500 \text{ ft}^2$$

Therefore, axial forces due to gravity loads are as follows:

$$\begin{aligned} P_D &= (19 \text{ psf})\left(500 \text{ ft}^2\right)(1 \text{ kip}/1{,}000 \text{ lb}) \\ &= 9.50 \text{ kips} \\ P_S &= (20 \text{ psf})\left(500 \text{ ft}^2\right)(1 \text{ kip}/1{,}000 \text{ lb}) \\ &= 10.0 \text{ kips} \end{aligned}$$



Example 5.2.7 MT-OCBF Column Design

From Example 5.2.6, the axial force in the brace due to seismic loading is 64.5 kips. The seismic force causing compression in the column (calculated for the lowest level of column section, including the vertical components of three braces):

$$\begin{aligned} P_{QE} &= 3(64.5 \text{ kips})\sin 38.7° \\ &= 121 \text{ kips} \end{aligned}$$

(From Example 5.2.6, SDM pg. 5-98)

$$\begin{aligned} P_u &= (1.2 + 0.2S_{DS})P_D + \rho P_{QE} + 0.5P_L + 0.2P_S \\ &= [1.2 + 0.2(0.738)](0 \text{ kips}) + (1.0)(64.5 \text{ kips}) + 0.5(0 \text{ kips}) + 0.2(0 \text{ kips}) \\ &= 64.5 \text{ kips} \end{aligned}$$

Seismic Provisions Section D1.4a says required axial strength for columns determined from the greater effect of the following:

- load effect from the analysis requirements per the system provisions
- compressive axial strength and tensile strength using the overstrength seismic load; that is, the seismic load multiplied by the overstrength factor, Ω_o.

Example 5.2.7 MT-OCBF Column Design

Per AISC *Seismic Provisions* Section F1.4c(f), the seismic force is to be increased by an additional factor of 1.5.

Considering the load combinations given in ASCE/SEI 7, with E_v and E_h incorporated as defined in Section 12.4.3, the required axial strength of the column from Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L)* is:

$$\begin{aligned} P_u &= \left(1.2+0.2S_{DS}\right)P_D+\Omega_o P_{QE}+0.5P_L+0.2P_S \\ &= \left[1.2+0.2(0.738)\right](9.50\text{ kips})+(1.5)(2)(121\text{ kips})+0.5(0\text{ kips})+0.2(10.0\text{ kips}) \\ &= 378\text{ kips} \end{aligned}$$

Ω_o

1.5 amplification per F1.4c(f)

Example 5.2.7 MT-OCBF Column Design

and from Load Combination 7 from ASCE/SEI 7, Section 2.3.6:

$$\begin{aligned} P_u &= \left(0.9-0.2S_{DS}\right)P_D+\Omega_o P_{QE} \\ &= \left[0.9-0.2(0.738)\right](9.50\text{ kips})+(1.5)(2)(121\text{ kips}) \\ &= 370\text{ kips} \end{aligned}$$

Ω_o

1.5 amplification per F1.4c(f)

Required Flexural Strength—Out-of-Plane

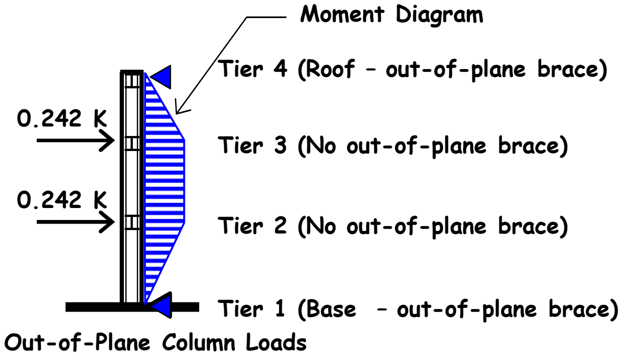
AISC *Seismic Provisions* Section F1.4c(g) stipulates that in an MT-OCBF, columns subjected to axial compression are also designed to resist bending moments due to second-order and geometric imperfection effects.

Example 5.2.7 MT-OCBF Column Design

To satisfy this requirement, an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace connecting to the column at the tier level may be developed.

Based on the required axial strength of the tension-only brace in Example 5.2.6, the horizontal loads at tier levels 2 and 3, due to imperfection effects, are:

$$\begin{aligned} P_u &= 0.006(64.5\text{ kips})\sin 38.7^\circ \\ &= 0.242\text{ kip} \end{aligned}$$



Out-of-Plane Column Loads

Example 5.2.7 MT-OCBF Column Design

The column is restrained against translation at its top and bottom, and as shown on the plan in Figure 5-10, the column strong axis is oriented to resist out-of-plane moments. Therefore:

$$\begin{aligned} M_{ux} &= (0.242\text{ kip})(20\text{ ft}) \\ &= 4.84\text{ kip-ft} \end{aligned}$$

Column Design

Using a column orientation with the strong axis out-of-plane of the frame, try a W14×90.

From AISC *Manual* Table 1-1, the geometric properties are as follows:

$A = 26.5\text{ in.}^2$ $d = 14.0\text{ in.}$ $r_x = 6.14\text{ in.}$ $r_y = 3.70\text{ in.}$

$I_x = 999\text{ in.}^4$

Example 5.2.7 MT-OCBF Column Design

For the available compressive strength, the element slenderness is checked according to AISC *Specification* Table B4.1a. As indicated in AISC *Manual* Table 1-1, the section is not slender for compression with F_y = 50 ksi.

From AISC *Specification* Commentary Table C-A-7.1, for a pinned-pinned condition, $K_x = K_y = 1.0$.

$$\frac{L_{cx}}{r_x} = \frac{K_x L_x}{r_x} = \frac{1.0(60\text{ ft})(12\text{ in./ft})}{6.14\text{ in.}} = 117\text{ (governs)}$$

$$\frac{L_{cy}}{r_y} = \frac{K_y L_y}{r_y} = \frac{1.0(20\text{ ft})(12\text{ in./ft})}{3.70\text{ in.}} = 64.9$$

Example 5.2.7 MT-OCBF Column Design

From AISC *Manual* Table 4-14 with L_c/r = 117 and using AISC *Specification* Equation E3-1, the available compressive strength is:

$$\begin{aligned} \phi_c F_{cr} &= 16.5\text{ ksi} \\ \phi_c P_n &= \phi_c F_{cr} A_g \\ &= (16.5\text{ ksi})(26.5\text{ in.}^2) \\ &= 437\text{ kips} > 378\text{ kips} \end{aligned}$$

o.k.

$P_{u\ (compression)}$ = 378 K

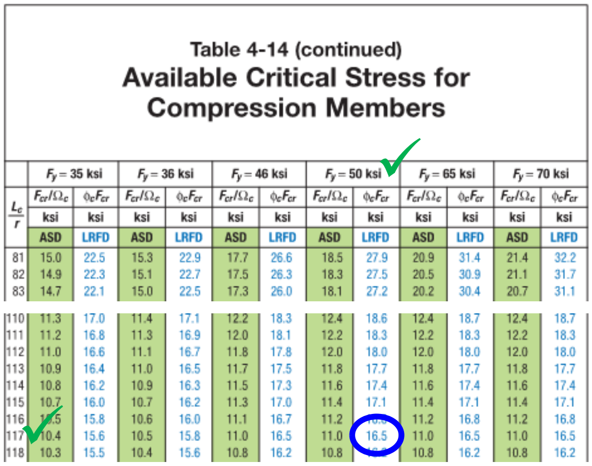


Table 4-14 (continued)

Available Critical Stress for Compression Members

$\frac{L_c}{r}$	$F_y = 35$ ksi		$F_y = 36$ ksi		$F_y = 46$ ksi		$F_y = 50$ ksi ✓		$F_y = 65$ ksi		$F_y = 70$ ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
81	15.0	22.5	15.3	22.9	17.7	26.6	18.5	27.9	20.9	31.4	21.4	32.2
82	14.9	22.3	15.1	22.7	17.5	26.3	18.3	27.5	20.5	30.9	21.1	31.7
83	14.7	22.1	15.0	22.5	17.3	26.0	18.1	27.2	20.2	30.4	20.7	31.1
110	11.3	17.0	11.4	17.1	12.2	18.3	12.4	18.6	12.4	18.7	12.4	18.7
111	11.2	16.8	11.3	16.9	12.0	18.1	12.2	18.3	12.2	18.3	12.2	18.3
112	11.0	16.6	11.1	16.7	11.8	17.8	12.0	18.0	12.0	18.0	12.0	18.0
113	10.9	16.4	11.0	16.5	11.7	17.5	11.8	17.7	11.8	17.7	11.8	17.7
114	10.8	16.2	10.9	16.3	11.5	17.3	11.6	17.4	11.6	17.4	11.6	17.4
115	10.7	16.0	10.7	16.2	11.3	17.0	11.4	17.1	11.4	17.1	11.4	17.1
116	10.5	15.8	10.6	16.0	11.1	16.7	11.2	16.8	11.2	16.8	11.2	16.8
117 ✓	10.4	15.6	10.5	15.8	11.0	16.5	11.0	16.5	11.0	16.5	11.0	16.5
118	10.3	15.5	10.4	15.6	10.8	16.2	10.8	16.2	10.8	16.2	10.8	16.2

Steel Manual Table 4-14

Example 5.2.7 MT-OCBF Column Design

From AISC *Manual* Table 6-2, with unbraced length, L_b = 20 ft, the available flexural strength about the *x*-*x* axis is:

M_{ux} from out-plane forces

$$\phi_b M_n = 539 \text{ kip-ft} > 4.84 \text{ kip-ft} \quad \textbf{o.k.}$$

Note that C_b = 1.0 for the center section of the column and 1.67 for the lower and upper sections of the column. Conservatively, use C_b = 1.0 for this column design.

Steel Manual Table 6-2

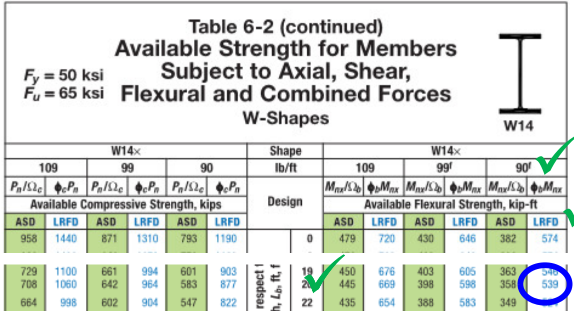


Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W14

W14× 109 P_n/Ω_c ASD	W14× 109 $\phi_c P_n$ LRFD	W14× 99 P_n/Ω_c ASD	W14× 99 $\phi_c P_n$ LRFD	W14× 90 P_n/Ω_c ASD	W14× 90 $\phi_c P_n$ LRFD	Shape lb/ft Design		W14× 109 M_{nx}/Ω_b ASD	W14× 109 $\phi_b M_{nx}$ LRFD	W14× 99^f M_{nx}/Ω_b ASD	W14× 99^f $\phi_b M_{nx}$ LRFD	W14× 90^f M_{nx}/Ω_b ASD	W14× 90^f $\phi_b M_{nx}$ LRFD
958	1440	871	1310	793	1190		0	479	720	430	646	382	574
729	1100	661	994	601	903	respect	19	450	676	403	605	363	546
708	1060	642	964	583	877	L_b, ft	20	445	669	398	598	358	539
664	998	602	904	547	822		22	435	654	388	583	349	[illegible]

Available Compressive Strength, kips / Available Flexural Strength, kip-ft

---

Example 5.2.7 MT-OCBF Column Design

Second-Order Effects
Calculated using *Specification* Appendix 8: "Approximate Second-Order Analysis"
For P-Δ: B_2 = 1.01 (very small impact, see SDM pgs. 5-104 – 5-107 for the calculations)

Calculate B_1x:

B_1 is multiplier to account for P-δ effects (see Spec App. 8). δ is deflected shape of member.

$$\alpha = 1.0$$

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

(Spec. Eq. A-8-3)

Example 5.2.7 MT-OCBF Column Design

As previously calculated, P_r is from Load Combination 8 from ASCE 7, Section 2.4.5:

$$B_{1x} = \frac{1.0}{1 - \frac{1.0(381\text{ kips})}{552\text{ kips}}} \geq 1$$

$$= 3.23 > 1$$

C_m may be taken as 1.0 for transversely loaded members (Spec. App. 8.2.1)

P_{e1} calculated on SDM pg. 5-106 using Spec. Eq. A-8-5)

$$P_{e1} = \frac{\pi^2 EI^*}{(L_{c1})^2}$$

$$P_{e1x} = \frac{\pi^2(29{,}000\text{ ksi})(999\text{ in.}^4)}{\left[1.0(60\text{ ft})(12\text{ in./ft})\right]^2}$$

$$= 552\text{ kips}$$

$$C_m = 1.0 \text{ as a conservative assumption}$$

Example 5.2.7 MT-OCBF Column Design

The *x-x* axis, out-of-plane moment is amplified as follows. Note that this moment is taken as M_{nt} with the structure restrained against lateral translation.

From Load Combination 8 from ASCE 7, Section 2.4.5:

$$B_{1x}M_{ux} = B_{1x}M_{ntx}$$

$$= 3.23(4.84\text{ kip-ft})$$

$$= 15.6\text{ kip-ft}$$

This is the amplified moment due to P-δ effects that accounts for deflected shape of column loaded out-of-plane.

Example 5.2.7 MT-OCBF Column Design

Combined axial compressive and flexural strength will be checked using AISC *Specification* Section H1. Determine the applicable interaction equation in AISC *Specification* Section H1.1:

$$\frac{P_r}{P_c} = \frac{381 \text{ kips}}{437 \text{ kips}}$$
$$= 0.872 > 0.2$$

Example 5.2.7 MT-OCBF Column Design

Because $P_r/P_c \geq 0.2$, the column design is controlled by the equation:

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(}\textit{Spec.}\text{ Eq. H1-1a)}$$

$$0.872 + \frac{8}{9}\left(\frac{15.6 \text{ kip-ft}}{539 \text{ kip-ft}} + 0\right) \leq 1.0$$
$$= 0.898 < 1.0 \quad \textbf{o.k.}$$

Therefore, a W14×90 is adequate for the column.

Example 5.2.7

MT-OCBF Column Design

Example 5.2.8

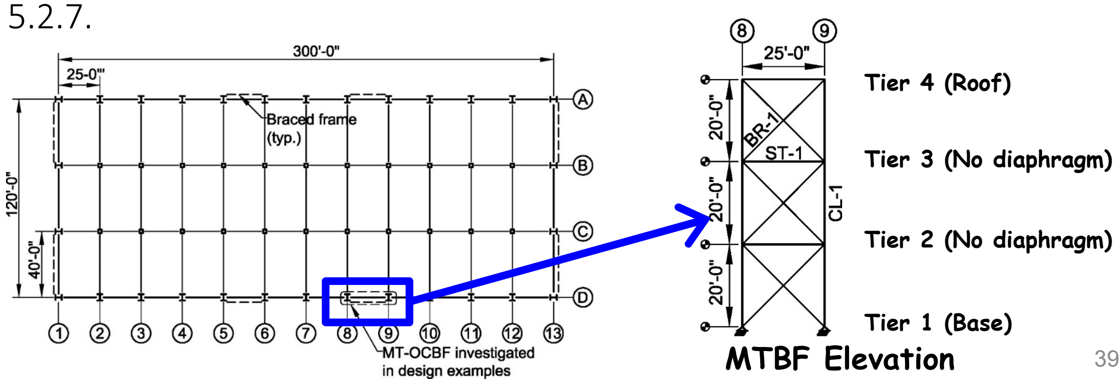
MT-OCBF Column Design Using Tension-Only Bracing Exception

- Example illustrating application of MTBF requirements on SDM pg. 5-109
- The example includes consideration of second order effects not emphasized in earlier examples
- Example worked in ASD

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Given:

Because tension-only bracing is being used, AISC *Seismic Provisions* Section F1.4c(h) may be applied if the conditions of Section F1.4c(h)(1) and Section F1.4c(h)(2) are met. Using this alternate approach, verify the column design in Example 5.2.7.



Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Note that the ends of the columns are assumed as pinned and braced against translation for both the *x-x* and *y-y* axes, and against rotation. Loading for the columns is determined from a first-order analysis. Use the approximate method given in AISC *Specification* Appendix 8 to account for second-order effects.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Solution:

The following changes are made to previous design requirements and applied in this example:

1. The required Brace BR-1 connection force is reduced to the basic requirement for OCBF frames:

$$\begin{aligned} P_{QE} &= \Omega_o Q_E \\ &= 2(64.5\text{ kips}) \\ &= 129\text{ kips} \end{aligned}$$

Per Section F1.4c(h), when tension-only bracing is used, required axial strength does not need to be amplified by 1.5 as was done in Example 5.2.7

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

2. The required Strut ST-1 axial force is reduced to the basic requirement for OCBF frames:

$$\begin{aligned} P_{QE} &= \Omega_o Q_E \\ &= 2(50.3\text{ kips}) \\ &= 101\text{ kips} \end{aligned}$$

Per Section F1.4c(h), when tension-only bracing is used, required axial strength does not need to be amplified by 1.5 as was done in Example 5.2.7

3. The required axial strength of the column is reduced to the basic requirements for OCBF frames. The basic load combinations with seismic load effects including overstrength from ASCE/SEI, Section 2.3.6 (LRFD) and Section 2.4.5 (ASD), are used, with E_v and E_h incorporated as defined in Section 12.4.3 (without the additional 1.5 factor on the seismic force). The gravity and seismic forces calculated in Example 5.2.7 are used.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

From Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(1.0+0.14S_{DS}\right)P_D+0.7\Omega_o P_{QE} \\ &= \left[1.0+0.14\left(0.738\right)\right]\left(9.50\text{ kips}\right)+0.7\left(2\right)\left(121\text{ kips}\right) \\ &= 180\text{ kips} \end{aligned}$$

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(1.0+0.105S_{DS}\right)P_D+0.525\Omega_o P_{QE}+0.75P_L+0.75P_S \\ &= \left[1.0+0.105\left(0.738\right)\right]\left(9.50\text{ kips}\right)+0.525\left(2\right)\left(121\text{ kips}\right)+0.75\left(0\text{ kips}\right)+0.75\left(10.0\text{ kips}\right) \\ &= 145\text{ kips} \end{aligned}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

and from Load Combination 10 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(0.6-0.14S_{DS}\right)P_D+0.7\Omega_o P_{QE} \\ &= \left[0.6-0.14\left(0.738\right)\right]\left(9.50\text{ kips}\right)+0.7\left(2\right)\left(121\text{ kips}\right) \\ &= 174\text{ kips} \end{aligned}$$

4. The column is now required to be designed for in-plane flexural moments, associated with possible inherent differences in the strength of the bracing tiers.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Per AISC *Seismic Provisions* Section F1.4c(h)(2), the expected yield strength of the ASTM A36 L4×4×5/16 brace in tension, with R_y = 1.5 from AISC *Seismic Provisions* Table A3.1, is:

$$\begin{aligned} R_y F_y A_g &= 1.5(36\text{ ksi})(2.40\text{ in.}^2) \\ &= 130\text{ kips} \end{aligned}$$

As previously determined, this is a very slender brace and the controlling slenderness ratio exceeds 200. Assume that the available compressive strength is negligible.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

The lateral force associated with developing the maximum expected tensile strength of the brace is calculated as follows:

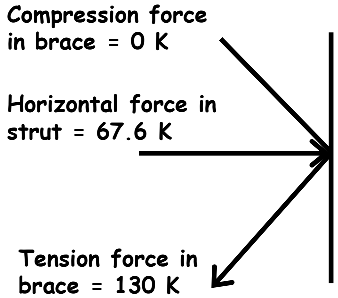
$$\begin{aligned} P_l &= \frac{R_y F_y A_g \cos 38.7°}{\alpha_s} \\ &= \frac{(130\text{ kips})\cos 38.7°}{1.5} \\ &= 67.6\text{ kips} \end{aligned}$$

AISC *Seismic Provisions* Section F1.4c(h)(2) requires that columns be designed for additional in-plane bending moments resulting from the unbalanced lateral forces at each tier. These unbalanced lateral forces are determined using expected brace strengths.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

For conditions where the same brace size and geometry occur at each tier, this method gives no unbalanced lateral force. For such cases, the column is designed for a minimum unbalanced lateral force equal to 5% of the larger horizontal shear applied above and below the tier to address unknown variations in strength, geometry and response. This is equal to:

$$\begin{aligned} P_a &= 0.05P_l \\ &= 0.05(67.6\text{ kips}) \\ &= 3.38\text{ kips} \end{aligned}$$



Compression force in brace = 0 K

Horizontal force in strut = 67.6 K

Tension force in brace = 130 K

Although horizontal components are in equilibrium, assumed minimum force of 0.05 × strut force is applied at each intermediate tier

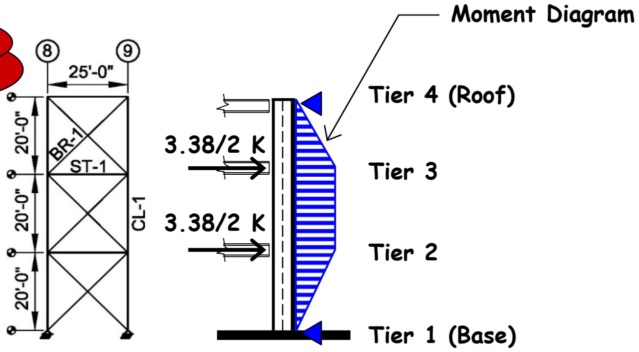
---

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

This in-plane force is applied at Tiers 2 and 3. The column is restrained at its top and bottom. Therefore:

In-plane force (and moment) divided between the two frame columns

$$\begin{aligned} M_{ay} &= M_a \\ &= \left(\frac{3.38\text{ kips}}{2}\right)(20\text{ ft}) \\ &= 33.8\text{ kip-ft} \end{aligned}$$

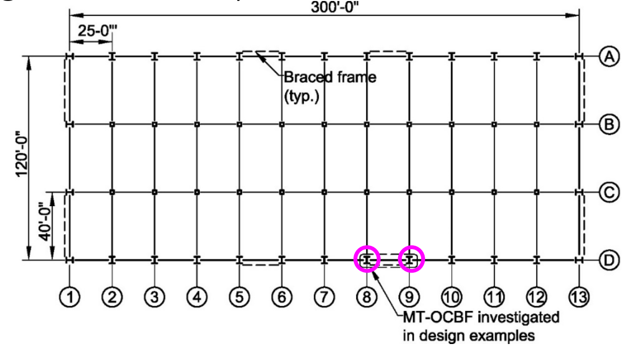


In-Plane Column Loads (braces not shown)

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Note that the braced frame is composed of two columns, and therefore, the in-plane moment is split to both columns. If more columns were connected at each tier level, the in-plane moment would be dispersed to more columns.

Using an orientation with strong-axis bending out-of-plane of the frame, as illustrated in Figure 5-10, verify the W14×90 chosen in the previous example.



Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

From AISC *Manual* Table 1-1, the geometric properties of a W14×90 are as follows:

A = 26.5 in.^{2} d = 14.0 in. r_x = 6.14 in. r_y = 3.70 in.

I_x = 999 in.^{4} I_y = 362 in.^{4}

From Example 5.2.7, the available compressive strength is:

$$\frac{P_n}{\Omega_c} = 292 \text{ kips} > 180 \text{ kips} \quad \textbf{o.k.}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

As determined in Example 5.2.7, with unbraced length, L_b = 20 ft, the available flexural strength about the *x*-*x* axis is:

$$\frac{M_{nx}}{\Omega_b} = 358 \text{ kip-ft} > 3.40 \text{ kip-ft} \quad \textbf{o.k.}$$

ASD value of out-of-plane moment calculated in Example 5.2.7

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

From AISC *Manual* Table 6-2, the available flexural strength about the *y*-*y* axis is:

$$\frac{M_{ny}}{\Omega_b} = 181 \text{ kip-ft} > 33.8 \text{ kip-ft} \quad \textbf{o.k.}$$

According to the footnote in Table 6-2, note that the W14×90 is noncompact for flexure, and AISC *Manual* Table 6-2 accounts for this.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Second-Order Effects

Calculated using *Specification* Appendix 8: “Approximate Second-Order Analysis” (see SDM pgs. 5-112 – 5-114 for calculations)

For P-Δ: B_2 = 1.01 (very small impact)

The value of B_1 is more significant.

For P-δ: B_{1y} = 1.19 (calculated as in Example 5.2.7 but for y-axis using ASD)

B_{1x} = 1.15 (calculated as in Example 5.2.7 but for x-axis using ASD)

Unlike in Example 5.2.7, the values of B1 are needed for both x-axis and y-axis because the column is subjected to out-of-plane and in-plane moments, and the following slides show how B_1 was obtained.

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Follow the approximate procedure of AISC *Specification* Appendix 8 to account for second-order effects.

$P_r = P_{nt} + B_2 P_{lt}$ (*Spec.* Eq. A-8-2)

$M_r = B_1 M_{nt} + B_2 M_{lt}$ (*Spec.* Eq. A-8-1)

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Calculate B$_1$ *for the* y-y *axis (in the plane of the frame) and* x-x *axis (out of the plane of the frame)*

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad \text{(}\textit{Spec.}\text{ Eq. A-8-3)}$$

$$P_{e1} = \frac{\pi^2 EI^*}{(L_{c1})^2} \qquad \text{(}\textit{Spec.}\text{ Eq. A-8-5)}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

$$P_{e1x} = \frac{\pi^2 (29{,}000 \text{ ksi})(999 \text{ in.}^4)}{[1.0(60 \text{ ft})(12 \text{ in./ft})]^2}$$

$$= 552 \text{ kips}$$

I$_x$ of W14x90 column

X-axis unbraced length

$$P_{e1y} = \frac{\pi^2 (29{,}000 \text{ ksi})(362 \text{ in.}^4)}{[1.0(20 \text{ ft})(12 \text{ in./ft})]^2}$$

$$= 1{,}800 \text{ kips}$$

I$_y$ of W14x90 column

Y-axis unbraced length

Use $C_m = 1.0$ as a conservative assumption

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Recalculate the required strengths including second-order effects:

From Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(1.0 + 0.14S_{DS}\right)P_D + B_2 0.7\Omega_o P_{QE} \\ &= \left[1.0 + 0.14\left(0.738\right)\right]\left(9.50\text{ kips}\right) + 1.01\left(0.7\right)\left(2\right)\left(121\text{ kips}\right) \\ &= 182\text{ kips} \end{aligned}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} P_a &= \left(1.0 + 0.105S_{DS}\right)P_D \\ &\quad + B_2 0.525\Omega_o P_{QE} + 0.75P_L + 0.75P_S \\ &= \left[1.0 + 0.105\left(0.738\right)\right]\left(9.50\text{ kips}\right) + 1.01\left(0.525\right)\left(2\right)\left(121\text{ kips}\right) + 0.75\left(0\text{ kips}\right) \\ &\quad + 0.75\left(10.0\text{ kips}\right) \\ &= 146\text{ kips} \end{aligned}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

Calculate B_{1x} and B_{1y} including second-order effects:

$$\alpha = 1.6$$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

As previously calculated, P_r is from Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$B_{1x} = \frac{1.0}{1 - \frac{1.6(182 \text{ kips})}{552 \text{ kips}}} \geq 1$$

$$= 2.12 > 1$$

$$B_{1y} = \frac{1.0}{1 - \frac{1.6(182 \text{ kips})}{1{,}800 \text{ kips}}} \geq 1$$

$$= 1.19 > 1$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$B_{1x} = \frac{1}{1 - \dfrac{1.6(146 \text{ kips})}{552 \text{ kips}}} \geq 1$$

$$= 1.73 > 1$$

$$B_{1y} = \frac{1}{1 - \dfrac{1.6(146 \text{ kips})}{1{,}800 \text{ kips}}} \geq 1$$

$$= 1.15 > 1$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

The *y-y* axis (in-plane moment) and *x-x* axis (out-of-plane moment) are amplified as follows. Note that this moment is taken as M_{nt} with the structure restrained against lateral translation.

From Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$\begin{aligned} B_{1x}M_{ax} &= B_{1x}M_{ntx} \\ &= 2.12(3.40 \text{ kip-ft}) \\ &= 7.21 \text{ kip-ft} \end{aligned}$$

$$\begin{aligned} B_{1y}M_{ay} &= B_{1y}M_{nty} \\ &= 1.19(33.8 \text{ kip-ft}) \\ &= 40.2 \text{ kip-ft} \end{aligned}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$
\begin{aligned}
B_{1x}M_{ax} &= B_{1x}M_{ntx} \\
&= 1.73(3.40\text{ kip-ft}) \\
&= 5.88\text{ kip-ft}
\end{aligned}
$$

$$
\begin{aligned}
B_{1y}M_{ay} &= B_{1y}M_{nty} \\
&= 1.15(33.8\text{ kip-ft}) \\
&= 38.9\text{ kip-ft}
\end{aligned}
$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

The combined flexural and compressive load will be checked using AISC *Specification* Section H1. Determine the applicable interaction equation from AISC *Specification* Section H1.1:

As previously calculated, P_r is from Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$
\begin{aligned}
\frac{P_r}{P_c} &= \frac{182\text{ kips}}{292\text{ kips}} \\
&= 0.623 > 0.2
\end{aligned}
$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$\frac{P_r}{P_c} = \frac{146 \text{ kips}}{292 \text{ kips}}$$
$$= 0.500 > 0.2$$

Because $P_r / P_c \geq 0.2$, the column design is controlled by the equation:

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$ (*Spec.* Eq. H1-1a)

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

As previously calculated, M_{rx} and M_{ry} are from Load Combination 8 from ASCE/SEI 7, Section 2.4.5:

$$0.623 + \frac{8}{9}\left(\frac{7.21 \text{ kip-ft}}{358 \text{ kip-ft}} + \frac{40.2 \text{ kip-ft}}{181 \text{ kip-ft}}\right) \leq 1.0$$
$$= 0.838 < 1.0 \quad \textbf{o.k.}$$

and from Load Combination 9 from ASCE/SEI 7, Section 2.4.5:

$$0.500 + \frac{8}{9}\left(\frac{5.88 \text{ kip-ft}}{358 \text{ kip-ft}} + \frac{38.9 \text{ kip-ft}}{181 \text{ kip-ft}}\right) \leq 1.0$$
$$= 0.706 < 1.0 \quad \textbf{o.k.}$$

Example 5.2.8 MT-OCBF Column Design Using Tension-Only Bracing Exception

As illustrated, use of the exception in AISC *Seismic Provisions* Section F1.4c(h) reduces the axial load on the column, but the additional in-plane column moments result in the column having a similar stress ratio. However, use of this exception results in a lower strut design force and lower brace connection design force.

Example 5.2.8

MT-OCBF Column Design Using Tension-Only Bracing Exception

Seismic Provisions Chapter F – Braced Frames

F2 Special Concentrically Braced Frames (SCBF)

- SCBF anticipate inelastic deformation demand and use a smaller seismic force for design than OCBF
- Goal is to maintain high level of ductility and hysteretic damping under severe seismic forces
- There are key detailing differences between OCBF and SCBF
  - More restrictive limits on slenderness, compressive strength, width-to-thickness ratios
  - Special detailing requirements for gusset plates and built-up brace members

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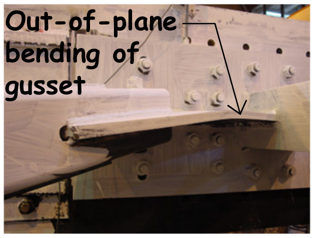
Seismic Provisions Chapter F – Braced Frames

F2 Special Concentrically Braced Frames (SCBF)

- There are key detailing differences between OCBF and SCBF (continued)
  - Connections must be detailed to accommodate effects of brace buckling:
    - Connection is strong and stiff enough to force plastic hinges to form at ends and middle of brace…or…
    - Out-of-plane bending of gusset plate results in plastic hinges in gusset and middle of brace



Seismic Provisions Chapter F – Braced Frames

F2 Special Concentrically Braced Frames (SCBF)

- There are key detailing differences between OCBF and SCBF (continued)
  - Beam-to-column connections must be detailed to accommodate large drifts
    - Option 1: Connection provides sufficient rotation capacity. Beam and column are not forced to rotate together (e.g., can accommodate a relative rotation of 0.025 rad using connections from Part 10 of AISC Manual and meeting ductility checks in Part 9 of AISC Manual)
    - Option 2: Connection establishes upper limit on connection demand via flexural yielding of beam or column

Seismic Provisions Chapter F – Braced Frames

F2 Special Concentrically Braced Frames (SCBF)

- There are key detailing differences between OCBF and SCBF (continued)
  - Braces and columns required to be highly ductile members
  - Column splices must develop shear strength equal to $\Sigma M_{pc} / H_c$
- Redundancy requirements
  - Maintain a relative balance between tension and compression braces per Section F2.4a (e.g., not too many tension braces compared to compression braces)
  - Requirements for beams in V- or inverted-V configurations per Section F2.4b; similar to those for OCBF

Seismic Provisions Chapter F – Braced Frames

F.2 Special Concentrically Braced Frames (SCBF)

- Braced frames with members connected concentrically
  - Eccentricities less than beam depth OK if resulting connection and member forces are considered in the design and do not change expected inelastic deformation capacity

---

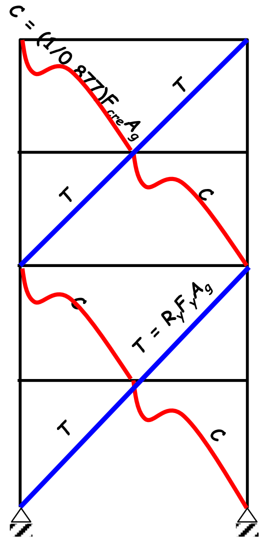
Seismic Provisions Chapter F – Braced Frames

For typical SCBF, this will require 2 analyses

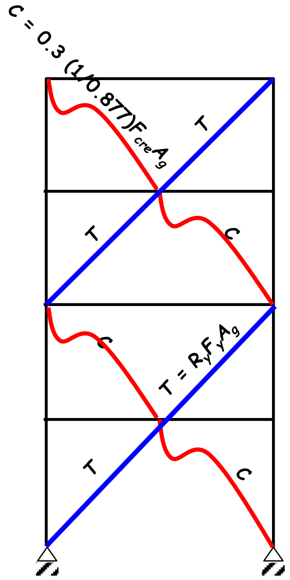
F2.3 SCBF - Analysis

- Required strength of beams, columns, struts and connections shall be determined using capacity-limited load effects
- Capacity-limited load effects, E_{cl}, taken as larger of the following:
  - Analysis (a): All braces produce forces corresponding to their **expected strength in compression or tension**
  - Analysis (b): All braces in **tension produce expected strength** and braces in compression **produce expected post-buckling strength**
  - For multi-tiered braced frames, perform analysis representing progressive yielding and buckling of braces from weakest to strongest. Consider both directions of frame loading.

Seismic Provisions Chapter F – Braced Frames



Analysis based on brace expected tension and compressive strengths



Analysis based on brace expected tension and post-buckling compression strength

Seismic Provisions Chapter F – Braced Frames

F2.3 SCBF - Analysis

- For these analyses:
  - Braces shall be in compression or tension neglecting effects of gravity loads
  - Analyses shall consider both directions of frame loading (i.e., asymmetry)
  - Expected strength in tension is $R_yF_yA_g$
  - Expected strength in compression may be taken as lesser of $R_yF_yA_g$ or (1/0.877)$F_{cre}A_g$
  - F_{cre} is based on F_{cr} from *Specification* Chapter E where R_yF_y is used in lieu of F_y

Seismic Provisions Chapter F – Braced Frames

F2.3 SCBF - Analysis

- For these analyses (continued):
  - Brace distance to determine F_{cre} may not exceed distance from brace end to brace end
  - Expected post-buckling brace strength shall not be taken as greater than 0.3 x (expected brace strength in compression)
- Exceptions:
  - Permitted to neglect flexural forces from seismic drift in these analyses
  - Required strength of columns need not exceed least of the following:
    - Forces corresponding to resistance of foundation uplift
    - Forces determined using nonlinear analysis

Example 5.3.2

SCBF Analysis

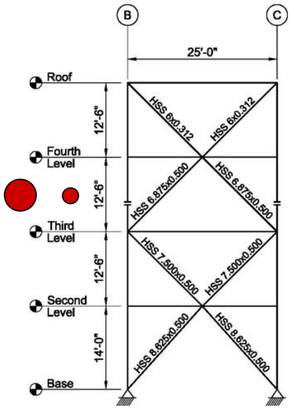
- Example illustrating application of SCBF analysis investigating full-strength and post-buckled brace forces
- Example worked in LRFD

Example 5.3.2 SCBF Analysis

Given:

Refer to the braced frame elevation and sizes shown in Figure 5-16. All braces are ASTM A500 Grade C round HSS. Perform an analysis to determine the expected strengths in tension and compression of the braces according to AISC *Seismic Provisions* Section F2.3.

The brace sizes in this example are the result of an analysis using seismic loads from ASCE 7. This example uses these sizes to determine demands on the horizontal beams, columns and column splices.



Example 5.3.2 SCBF Analysis

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C (round)
F_y = 46 ksi
F_u = 62 ksi

Example 5.3.2 SCBF Analysis

From AISC *Manual* Table 1-13, the geometric properties of the braces are:

HSS6.000×0.312
A = 5.22 in.^{2} r = 2.02 in.

HSS6.875×0.500
A = 9.36 in.^{2} r = 2.27 in.

HSS7.500×0.500
A = 10.3 in.^{2} r = 2.49 in.

HSS8.625×0.500
A = 11.9 in.^{2} r = 2.89 in.

Example 5.3.2 SCBF Analysis

The AISC *Seismic Provisions* recommend proportioning braces to their required strength. For the two-story-X configuration, it is efficient to minimize the required beam strength by coordinating the brace size used above and below the intersected beam (i.e., making them the same).

This was not done in this example to better illustrate the differences between of the two analyses.

Example 5.3.2 SCBF Analysis

According to AISC *Seismic Provisions* Section F2.3, the required strengths of columns, beams and connections are based on the load combinations in the applicable building code, where the seismic load effect with overstrength, E_{mh}, is based on the larger force determined from the following two analyses:

a. An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension
b. An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength

Example 5.3.2 SCBF Analysis

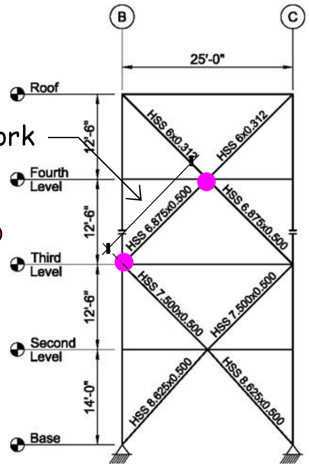
In order to study the effects of analyses (a) and (b) on the rest of the frame, the expected strengths in tension and compression and the post-buckling strength in compression must be determined for all the braces.

Example 5.3.2 SCBF Analysis

For determining the expected strength in compression, AISC *Seismic Provisions* Section F2.3 requires that the brace length used not exceed the distance from brace end-to-brace end. The work point-to-work point length of the typical brace above the base level is:

$$L = \sqrt{(12.5\text{ ft})^2 + (25\text{ ft}/2)^2}$$
$$= 17.7\text{ ft}$$

Typical story height is 12.5' and half-bay width is 25'/2 = 12.5'



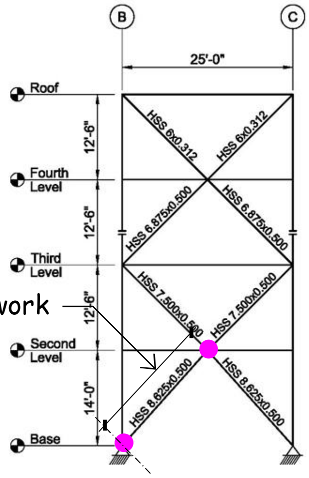
Work point-to-work point length

Example 5.3.2 SCBF Analysis

The work point-to-work point length of the brace at the base level is:

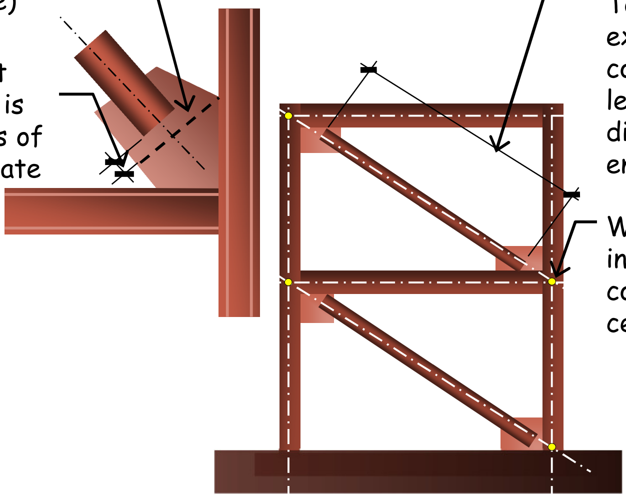
$$L = \sqrt{(14\text{ ft})^2 + (25\text{ ft}/2)^2}$$
$$= 18.8\text{ ft}$$

Story height is 14' and half-bay width is 25'/2 = 12.5'



Work point-to-work point length

Example 5.3.2 SCBF Analysis



Line of restraint (fold line)

2*t* offset (where *t* is thickness of gusset plate

To determine expected strength in compression, brace length may not exceed distance from brace-end to brace-end

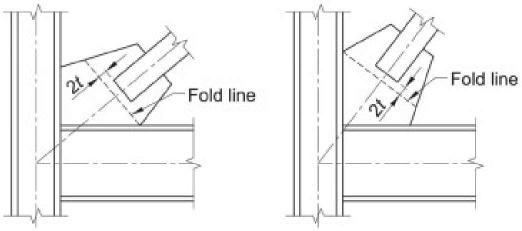
Work point: intersection of brace, column (and beam) centerlines

Length of Brace (Brace-End to Brace-End) for Calculating Compression Strength

Example 5.3.2 SCBF Analysis

The brace length will be less than work point distance because:

- Column and beam depth
- The gusset will accommodate brace buckling [AISC *Seismic Provisions* Section F2.6c.3(b)] by allowing a 2*t* clearance between the end of the brace and the line of restraint. AISC *Seismic Provisions* Commentary Figure C-F2.19 shows how the line of restraint is measured.



t = thickness of gusset plate

Example 5.3.2 SCBF Analysis

Example 5.3.8 verifies that the actual length of the brace is approximately 12 to 13 ft for the third- and fourth-level braces; therefore, use a length of 12 ft for determining the expected strength in compression for all braces. The brace lengths used in Table 5-2 could be modified once the connection length is known.

These revised lengths might require re-running the analyses using revised compression strengths

Example 5.3.2 SCBF Analysis

Tables 5-1 and 5-2 show the expected strengths in tension and the expected and post-buckling strengths in compression of all braces. A sample calculation is given for the HSS6.000×0.312, and a similar procedure is used to determine the strengths of the other braces.

From AISC *Seismic Provisions* Table A3.1:

R_y = 1.3 for ASTM A500 Grade C

Example 5.3.2 SCBF Analysis

From AISC *Seismic Provisions* Section F2.3, the expected strength of the brace in tension is:

$$\begin{aligned} P_t &= R_y F_y A_g \\ &= 1.3(46\text{ ksi})(5.22\text{ in.}^2) \\ &= 312\text{ kips} \end{aligned}$$

Example 5.3.2 SCBF Analysis

In compression, $R_y F_y$ is used in lieu of F_y for the determination of F_{cre} according to AISC *Seismic Provisions* Section F2.3. F_{cre} is determined from AISC *Specification* Chapter E using the equations for F_{cr}.

$$\begin{aligned} \frac{L_c}{r} &= \frac{1.0(12\text{ ft})(12\text{ in./ft})}{2.02\text{ in.}} \\ &= 71.3 \end{aligned}$$

L/r ratio based on brace-end to brace-end length

$$\begin{aligned} 4.71\sqrt{\frac{E}{R_y F_y}} &= 4.71\sqrt{\frac{29{,}000\text{ ksi}}{1.3(46\text{ ksi})}} \\ &= 104 \end{aligned}$$

Example 5.3.2 SCBF Analysis

Because 71.3 < 104, AISC *Specification* Equation E3-2 applies, and F_{cre} is determined as follows:

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \qquad \text{(}\textit{Spec}\text{. Eq. E3-4)}$$

$$= \frac{\pi^2 (29{,}000 \text{ ksi})}{(71.3)^2}$$

$$= 56.3 \text{ ksi}$$

Example 5.3.2 SCBF Analysis

$$F_{cre} = \left(0.658^{\frac{R_y F_y}{F_e}} \right) R_y F_y \qquad \text{(}\textit{Spec}\text{. Eq. E3-2)}$$

$$= \left[0.658^{\frac{1.3(46 \text{ ksi})}{(56.3 \text{ ksi})}} \right] (1.3)(46 \text{ ksi})$$

$$= 38.3 \text{ ksi}$$

Example 5.3.2 SCBF Analysis

From AISC *Seismic Provisions* Section F2.3, the expected strength of the brace in compression is permitted to be taken as the lesser of $R_yF_yA_g$ (= 312 kips) and $(1/0.877)F_{cre}A_g$:

$$\begin{aligned} P_c &= (1/0.877)F_{cre}A_g \\ &= (1/0.877)(38.3\text{ ksi})(5.22\text{ in.}^2) \\ &= 228\text{ kips} \end{aligned}$$

Example 5.3.2 SCBF Analysis

Therefore, the expected strength of the brace in compression is 228 kips.

Also from Section F2.3, the maximum post-buckling brace strength is 0.3 times the expected brace strength in compression.

Table 5-1
Expected Brace Strength in Tension

Brace Member	A	$R_yF_yA_g$
	in.^{2}	kips
HSS6.000×0.312	5.22	312
HSS6.875×0.500	9.36	560
HSS7.500×0.500	10.3	616
HSS8.625×0.500	11.9	712

Example 5.3.2 SCBF Analysis

Table 5-2
Expected Brace Strength and Post-Buckling
Brace Strength in Compression

Brace Member	$A=A_g$	r	Length	L_c/r	F_{cre}	Expected Strength in Compression $(1/0.877)F_{cre}A_g$	Expected Post-Buckling Strength in Compression $0.3[(1/0.877)F_{cre}A_g]$
	in.^{2}	in.	ft		ksi	kips	kips
HSS6.000×0.312	5.22	2.02	12	71.3	38.3	228	68.4
HSS6.875×0.500	9.36	2.27	12	63.4	42.1	449	135
HSS7.500×0.500	10.3	2.49	12	57.8	44.6	524	157
HSS8.625×0.500	11.9	2.89	12	49.8	48.1	653	196

Example 5.3.2 SCBF Analysis

The diagrams in Figures 5-17a and 5-17b show the forces imposed on the frame from buckling and yielding of the braces. For the analysis provisions of AISC *Seismic Provisions* F2.3(b), the expected strengths of the braces in compression shown in Figure 5-17a are multiplied by 0.3 (expected post-buckling brace strength) and shown in Figure 5-17b.

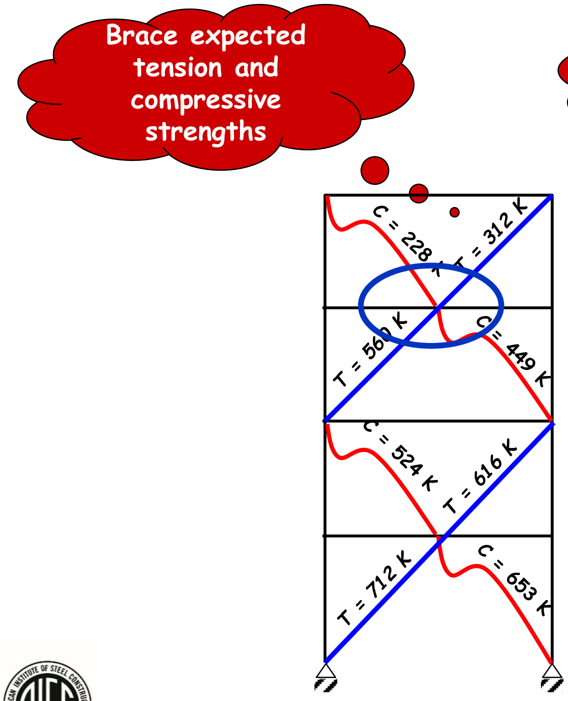

Brace expected tension and compressive strengths

Fig. 5-17a. Forces imposed on frame from braces according to AISC *Seismic Provisions* Section F2.3(a).

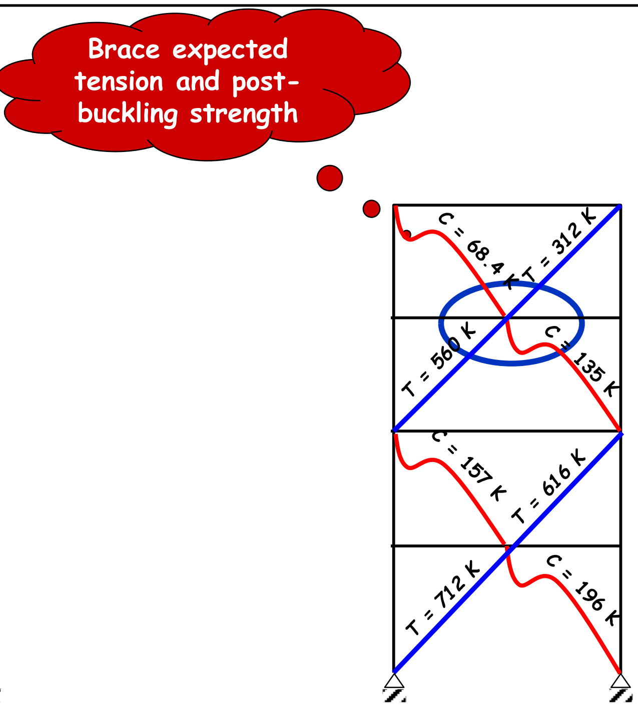

Brace expected tension and post-buckling strength

Fig. 5-17b. Forces imposed on frame from braces according to AISC *Seismic Provisions* Section F2.3(b).

Example 5.3.2
SCBF Analysis

Seismic Provisions Chapter F – Braced Frames

F2.4 SCBF – Lateral Force Distribution

- Along any line of braces, braces shall be deployed in alternate directions
  - For either direction of force parallel to bracing, at least 30% but no more than 70% of total horizontal force is resisted by tension braces
  - Unless available strength of each brace in compression is larger than required strength from load combinations in applicable building code, including overstrength

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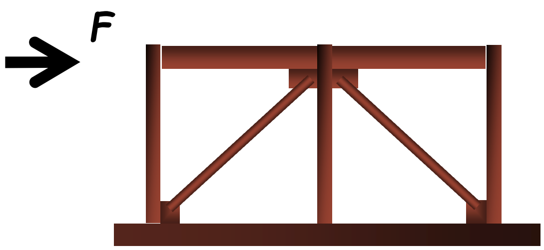
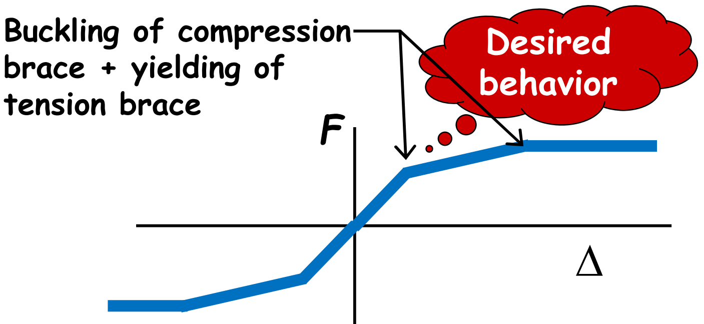
Seismic Provisions Chapter F – Braced Frames

F2.4 SCBF – Lateral Force Distribution



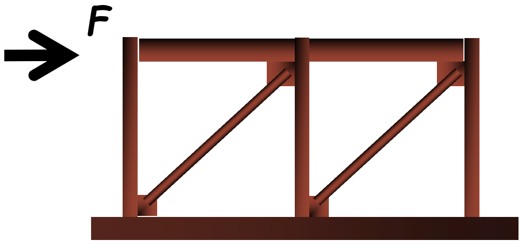
Braces oriented in alternate directions



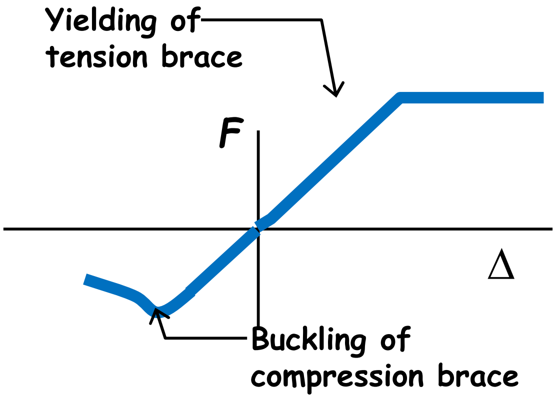
Symmetric

Seismic Provisions Chapter F – Braced Frames

F2.4 SCBF – Lateral Force Distribution



Braces oriented in same direction



Asymmetric

Example 5.3.1

SCBF Brace Design

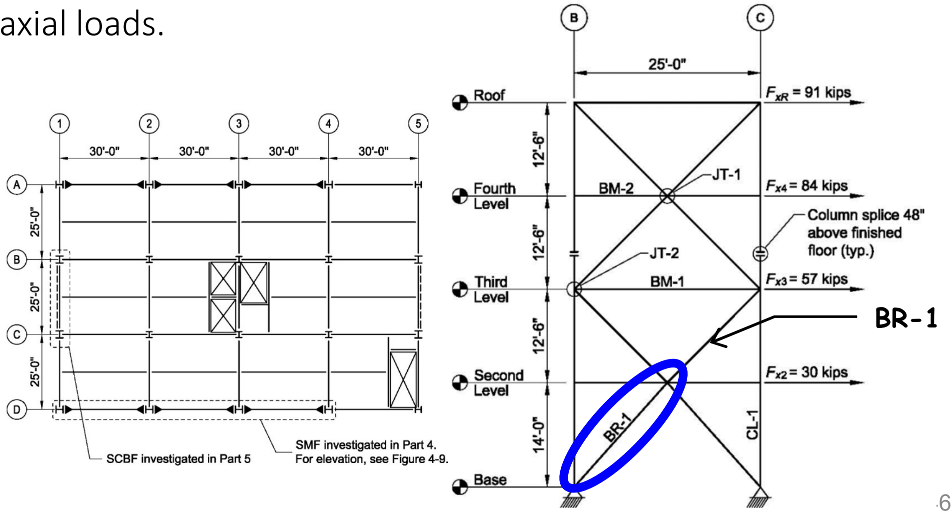
- Partial example illustrating application of tension and compression force distribution to the braces in the frame (see SDM page 5-127)
- Example worked in LRFD

Example 5.3.1 SCBF Brace Design

Given:

Refer to Brace BR-1 in Figure 5-15. Select an ASTM A500 Grade C round HSS to resist the following axial loads.

P_D = 18.0 kips
P_L = 9.50 kips
P_{QE} = ±197 kips



Example 5.3.1 SCBF Brace Design

The applicable building code specifies the use of ASCE/SEI 7 for calculation of loads. The axial force due to the snow load is negligible.

Relevant seismic design parameters were given in the SCBF Design Example Plan and Elevation section.

From an elastic analysis, the first-order interstory drift between the base and the second level is Δ_H = 0.200 in.

Example 5.3.1 SCBF Brace Design

Assume that the ends of the brace are pinned and braced against translation for both the *x*-*x* and *y*-*y* axes.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A500 Grade C (round)
F_y = 46 ksi
F_u = 62 ksi

Example 5.3.1 SCBF Brace Design

Required Strength
Determine the required strength

Considering the load combinations given in ASCE/SEI 7, the maximum compressive axial force in the diagonal brace, with E_v and E_h incorporated from Section 12.4.2, is determined as follows.

From Load Combination 6 from ASCE 7, Section 2.3.6:

$$\begin{aligned} P_u &= (1.2 + 0.2S_{DS})P_D + \rho P_{QE} + 0.5L + 0.2S \\ &= [1.2 + 0.2(1.0)](18.0\text{ kips}) + 1.3(197\text{ kips}) + 0.5(9.50\text{ kips}) + 0.2(0\text{ kips}) \\ &= 286\text{ kips} \end{aligned}$$

Example 5.3.1 SCBF Brace Design

The ASCE/SEI 7 load combination that results in the maximum **axial tensile force** in the diagonal brace, with E_v and E_h incorporated from Section 12.4.2, is:

Load Combination 7 from ASCE/SEI 7, Section 2.3.6:

$$\begin{aligned} P_u &= \left(0.9 - 0.2S_{DS}\right)P_D + \rho P_{QE} \\ &= \left[0.9 - 0.2(1.0)\right](18.0 \text{ kips}) + 1.3(-197 \text{ kips}) \\ &= -244 \text{ kips} \end{aligned}$$

Example 5.3.1 SCBF Brace Design

The unbraced length of the brace from work point-to-work point is:

$$\begin{aligned} L &= \sqrt{(14 \text{ ft})^2 + (25 \text{ ft}/2)^2} \\ &= 18.8 \text{ ft} \end{aligned}$$

This length has been determined by calculating the distance between the work points based on the intersection of the centerlines of the brace, column and beams. Shorter unbraced lengths of the brace may be used if justified by the engineer of record.

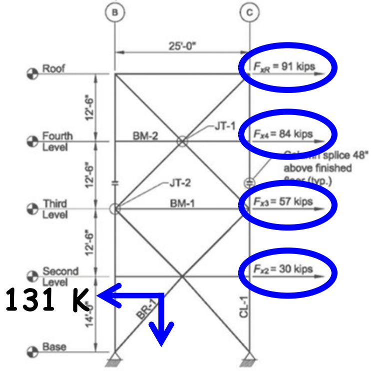
Example 5.3.1 SCBF Brace Design

AISC *Seismic Provisions* Section F2.4a requires that between 30% and 70% of the total horizontal force is resisted by braces in tension. From analysis, the total horizontal force in the line of the braced frame is 91 kips + 84 kips + 57 kips + 30 kips = 262 kips.

The horizontal component of the axial force due to earthquake force in Brace BR-1, when it is in tension is:

$$\left(\frac{25\text{ ft}/2}{18.8\text{ ft}}\right)\left|-197\text{ kips}\right| = 131\text{ kips}$$

P_{QE}



Example 5.3.1 SCBF Brace Design

which is 50% of the total horizontal force in the line of the braced frame.

Therefore, it meets the lateral force distribution requirements in AISC *Seismic Provisions* Section F2.4a.

Example 5.3.1
SCBF Brace Design

Example 5.3.4
SCBF Beam Design

- Partial example illustrating determination of demands on the horizontal beam in the SCBF analyzed in Example 5.3.2 (see SDM pg. 5-142)
- Example worked in LRFD

Example 5.3.4 SCBF Beam Design

Given:

Refer to Beam BM-2 in Figure 5-15. Select an ASTM A992 W-shape with a maximum depth of 36 in.

Design the beam as a noncomposite beam for strength, although the composite deck can be considered to brace the beam as discussed later in this example.

The applicable building code specifies the use of ASCE 7 for calculation of loads.

Example 5.3.4 SCBF Beam Design

Assume the brace sizes are as shown in Figure 5-16. Relevant seismic parameters were given in the SCBF Design Example Plan and Elevation section. The shears and moments on the beam due to gravity, assuming a simple span from column line B to C, are:

V_D = 11.2 kips V_L = 8.50 kips

M_D = 120 kip-ft M_L = 100 kip-ft

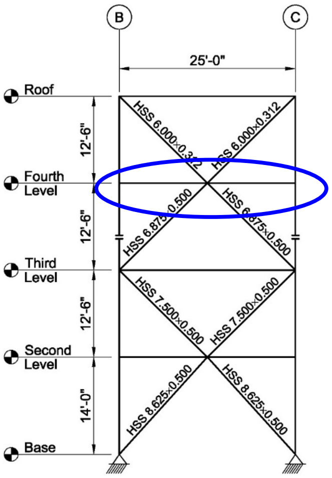
Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

ASTM A992

F_y = 50 ksi

F_u = 65 ksi



Example 5.3.4 SCBF Beam Design

As required by AISC *Seismic Provisions* Section F2.3, the required strength of the beam is determined using the capacity-limited load effect. The required strength is determined from the larger of:

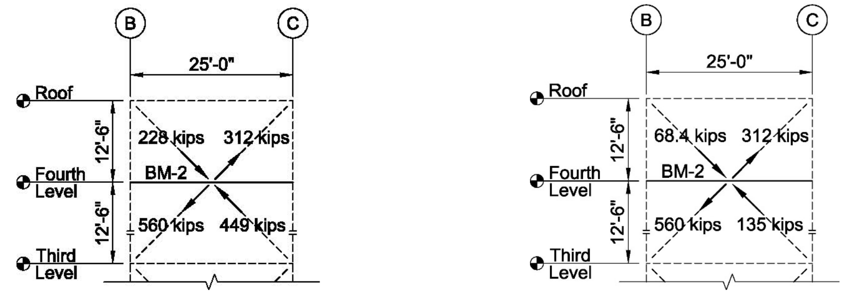
a. An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension

b. An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength

These analyses were performed in Example 5.3.2

Example 5.3.4 SCBF Beam Design

These brace required strengths are shown in Tables 5-1 and 5-2, and the forces acting on Beam BM-2 are shown in Figure 5-19.



(a) Forces from AISC Seismic Provisions Section F2.3(a)

(b) Forces from AISC Seismic Provisions Section F2.3(b)

Fig. 5-19. Forces acting on Beam BM-2 from a mechanism analysis of AISC *Seismic Provisions* Section F2.3 as carried out in Example 5.3.2.

Example 5.3.4 SCBF Beam Design

Required Strength
Determine the required axial strength of the beam based on AISC Seismic Provisions Section F2.3(a)

From AISC *Seismic Provisions* Section F2.3(a), the required axial strength of the beam is based on the braces at their expected strengths in tension and compression.

This is the first of two investigations to determine demands on the beam. The more demanding results govern the beam design.

Example 5.3.4 SCBF Beam Design

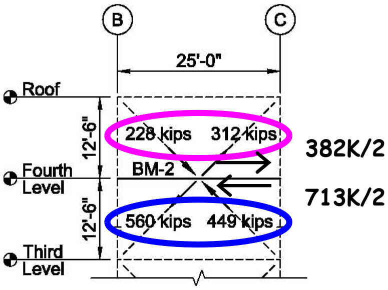
To determine the required axial force of the beam, the horizontal component of the difference between the sum of the expected strengths of the braces below the beam and the sum of the expected strengths of the braces above the beam can be thought of as a “story force” that the beam must deliver to the braces.

Because the braced frame is in the middle bay of a three-bay building, half of this story force can be considered to enter the braces from each side, and is carried by Beam BM-2 to the braces connected to the beam midspan. This force could act in either direction and is shown as positive. See next slide.

Example 5.3.4 SCBF Beam Design

$$P_{x3} = (560 \text{ kips} + 449 \text{ kips})\sin 45° = 713 \text{ kips}$$

$$P_{x4} = (228 \text{ kips} + 312 \text{ kips})\sin 45° = 382 \text{ kips}$$



The axial force on either side of the beam will be one-half of the difference:

$$P_x = 0.5(713 \text{ kips} - 382 \text{ kips}) = 166 \text{ kips}$$

Determination of net axial force in beam

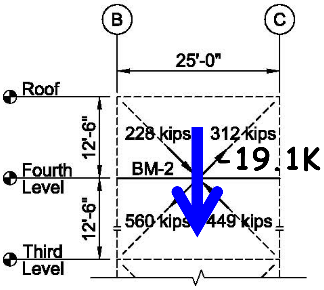
Example 5.3.4 SCBF Beam Design

The required axial strength due to brace forces is equal to this force:

$$P_{Emh} = P_x = 166 \text{ kips}$$

The “unbalanced” vertical force is determined from the vertical component of all four brace forces.

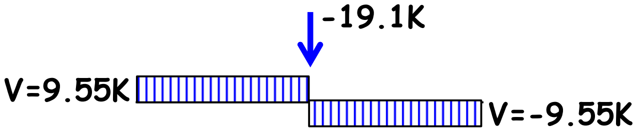
$$P_y = (\overset{\uparrow}{312 \text{ kips}} - \overset{\downarrow}{228 \text{ kips}} + \overset{\uparrow}{449 \text{ kips}} - \overset{\downarrow}{560 \text{ kips}})\cos 45° = \underset{\downarrow}{-19.1} \text{ kips}$$



Example 5.3.4 SCBF Beam Design

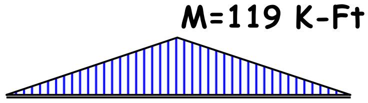
This unbalanced vertical force can be considered as a load acting downward at the midpoint of the beam, and produces the following shear and moment from the global beam analysis:

$$\begin{aligned} V_{Eg} &= \frac{-P_y}{2} \\ &= \frac{-(-19.1\text{ kips})}{2} \\ &= 9.55\text{ kips} \end{aligned}$$



Shear Diagram

$$\begin{aligned} M_{Eg} &= \frac{-P_y L}{4} \\ &= \frac{-(-19.1\text{ kips})(25\text{ ft})}{4} \\ &= 119\text{ kip-ft} \end{aligned}$$



Moment Diagram

Example 5.3.4 SCBF Beam Design

Note that the unbalanced vertical force from the braces is to be considered when evaluating member limit states in the beam. In the connection design presented in Example 5.3.7, beam local limit states are evaluated using internal forces determined in the brace connection design.

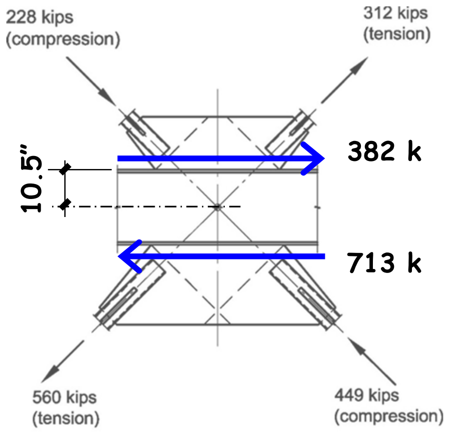
In combination with these overall member effects, the brace forces create localized shear and moment in the connection region (Fortney and Thornton, 2017). The local seismic moment due to the horizontal forces, M_{EL}, must be computed separately for the braces above and below and summed:

$$M_{EL} = \frac{(P_1 + P_2)\sin\theta e_b}{8}$$

Example 5.3.4 SCBF Beam Design

For evaluation here, the beam is assumed to be 21 in. deep. Therefore, e_b is 10.5 in.

$$M_{EL}=\frac{(P_1+P_2)_3\sin\theta e_b}{8}+\frac{(P_1+P_2)_4\sin\theta e_b}{8}$$
$$=\frac{\left[(P_1+P_2)_3\sin\theta+(P_1+P_2)_4\sin\theta\right]e_b}{8}$$
$$=\frac{(P_{x3}+P_{x4})e_b}{8}$$
$$=\frac{(713\text{ kips}+382\text{ kips})(10.5\text{ in.})}{8(12\text{ in./ft})}$$
$$=120\text{ kip-ft}$$



Example 5.3.4 SCBF Beam Design

For design purposes, the required flexural strength due to brace forces may be taken as the sum of M_{Eg} and M_{EL}. (An exact evaluation of this condition will show a somewhat smaller moment.)

$$M_{Emh}=M_{Eg}+M_{EL}$$
$$=119\text{ kip-ft}+120\text{ kip-ft}$$
$$=239\text{ kip-ft}$$

Summation of moments from unbalanced vertical force plus local forces at gusset above and below the beam

Example 5.3.4 SCBF Beam Design

The localized shear is V_{EL}:

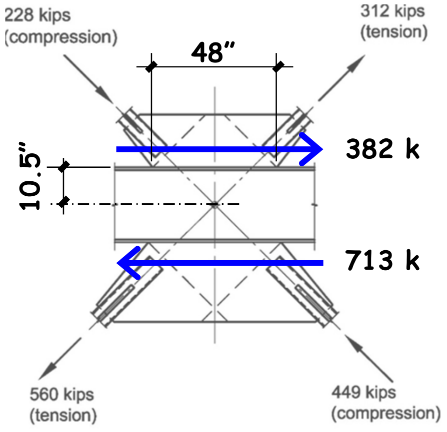
$$V_{EL} = \frac{2\left(P_1 + P_2\right)\sin\theta e_b}{L_g}$$

where L_g is the gusset length.

As with the localized moment, the localized shear from the braces above and below is additive. For evaluation here, the gussets above and below will be assumed to be 48 in. long, which is roughly one-sixth of the beam span.

Example 5.3.4 SCBF Beam Design

$$\begin{aligned}
V_{EL} &= \frac{2\left(P_1 + P_2\right)_3 \sin\theta e_b}{L_g} + \frac{2\left(P_1 + P_2\right)_4 \sin\theta e_b}{L_g} \\
&= \frac{2\left[\left(P_1 + P_2\right)_3 \sin\theta + \left(P_1 + P_2\right)_4 \sin\theta\right] e_b}{L_g} \\
&= \frac{2\left(P_{x3} + P_{x4}\right) e_b}{L_g} \\
&= \frac{2\left(713 \text{ kips} + 382 \text{ kips}\right)\left(10.5 \text{ in.}\right)}{48 \text{ in.}} \\
&= 479 \text{ kips}
\end{aligned}$$



This shear is not additive to the unbalanced shear computed previously, and the required shear strength is the larger of the two: V_{Emh} = 479 kips

Example 5.3.4 SCBF Beam Design

The following load combinations in ASCE/SEI 7 were found to govern. The required **axial strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(a) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$\begin{aligned} P_u &= (1.2 + 0.2S_{DS})P_D + P_{Emh} + 0.5P_L \\ &\quad + 0.2P_S \\ &= [1.2 + 0.2(1.0)](0 \text{ kips}) + 166 \text{ kips} \\ &\quad + 0.5(0 \text{ kips}) + 0.2(0 \text{ kips}) \\ &= 166 \text{ kips} \end{aligned}$$

Example 5.3.4 SCBF Beam Design

The required **shear strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(a) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$\begin{aligned} V_u &= (1.2 + 0.2S_{DS})V_D + V_{Emh} + 0.5V_L + 0.2V_S \\ &= [1.2 + 0.2(1.0)](0 \text{ kips}) + 479 \text{ kips} + 0.5(0 \text{ kips}) + 0.2(0 \text{ kips}) \\ &= 479 \text{ kips} \end{aligned}$$

Example 5.3.4 SCBF Beam Design

The shear due to gravity is zero at the beam midpoint for this loading, therefore, the required shear strength is due only to the local effect of the seismic forces.

The required **flexural strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(a) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$
\begin{aligned}
M_u &= \left(1.2 + 0.2S_{DS}\right)M_D + M_{Emh} + 0.5M_L + 0.2M_S \\
&= \left[1.2 + 0.2(1.0)\right](120 \text{ kip-ft}) + 239 \text{ kip-ft} + 0.5(100 \text{ kip-ft}) + 0.2(0 \text{ kip-ft}) \\
&= 457 \text{ kip-ft}
\end{aligned}
$$

Example 5.3.4 SCBF Beam Design

Determine the required axial strength of the beam based on AISC Seismic Provisions Section F2.3(b)

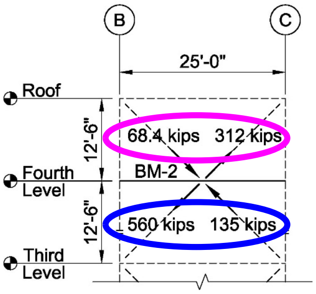
From AISC *Seismic Provisions* Section F2.3(b), the required axial strength of the beam is based on the braces at their expected strengths in tension and post-buckling strengths in compression. For this analysis, the expected strengths of the braces in compression must be multiplied by 0.3 to approximate their post-buckling strengths as shown in Table 5-2.

This is the second of two investigations to determine demands on the beam. The steps are the same as for the first investigation.

Example 5.3.4 SCBF Beam Design

To determine the required axial force of the beam, the horizontal component of the difference between the sum of the expected strengths of the braces below the beam and the sum of the expected strengths of the braces above the beam can be thought of as a "story force" that the beam must deliver to the braces. Because the braced frame is in the middle bay of a three-bay building, half of this story force can be considered to enter the braces from each side. See Figure 5-19(b).

$$\begin{aligned} P_{x3} &= (560\text{ kips} + 135\text{ kips})\sin 45° \\ &= 491\text{ kips} \\ P_{x4} &= (68.4\text{ kips} + 312\text{ kips})\sin 45° \\ &= 269\text{ kips} \end{aligned}$$



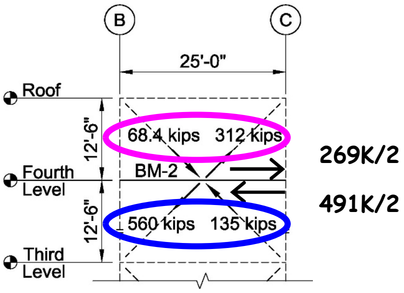
Example 5.3.4 SCBF Beam Design

The axial force in either side of the beam will be one-half of the difference:

$$\begin{aligned} P_x &= 0.5(491\text{ kips} - 269\text{ kips}) \\ &= 111\text{ kips} \end{aligned}$$

The required axial strength due to brace forces is equal to this force:

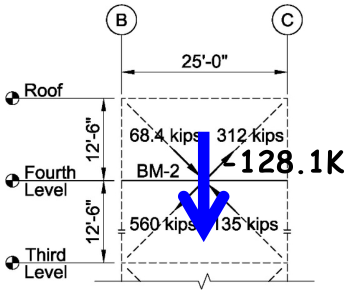
$$\begin{aligned} P_{Emh} &= P_x \\ &= 111\text{ kips} \end{aligned}$$



Example 5.3.4 SCBF Beam Design

The “unbalanced” vertical force is determined from the vertical component of all four brace forces.

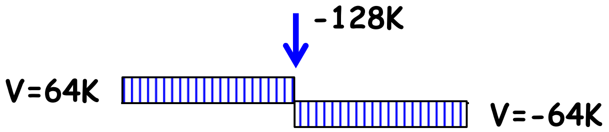
$$P_y = \left(\overset{\uparrow}{312\text{ kips}} - \overset{\downarrow}{68.4\text{ kips}} + \overset{\uparrow}{135\text{ kips}} - \overset{\downarrow}{560\text{ kips}}\right)\cos 45^\circ$$
$$= \underset{\downarrow}{-128\text{ kips}}$$



Example 5.3.4 SCBF Beam Design

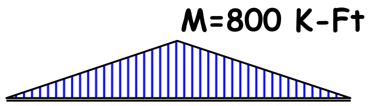
This unbalanced vertical force can be considered as a load acting downward on the beam, and produces the following shear and moment:

$$V_{Eg} = \frac{-P_y}{2} = \frac{-(-128\text{ kips})}{2} = 64.0\text{ kips}$$



Shear Diagram

$$M_{Eg} = \frac{-P_y L}{4} = \frac{-(-128\text{ kips})(25\text{ ft})}{4} = 800\text{ kip-ft}$$

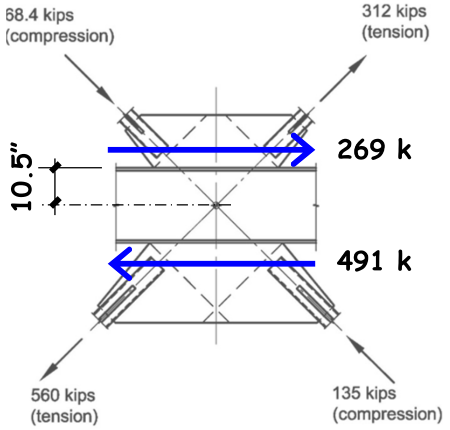


Moment Diagram

Example 5.3.4 SCBF Beam Design

The local connection moment is:

$$
\begin{aligned}
M_{EL} &= \frac{\left(P_{x3}+P_{x4}\right)e_b}{8} \\
&= \frac{\left(491\text{ kips}+269\text{ kips}\right)\left(10.5\text{ in.}\right)}{8\left(12\text{ in./ft}\right)} \\
&= 83.1\text{ kip-ft}
\end{aligned}
$$

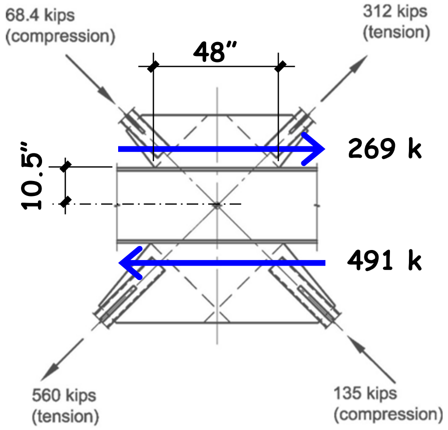


Example 5.3.4 SCBF Beam Design

For design purposes, the required flexural strength due to brace forces may be taken as the sum of M_{Eg} and M_{EL}.

$$
\begin{aligned}
M_{Emh} &= M_{Eg} + M_{EL} \\
&= 800\text{ kip-ft} + 83.1\text{ kip-ft} \\
&= 883\text{ kip-ft}
\end{aligned}
$$

Example 5.3.4 SCBF Beam Design



The localized shear is V_{EL}:

$$\begin{aligned} V_{EL} &= \frac{2\left(P_{x3}+P_{x4}\right)e_b}{L_g} \\ &= \frac{2\left(491\text{ kips}+269\text{ kips}\right)\left(10.5\text{ in.}\right)}{48\text{ in.}} \\ &= 333\text{ kips} \end{aligned}$$

This shear is not additive to the unbalanced shear computed previously, and the required shear strength is the larger of the two:

$$V_{Emh} = 333\text{ kips}$$

Example 5.3.4 SCBF Beam Design

Using the load combinations in ASCE/SEI 7, the required **axial strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(b) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$\begin{aligned} P_u &= \left(1.2+0.2S_{DS}\right)P_D + P_{Emh} + 0.5P_L + 0.2P_S \\ &= \left[1.2+0.2\left(1.0\right)\right]\left(0\text{ kips}\right)+111\text{ kips}+0.5\left(0\text{ kips}\right)+0.2\left(0\text{ kips}\right) \\ &= 111\text{ kips} \end{aligned}$$

Example 5.3.4 SCBF Beam Design

The required **shear strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(b) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$\begin{aligned} V_u &= \left(1.2+0.2S_{DS}\right)V_D + V_{Emh} + 0.5V_L + 0.2V_S \\ &= \left[1.2+0.2\left(1.0\right)\right]\left(0\text{ kips}\right) + 333\text{ kips} + 0.5\left(0\text{ kips}\right) + 0.2\left(0\text{ kips}\right) \\ &= 333\text{ kips} \end{aligned}$$

Example 5.3.4 SCBF Beam Design

As with the other condition analyzed, the shear due to gravity at the beam midpoint is zero.

The required **flexural strength** of Beam BM-2 according to the analysis requirements of AISC *Seismic Provisions* Section F2.3(b) is:

Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*), with the seismic load effects including overstrength incorporated from Section 12.4.3:

$$\begin{aligned} M_u &= \left(1.2+0.2S_{DS}\right)M_D + M_{Emh} + 0.5M_L + 0.2M_S \\ &= \left[1.2+0.2\left(1.0\right)\right]\left(120\text{ kip-ft}\right) + 883\text{ kip-ft} + 0.5\left(100\text{ kip-ft}\right) + 0.2\left(0\text{ kip-ft}\right) \\ &= 1{,}100\text{ kip-ft} \end{aligned}$$

Example 5.3.4 SCBF Beam Design

Note that the analysis of AISC *Seismic Provisions* Section F2.3(b), with the braces at post-buckling strength in compression, gives a significantly higher required moment for the beam and moderately lower required axial and shear forces.

The moment resulting from the analysis of Section F2.3(b) does not act simultaneously with the axial and shear forces resulting from Section F2.3(a).

Example 5.3.4 SCBF Beam Design

In summary, the required strength of Beam BM-2 determined by the analysis provisions of AISC *Seismic Provisions* Section F2.3(a) is:

$$P_u = 166 \text{ kips}$$
$$V_u = 479 \text{ kips}$$
$$M_u = 457 \text{ kip-ft}$$

The required strength of Beam BM-2 determined by the analysis provisions of AISC *Seismic Provisions* Section F2.3(b) is:

$$P_u = 111 \text{ kips}$$
$$V_u = 333 \text{ kips}$$
$$M_u = 1{,}100 \text{ kip-ft}$$

Example 5.3.4 SCBF Beam Design

These two sets of required strengths do not occur simultaneously, so the worst axial load, shear and moment are not used to design the beam.

Instead, a proposed beam (e.g., W21x147) is checked against the results from each analysis independently.

The balance of Example 5.3.4, starting on SDM pg. 5-150 but not shown here, illustrates checking the proposed beam-column against the two sets of demands.

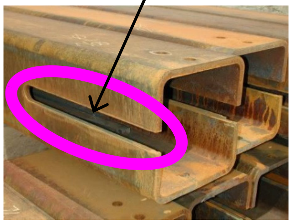
Example 5.3.4

SCBF Beam Design

Seismic Provisions Chapter F – Braced Frames

F2.5b Diagonal Braces - Effective Net Area

- Brace effective net area shall not be less than brace gross area
- Where reinforcement used:
  - F_y of reinforcement ≥ F_y of brace
  - Connections shall have sufficient strength to develop reinforcement on each side of reduced section



Reduced section created by slot to engage gusset plate

Plate added to each side to compensate for slot

Example 5.3.7

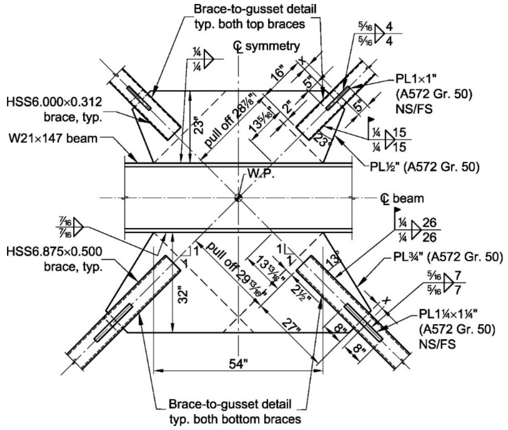
SCBF Brace-to-Beam Connection Design

- Partial example emphasizes the net section check and brace reinforcement design due to the slot in the brace for the gusset plate (see SDM pg. 5-220).
- The entire example begins on SDM pg. 5-175.
- Example worked in LRFD

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Top Brace-to-Gusset Connection

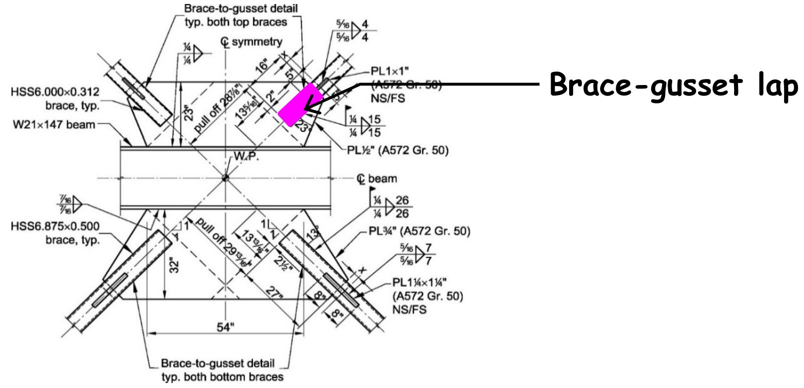
The required tensile strength of the connection is based upon $R_y F_y A_g$ of the braces as stipulated in AISC *Seismic Provisions* Section F2.6c.1. All limit states applicable to tension or compression in the brace must be checked.



Example 5.3.7 SCBF Brace-to-Beam Connection Design

Determine the minimum length, l, required for the brace-gusset lap

The limit state of shear rupture in the brace wall is used to determine the minimum brace-gusset lap length. Note that the expected brace rupture strength, $R_t F_u$, may be used in the determination of the available strength according to AISC *Seismic Provisions* Section A3.2.



Example 5.3.7 SCBF Brace-to-Beam Connection Design

Using AISC *Specification* Section J4.2, including R_t from AISC *Seismic Provisions* Table A3.1:

$$R_t = 1.2$$

$$R_n = 0.60R_tF_uA_{nv} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-4)}$$

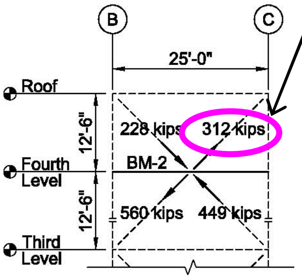
In this equation, A_{nv} is taken as the cross-sectional area of the four walls of the brace, $A_{nv} = 4lt_{des}$. Therefore:

$$\begin{aligned} R_n &= 0.60R_tF_u\left(4lt_{des}\right) \\ &= 0.60(1.2)(62 \text{ ksi})(4)(0.291 \text{ in.})l \\ &= (52.0 \text{ kip/in.})l \end{aligned}$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Setting the available shear rupture strength equal to the larger required tensile strength between the two braces ($P2$) and solving for the minimum lap length, l:

$$\begin{aligned} l &\geq \frac{P_u}{\phi(0.60)R_tF_u\left(4t_{des}\right)} \\ &\geq \frac{312 \text{ kips}}{0.75(52.0 \text{ kip/in.})} \\ &\geq 8.00 \text{ in.} \end{aligned}$$



This is the same result for both analyses

Note that this length is the minimum required for the limit state of shear rupture in the brace wall. A longer length may be used when designing the fillet welds between the brace and the gusset plate, if desired, to allow a smaller fillet weld size as is implemented in the following example.

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Size the weld between the brace and the gusset plate

The strength of fillet welds defined in AISC *Specification* Section J2 can be simplified, as explained in AISC *Manual* Part 8, to AISC *Manual* Equations 8-2a and 8-2b:

$$\phi R_n = (1.392 \text{ kip/in.}) D l_w$$

Based on the thickness of the thinner connected part, the minimum fillet weld size required by AISC *Specification* Table J2.4 is 3/16 in.

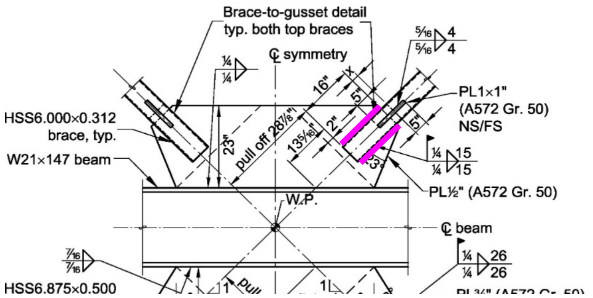
Example 5.3.7 SCBF Brace-to-Beam Connection Design

Try ¼” fillet welds for the four lines of weld, which can be made in a single pass:

$$l \geq \frac{P_u}{4(1.392 \text{ kip/in.})D}$$

$$\geq \frac{312 \text{ kips}}{4(1.392 \text{ kip/in.})(4 \text{ sixteenths})}$$

$$\geq 14.0 \text{ in.}$$



Use four 15-in.-long ¼” fillet welds to connect the braces above the beam to the gusset plate.

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Check block shear rupture of the gusset plate

The nominal strength for the limit state of block shear rupture relative to the axial load on the gusset plate is:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \qquad (\textit{Spec.}\ \text{Eq. J4-5})$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

where

$$\begin{aligned} A_{gv} &= (2\text{ planes})lt_p \\ &= (2\text{ planes})(15\text{ in.})\left(\frac{1}{2}\text{ in.}\right) \\ &= 15.0\text{ in.}^2 \\ A_{nt} &= Dt_p \\ &= (6.000\text{ in.})\left(\frac{1}{2}\text{ in.}\right) \\ &= 3.00\text{ in.}^2 \\ A_{nv} &= (2\text{ planes})lt_p \\ &= (2\text{ planes})(15\text{ in.})\left(\frac{1}{2}\text{ in.}\right) \\ &= 15.0\text{ in.}^2 \\ U_{bs} &= 1.0 \end{aligned}$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

and $R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$

$$= 0.60(65\text{ ksi})\left(15.0\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(3.00\text{ in.}^2\right)$$

$$\le 0.60(50\text{ ksi})\left(15.0\text{ in.}^2\right) + 1.0(65\text{ ksi})\left(3.00\text{ in.}^2\right)$$

$$= 780\text{ kips} > 645\text{ kips}$$

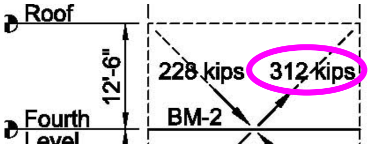
Therefore:

$$R_n = 645\text{ kips}$$

The available strength for the limit state of block shear rupture on the gusset plate is:

$$\phi R_n = 0.75(645\text{ kips})$$

$$= 484\text{ kips} > 312\text{ kips} \quad \textbf{o.k.}$$



Example 5.3.7 SCBF Brace-to-Beam Connection Design

Check upper brace effective net area

From AISC *Seismic Provisions* Section F2.5b(c), the brace effective net area, A_e, must not be less than the brace gross area, A_g:

$$A_n = A_g - 2\left[t_p + 2(gap)\right]t_{des}$$

This calculation is conservatively assumed to accommodate up to a 3/4-in.-thick plate A_n = 5.22 in.^{2} – 2 [3/4 in. + 2(1/16 in.)](0.291 in.). Using a gap of 1/16 in. on each side of the brace slot to allow clearance for erection:

Example 5.3.7 SCBF Brace-to-Beam Connection Design

From AISC *Specification* Table D3.1, Case 5, because $l > 1.3D$, $U = 1.0$, and the effective net area is:

$$\begin{aligned} A_e &= 1.0\left(4.71\text{ in.}^2\right) \\ &= 4.71\text{ in.}^2 \end{aligned}$$

Because $A_e < A_g$, brace reinforcement is required. The approximate required reinforcement area, A_{rn}, is the area removed, but the position of the reinforcement will reduce U to less than 1.0.

519

Example 5.3.7 SCBF Brace-to-Beam Connection Design

The required area of reinforcement can be obtained from:

$$\begin{aligned} \left(A_n + A_{rn}\right)U &= A_g \\ A_{rn} &= \frac{A_g}{U} - A_n \end{aligned}$$

Assuming a value of $U = 0.80$:

$$\begin{aligned} A_{rn} &= \frac{A_g}{0.80} - A_n \\ &= \frac{5.22\text{ in.}^2}{0.80} - 4.71\text{ in.}^2 \\ &= 1.82\text{ in.}^2 \end{aligned}$$

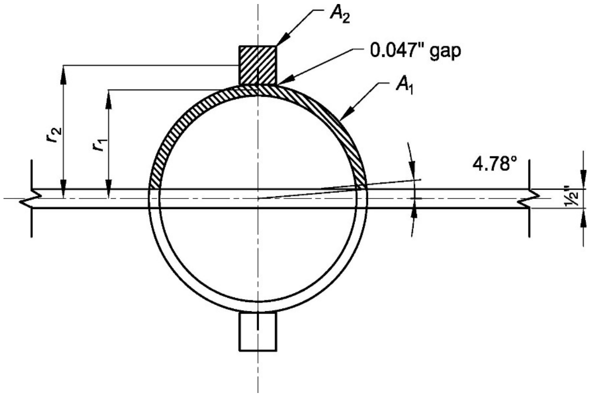
520

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Try two 1-in. × 1-in. flat bars, with a total area of A_{pn} = 2.00 in.^{2} AISC *Seismic Provisions* Section F2.5b(c)(1) requires that the specified minimum yield strength of the reinforcement be at least that of the brace; therefore, use ASTM A572 Grade 50 material for the flat bar. The cross-sectional geometry is shown in Figure 5-37.

$$
\begin{aligned}
r_1 &= \frac{D - t_{des}}{2} \\
&= \frac{6.000\text{ in.} - 0.291\text{ in.}}{2} \\
&= 2.85\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
r_2 &= \frac{D + t_{pr}}{2} \\
&= \frac{6.000\text{ in.} + 1\text{ in.}}{2} \\
&= 3.50\text{ in.}
\end{aligned}
$$



Example 5.3.7 SCBF Brace-to-Beam Connection Design

The distance to the centroid of a partial circle is given by:

$$\bar{x} = \frac{r_1 \sin\theta}{\theta}$$

where the total arc of the partial circle is 2θ, and θ is measured in radians.

Although the brace is slightly less than a full half-circle because of the slot as shown in Figure 5-37, use an angle, θ, of π/2 for simplicity. This is slightly unconservative for calculating the value of the shear lag factor, *U*. A more precise calculation could be performed using the exact angle.

Example 5.3.7 SCBF Brace-to-Beam Connection Design

$$\bar{x}_{brace} = (2.85\text{ in.})\left[\frac{\sin\,(\pi/2)\text{ rad}}{(\pi/2)\text{ rad}}\right]$$
$$= 1.81\text{ in.}$$
$$\bar{x}_{re} = r_2$$
$$= 3.50\text{ in.}$$

Determine $\bar{x}$ for the composite cross section.

Part	$\bar{x}$	A	A
	in.	in.^{2}	in.^{3}
Half of brace	1.81	2.36	4.27
One flat bar	3.50	1.00	3.50
Σ	–	3.36	7.77

Example 5.3.7 SCBF Brace-to-Beam Connection Design

$$\bar{x} = \frac{\Sigma \bar{x} A}{\Sigma A}$$
$$= \frac{7.77\text{ in.}^3}{3.36\text{ in.}^2}$$
$$= 2.31\text{ in.}$$

From AISC *Specification* Table D3.1, Case 2, which applies to round HSS with reinforcement added:

$$U = 1 - \frac{\bar{x}}{l}$$
$$= 1 - \frac{2.31\text{ in.}}{15\text{ in.}}$$
$$= 0.846$$

Specifications Table D3.1

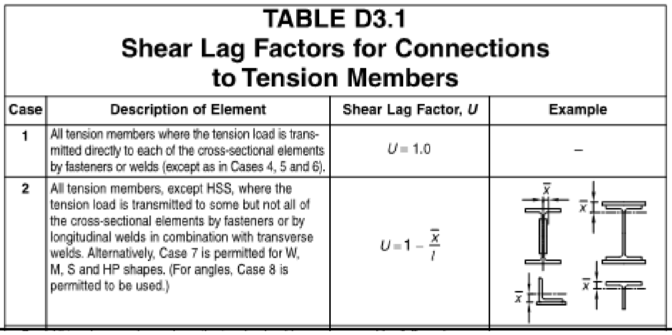


TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element	Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).	$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)	$U = 1 - \frac{\bar{x}}{l}$	

Example 5.3.7 SCBF Brace-to-Beam Connection Design

$$
\begin{aligned}
A_n &= A_{n(brace)} + A_{pn} \\
&= 4.71 \text{ in.}^2 + 2(1 \text{ in.})(1 \text{ in.}) \\
&= 6.71 \text{ in.}^2
\end{aligned}
$$

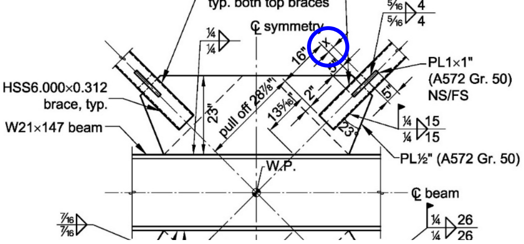
$$
\begin{aligned}
A_e &= UA_n \\
&= 0.846\left(6.71 \text{ in.}^2\right) \\
&= 5.68 \text{ in.}^2 > 5.22 \text{ in.}^2 \quad \textbf{o.k.}
\end{aligned}
$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Design welds connecting flat bars to brace

According to AISC *Seismic Provisions* Section F2.5b(c)(2), the flat bar must be connected to the pipe brace to develop the expected strength of the flat bar on each side of the reduced section (the expected yield strength, R_yF_y, is used here).

The reduced section is the length of the HSS from the extent of the slot (dimension *x* in Figure 5-25) to the start of the HSS-to-gusset weld.



Example 5.3.7 SCBF Brace-to-Beam Connection Design

The required strength of the weld is based on the expected flat bar yield strength, using R_y from AISC *Seismic Provisions* Table A3.1 for ASTM A572 Grade 50 bars. The expected strength of the flat bar reinforcement is:

$$\begin{aligned}\frac{R_y F_y A_{fb}}{\alpha_s} &= \frac{1.1(50\text{ ksi})(1.00\text{ in.}^2)}{1.0}\\ &= 55.0\text{ kips}\end{aligned}$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

There is a small gap of approximately 0.047 in. between the face of the pipe brace and the edge of the flat bar, as indicated in Figure 5-37. Because this is less than 1/16 in., it can be neglected according to AWS D1.1, clause 5.21.1. A single-pass 5/16-in. fillet weld can be used.

With two welds, the length of 5/16-in. fillet welds connecting the flat bar to the brace is determined from AISC *Manual* Equations 8-2a and 8-2b as follows:

$$\begin{aligned}l_w &\geq \frac{55.0\text{ kips}}{2(1.392\text{ kip/in.})(5\text{ sixteenths})}\\ &\geq 3.95\text{ in.}\end{aligned}$$

Example 5.3.7 SCBF Brace-to-Beam Connection Design

Use a 1-in. × 1-in. flat bar with 4-in.-long 5/16-in. fillet welds; the detail extends past both sides of the reduced section of the brace by 5 in.

The flat bar fillet weld develops the expected strength of the bar on each side of the end of the brace slot. The brace slot may be longer than the slot length by a maximum erection clearance of *x* in. (see Figure 5-25), as determined by the fabricator. The length of the flat bar will be 5 in. + 5 in. + *x* in. = 10 in. + *x* in.

Example 5.3.7

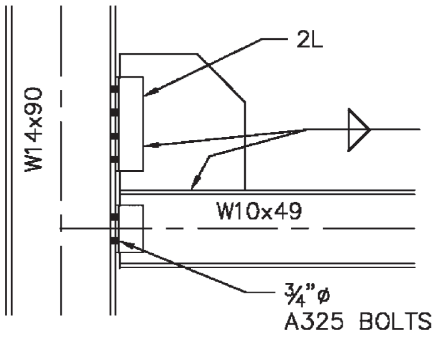
SCBF Brace-to-Beam Connection Design

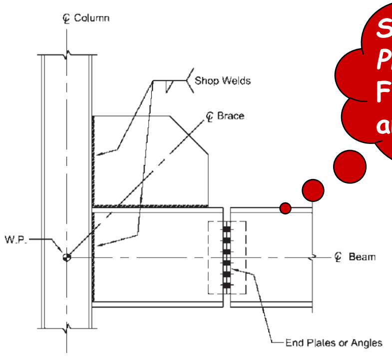
Seismic Provisions Chapter F – Braced Frames

F2.6b Beam-to-Column Connections

Where brace or gusset plate connects to both beam and column, conform to one of the following:

1. Connection shall be “simple” per *Specification* Section B3.6a (required rotation = 0.025 rad.)





***See Seismic Provisions* Figures C-F2.7 and C-F2.8**

Seismic Provisions Chapter F – Braced Frames

Beam-to-Column Connections

Where brace or gusset plate connects to both beam and column, conform to one of the following (continued):

2. Designed to resist moment equal to lesser of 1.1 x beam expected flexural strength dived by α_s (i.e., $1.1R_yF_yZ/\alpha_s$) or sum of column expected flexural strength divided by α_s (i.e., $\Sigma 1.1R_yF_yZ/\alpha_s$)

(Moments from (1) and (2) shall be considered in combination with required strength of brace and beam connection, including overstrength connector forces)

Example 5.3.9

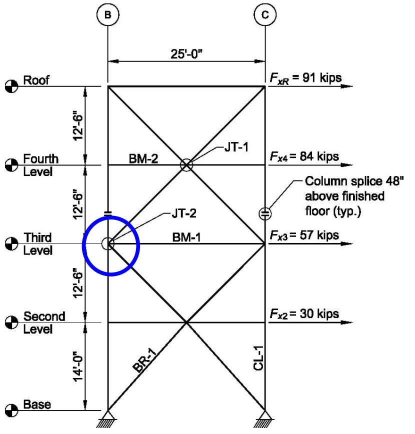
SCBF Brace-to-Beam/Column Connection Design

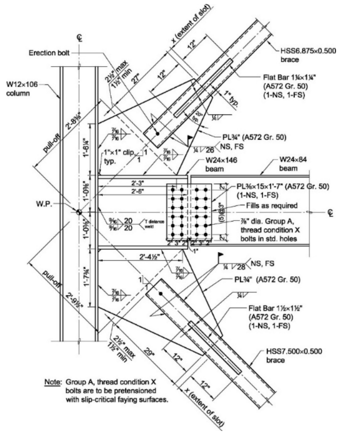
- Partial example illustrates determination of forces required for design of the brace to beam/column connection (see SDM pg. 5-259).
- Example worked in LRFD

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Given:

Refer to Joint JT-2 at Level 3 in Figure 5-15. Design the connection between braces, beam and column using splices in the beam away from the gusset plates. The brace is designed to buckle out-of-plane.





Note: Group A, thread condition X bolts are to be pretensioned with slip-critical faying surfaces.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Use ASTM A572 Grade 50 welded gusset plates concentric to the braces and 70-ksi electrodes to connect the braces to the gusset plates and the gusset plates to the beam and column.

As designed in previous examples, the braces are ASTM A500 Grade C round HSS sections, the column is an ASTM A913 Grade 65 W12×106, and the beam is an ASTM A992 W24×84.

The brace reinforcing bars are ASTM A572 Grade 50 material. As noted in Example 5.3.5, this connection uses ASTM A572 Grade 50 splices in the beam away from the connection. ASTM A992 W24×146 beam stubs are used at the beam ends to meet the high shear demand from the braces over the connection.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Use Group A bolts with threads excluded from the shear plane (thread condition X) and 70-ksi weld electrodes. The applicable building code specifies the use of ASCE/SEI 7 for calculation of loads. The shears and moments on the beam due to gravity are:

V_D = 11.2 kips V_L =8.50 kips M_D = 120 kip-ft M_L =100 kip-ft

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Solution:

This connection design uses splices in the beam to provide a simple beam-to-column connection satisfying AISC *Seismic Provisions* Section F2.6b(a).

From AISC *Manual* Tables 2-4 and 2-5, the material properties are as follows:

ASTM A572 Grade 50
F_y = 50 ksi
F_u = 65 ksi

ASTM A500 Grade C (round)
F_y = 46 ksi
F_u = 62 ksi

ASTM A913 Grade 65
F_y = 65 ksi
F_u = 80 ksi

ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

From AISC *Manual* Tables 1-1 and 1-13, the geometric properties are as follows:

Brace (above the beam)
HSS6.875×0.500
A = 9.36 in.^{2} D = 6.875 in. t_{des} = 0.465 in. r = 2.27 in.

Brace (below the beam)
HSS7.500×0.500
A = 10.3 in.^{2} D = 7.500 in. t_{des} = 0.465 in. r = 2.49 in.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

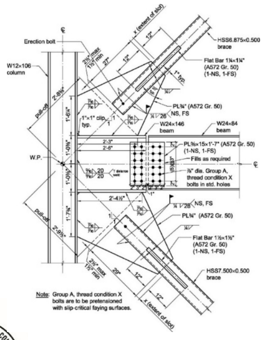
Beam
W24×84
d = 24.1 in. t_w = 0.470 in. t_f = 0.770 in. k_{des} = 1.27 in.

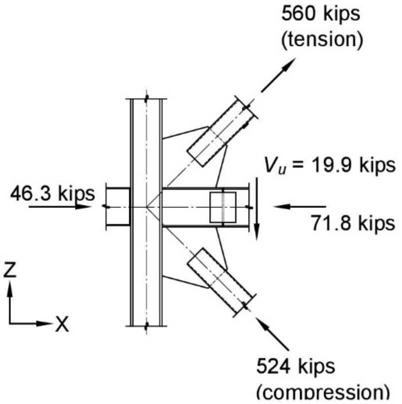
Beam stub
W24×146
A = 43.0 in.^{2} d = 24.7 in. t_w = 0.650 in. t_f = 1.09 in.
k_{des} = 1.59 in. T = 20 in. r_y = 3.01 in.

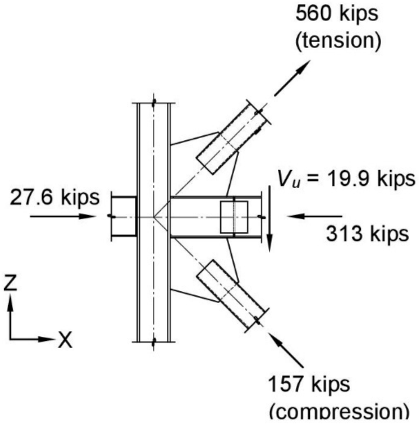
Column
W12×106
d = 12.9 in. t_w = 0.610 in. t_f = 0.990 in. k_{des} = 1.59 in.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

The complete connection design is shown in Figure 5-50. The connection geometry and member forces are as shown in Figures 5-51 and 5-52. These were determined in Examples 5.3.2 and 5.3.5.







Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

In Example 5.3.8, there were two braces above the beam and two braces below, so the direction of loading did not affect the connection design. In this corner connection, because the braces above and below the beam are not the same size, the direction of loading affects the amount of force that must be considered in the connection design. Two design cases will be considered.

AISC *Seismic Provisions* Sections F2.3(a) and F2.3(b) define the two mechanism analyses that must be considered in determining the required strength of beams, columns and connections. AISC *Seismic Provisions* Section F2.6c specifies the required strength of brace connections.

For this SCBF connection example, the requirements of AISC *Seismic Provisions* Section F2.3 will be used.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the expected ***tensile*** *strength of the HSS6.875×0.500 brace* ***above*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in tension, based on the expected strength, is:

$$\begin{aligned} P_u &= \frac{560\text{ kips}}{\alpha_s} \\ &= \frac{560\text{ kips}}{1.0} \\ &= 560\text{ kips} \end{aligned}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the expected ***compressive*** *strength of the HSS6.875×0.500 brace* ***above*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in compression, based on the expected strength, is:

$$
\begin{aligned}
P_u &= \frac{449\text{ kips}}{\alpha_s} \\
&= \frac{449\text{ kips}}{1.0} \\
&= 449\text{ kips}
\end{aligned}
$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the ***post-buckling compressive*** *strength of the HSS6.875×0.500 brace* ***above*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in compression, based on post-buckling strength, is:

$$
\begin{aligned}
P_u &= \frac{135\text{ kips}}{\alpha_s} \\
&= \frac{135\text{ kips}}{1.0} \\
&= 135\text{ kips}
\end{aligned}
$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the ***expected tensile*** *strength of the HSS7.500×0.500 brace* ***below*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in tension, based on the expected strength, is:

$$\begin{aligned} P_u &= \frac{616 \text{ kips}}{\alpha_s} \\ &= \frac{616 \text{ kips}}{1.0} \\ &= 616 \text{ kips} \end{aligned}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the expected ***compressive*** *strength of the HSS7.500×0.500 brace* ***below*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in compression is:

$$\begin{aligned} P_u &= \frac{524 \text{ kips}}{\alpha_s} \\ &= \frac{524 \text{ kips}}{1.0} \\ &= 524 \text{ kips} \end{aligned}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the ***post-buckling compressive*** *strength of the HSS7.500×0.500 brace* ***below*** *the beam*

From Example 5.3.2, the required strength of the brace connection when the brace is in compression, based on post-buckling strength, is:

$$\begin{aligned} P_u &= \frac{157\text{ kips}}{\alpha_s} \\ &= \frac{157\text{ kips}}{1.0} \\ &= 157\text{ kips} \end{aligned}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the required axial strength of the beam based on AISC Seismic Provisions Section ***F2.3(a)***

From AISC *Seismic Provisions* Section F2.3(a), the required axial strength of the beam is based on the braces at their expected strengths in tension and compression. To determine the required axial force on the beam, the horizontal component of the difference between the sum of the expected strengths of the braces below the beam and the sum of the expected strengths of the braces above the beam can be thought of as a “story force.”

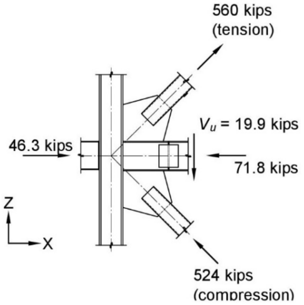
Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

The story force for the analysis in AISC *Seismic Provisions* Section F2.3(a), with tension and compression braces at their expected strengths, is:

$$P_x = \begin{bmatrix} \Sigma(\text{Forces below beam}) \\ -\Sigma(\text{Forces above beam}) \end{bmatrix} \sin 45^\circ$$

$$= \begin{bmatrix} (524\text{ kips} + 616\text{ kips}) \\ -(560\text{ kips} + 449\text{ kips}) \end{bmatrix} \sin 45^\circ$$

$$= 92.6\text{ kips}$$



Required strength of brace connection (compression) = 449 k

Required strength of brace connection (tension) = 616 k

Because the braced frame is in the middle of a three-bay building, half of this story force enters the braced bay from each side.

$$P_u = (92.6\text{ kips})/2$$

$$= 46.3\text{ kips}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Determine the required axial strength of the beam based on AISC Seismic Provisions Section ***F2.3(b)***

From AISC *Seismic Provisions* Section F2.3(b), the required axial strength of the beam is based on the braces at their expected strengths in tension and post-buckling strengths in compression. To determine the required axial force on the beam, the horizontal component of the difference between the sum of the expected strengths of the braces below the beam and the sum of the expected strengths of the braces above the beam can be thought of as a “story force.”

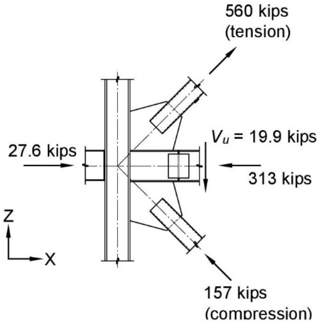
Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

The story force for the analysis in AISC *Seismic Provisions* Section F2.3(b), with tension braces at their expected strengths and compression braces at their post-buckling strengths, is:

$$P_x = \begin{bmatrix} \Sigma(\text{Forces below beam}) \\ -\Sigma(\text{Forces above beam}) \end{bmatrix} \sin 45^\circ$$

$$= \begin{bmatrix} (157\text{ kips} + 616\text{ kips}) \\ -(560\text{ kips} + 135\text{ kips}) \end{bmatrix} \sin 45^\circ$$

$$= 55.2\text{ kips}$$



Required strength of brace connection (compression) = 135 k

Required strength of brace connection (tension) = 616 k

Because the braced frame is in the middle of a three-bay building, half of this story force enters the braced bay from each side.

$$P_u = (55.2\text{ kips})/2$$
$$= 27.6\text{ kips}$$

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

From Example 5.3.5, the required axial strength of the beam is:

P_u = 313 kips

Determine the required shear strength of the beam

There is no shear in the beam due to seismic loads. From Example 5.3.5, the required shear strength of the beam due to gravity loads is:

V_u = 19.9 kips

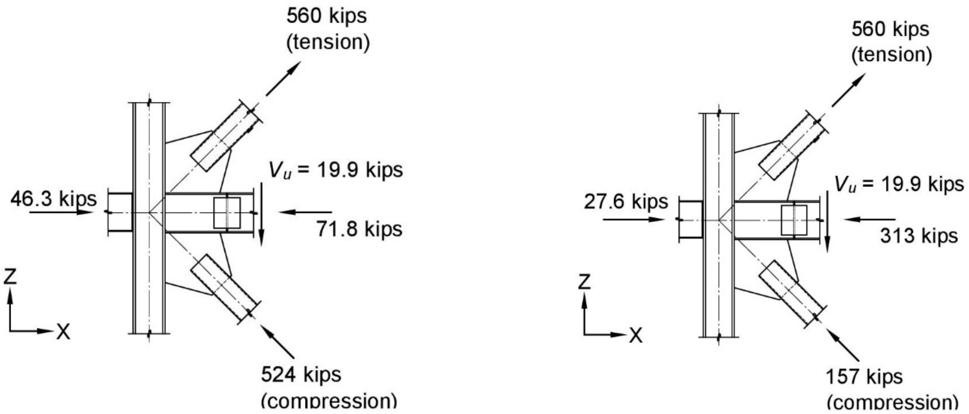
Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Design Case I

Design Case I, shown in Figure 5-51, consists of brace strengths that correspond to lateral forces applied in the positive *x*-direction. The braces forces above and below the beam must be considered simultaneously, including the expected strength in tension, the expected strength in compression, and the post-buckling compressive strength.

The two sets of forces to be considered in Design Case I are shown in Figures 5-51 and 5-52.

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design



Design Case I.

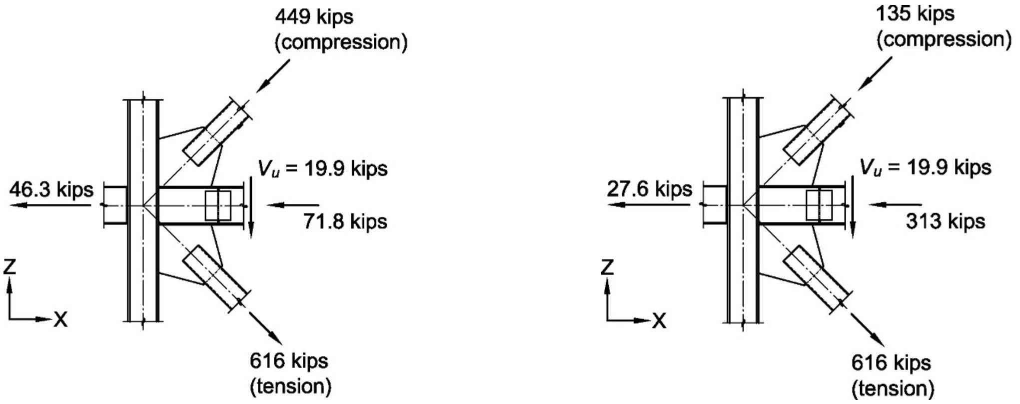
Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design

Design Case II

Design Case II shows brace strengths corresponding to lateral forces applied in the negative *x*-direction. The brace above the beam is at its expected strength (or post-buckling strength) in compression, and the brace below the beam is at its expected strength in tension. These forces must be considered simultaneously.

The two sets of forces to be considered in Design Case II are shown in Figures 5-53 and 5-54 (also see Figures 5-21 and 5-22 of Example 5.3.5).

Example 5.3.9 SCBF Brace-to-Beam/Column Connection Design



Design Case II.

Example 5.3.9

SCBF Brace-to-Beam/Column Connection Design

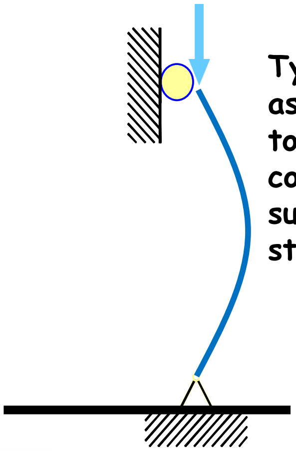
Seismic Provisions Chapter F – Braced Frames

F4 Buckling Restrained Braced Frames (BRBF)

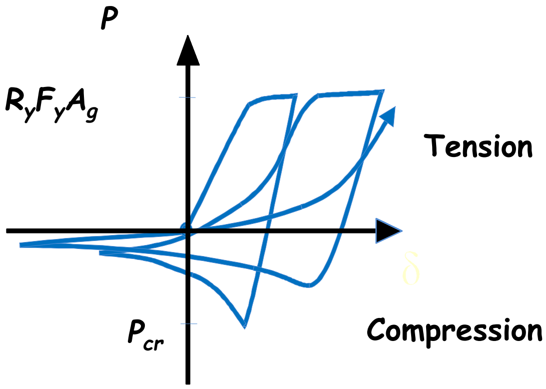
- BRBF are expected to withstand significant inelastic deformation (R = 8) in the links when subjected to the design earthquake.
- Bracing members shall be composed of a structural core and a system that retrains steel core from buckling
- Braces shall be designed, tested and detailed to accommodate expected deformations
- Expected deformations are minimum 2% story drift or 2 x (design story drift), whichever is larger, in addition to brace deformations
- BRBF to be designed so that inelastic deformations under design earthquake occur as brace yielding in compression or tension

Seismic Provisions Chapter F – Braced Frames

F4 Buckling Restrained Braced Frames (BRBF)



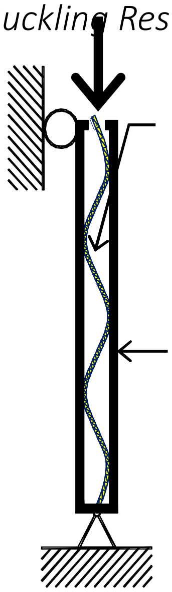
Typical brace behavior is asymmetric with respect to tension and compression and is subject to strength and stiffness degradation



Conventional brace behavior

Seismic Provisions Chapter F – Braced Frames

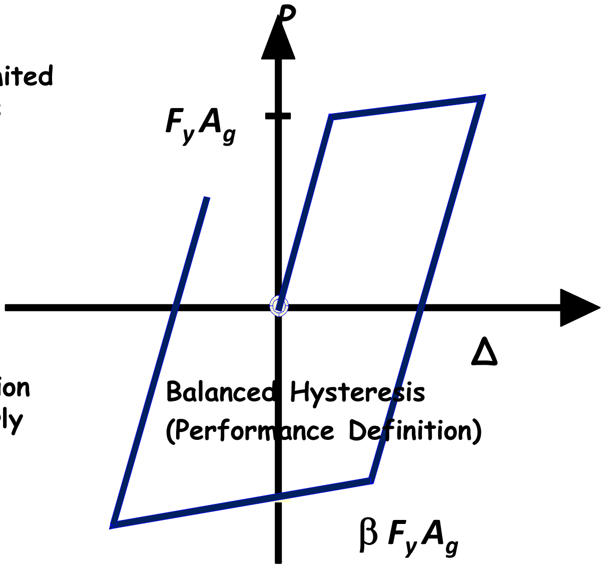
F4 Buckling Restrained Braced Frames (BRBF)



Compression buckling of limited capacity tension member is resisted by "sleeve"

"Sleeve"

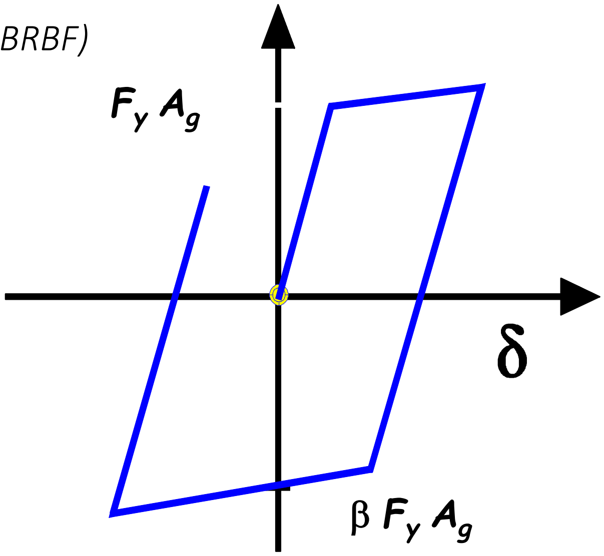
Compression and tension performance are nearly identical



Seismic Provisions Chapter F – Braced Frames

F4 Buckling Restrained Braced Frames (BRBF)

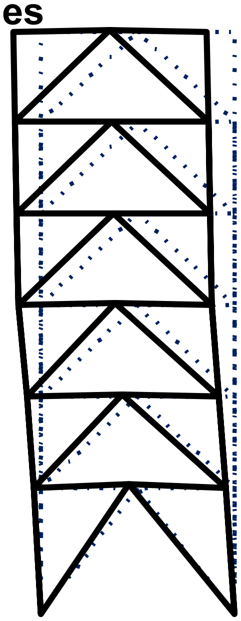
- Advantages of BRBF
  - Balanced Hysteresis
  - Slightly Stronger in Compression
  - Hysteretic Energy Dissipation
  - Hysteretic Stability
  - Strength
  - Stiffness
  - Long Fracture Life



Seismic Provisions Chapter F – Braced Frames

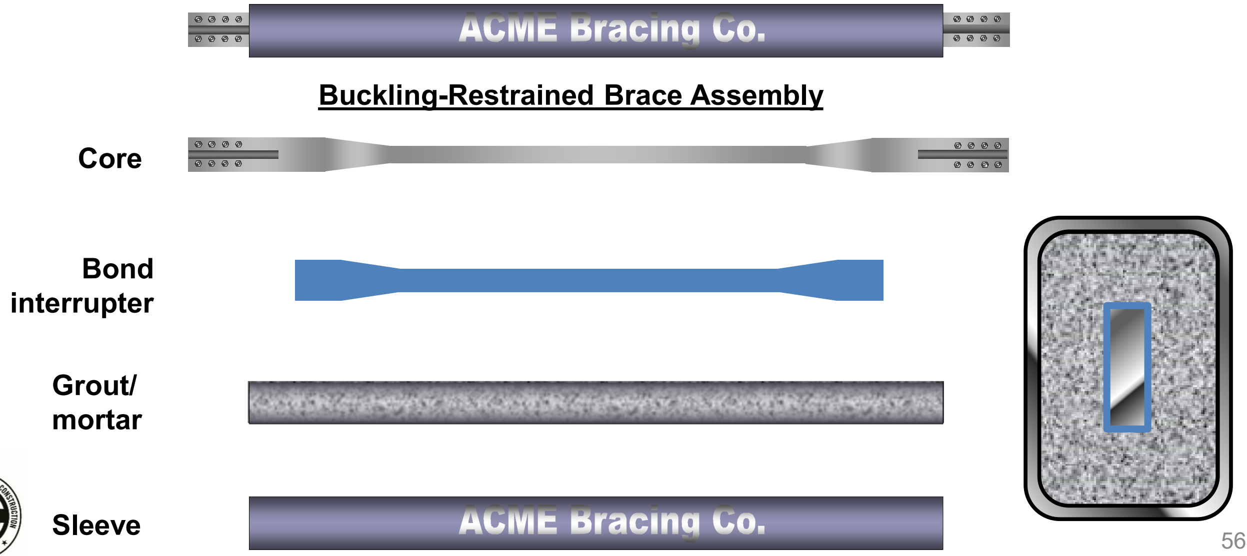
F4 Buckling Restrained Braced Frames (BRBF)

- Performance Advantages of BRBF
  - Story Mechanisms Uncommon
    - Fine-Tuning of Sizes
    - No Story Degradation
    - Distributed Yielding
  - Reduced Drift
  - Reduced Chevron Problems



Seismic Provisions Chapter F – Braced Frames

F4 Buckling Restrained Braced Frames (BRBF)



Buckling-Restrained Brace Assembly

Core

Bond interrupter

Grout/ mortar

Sleeve

Seismic Provisions Chapter F – Braced Frames

F4.2 BRBF – Basis of Design: Braces

- Adjusted brace strength in **compression**: $\beta\omega R_y P_{ysc}$

  where: β = compression adjustment factor ≥ 1.0

  ω = strain hardening adjustment factor (ratio of maximum tension force to $R_y P_{ysc}$)

  P_{ysc} = axial yield strength of steel core

- Adjusted brace strength in **tension**: $\omega R_y P_{ysc}$
- Note: R_y need not be applied if P_{ysc} established using coupon tests

Example 5.5.1
BRBF Brace Design

- Complete example illustrates determination of forces required for design of the brace and confirm deformation capability (see SDM pg. 5-492).
- Example worked in LRFD

Example 5.5.1 BRBF Brace Design

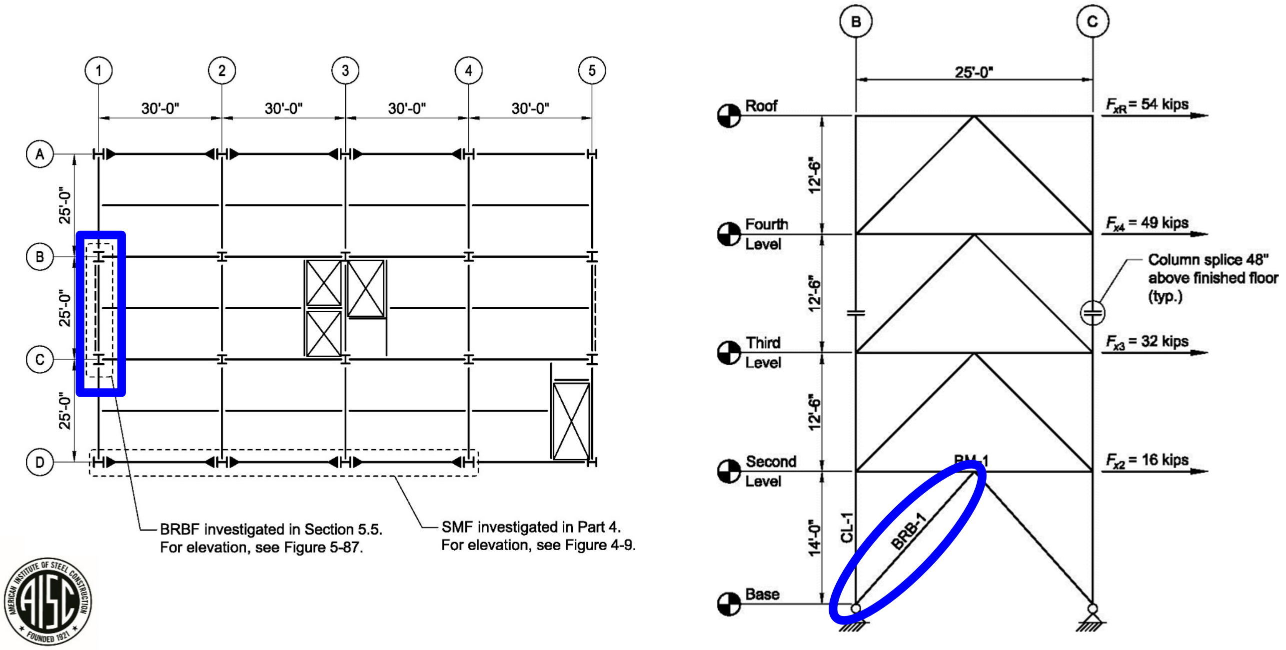
Given:
Refer to Brace BRB-1 in Figure 5-87.

Frame configurations and preliminary loads have been sent to a BRB manufacturer, and the elastic stiffness of the braces have been found to be 1.5 times higher than the stiffness of the yielding core area alone, if it were extended from work point-to-work point ($KF = K_{actual}/K_{core} = 1.28$).

These stiffness factors may be used to determine the horizontal load distribution on each story.

Assume $\rho = 1.3$.

Example 5.5.1 BRBF Brace Design



Example 5.5.1 BRBF Brace Design

Design a buckling-restrained brace to resist the resulting axial loading, P_{QE} = 113 kips.

The applicable building code specifies the use of ASCE/SEI 7 for calculation of loads. According to AISC *Seismic Provisions* Section F4.3, buckling-restrained braces should not be considered as resisting gravity forces.

Example 5.5.1 BRBF Brace Design

Using the area-based approach, allow for material variability of 42 ksi ± 4 ksi.

$F_{ysc\,min}$ = 38 ksi
$F_{ysc\,max}$ = 46 ksi

From an elastic analysis, the first-order interstory drift is Δ_H = 0.223 in.

Assume that the ends of the brace are pinned and braced against translation for both the *x*-*x* and *y*-*y* axes.

Example 5.5.1 BRBF Brace Design

Solution:

As indicated in AISC *Seismic Provisions* Section F4.3, the required brace strengths are not based on gravity loads; therefore, the required compressive and tensile strengths of the brace are:

$$\begin{aligned} P_u &= T_u \\ &= \rho P_{QE} \\ &= 1.3(113\text{ kips}) \\ &= 147\text{ kips} \end{aligned}$$

Example 5.5.1 BRBF Brace Design

Required Strength

Consider second-order effects

AISC *Specification* Appendix 8 is used to address second-order effects. The required second-order axial strength is:

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(}\textit{Spec.}\text{ Eq. A-8-2)}$$

For the calculation of B_2:

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\ story}}} \geq 1 \qquad \text{(}\textit{Spec.}\text{ Eq. A-8-6)}$$

Example 5.5.1 BRBF Brace Design

To determine P_{story}, use an area of 9,000 ft^2 on each floor and include only the surface gravity loads given in the BRBF Design Example Plan and Elevation section. The governing load combinations are as follows:

From Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on *L*):

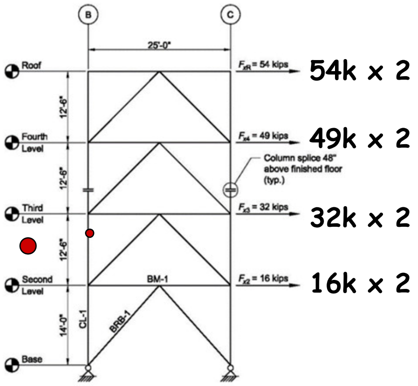
$$P_{story} = \left(9{,}000\text{ ft}^2\right) \times \left\{ \begin{array}{l} \left[1.2 + 0.2(1.0)\right] \\ \times\left[68\text{ psf} + 3(85\text{ psf})\right] \\ +1.3(0\text{ psf}) \\ +0.5(3)(50\text{ psf}) \\ +0.2(20\text{ psf}) \end{array} \right\} \times \left(1\text{ kip/1,000 lb}\right) + \left\{ \begin{array}{l} \left[1.2 + 0.2(1.0)\right] \\ \times(175\text{ lb/ft})(4)(390\text{ ft}) \\ +1.3(0\text{ lb}) \\ +0.5(0\text{ lb}) \\ +0.2(0\text{ lb}) \end{array} \right\} \times \left(1\text{ kip/1,000 lb}\right)$$

$$= 5{,}160\text{ kips}$$

Example 5.5.1 BRBF Brace Design

The total story shear, H, with two bays of bracing in the direction under consideration where each braced frame is designed to resist the seismic loads shown in Figure 5-87, is determined in the following. From an elastic analysis, the first-order interstory drift is Δ_H = 0.223 in.

$$\begin{aligned} H &= 2(54\text{ kips} + 49\text{ kips} + 32\text{ kips} + 16\text{ kips}) \\ &= 302\text{ kips} \\ L &= 14\text{ ft} \\ R_M &= 1.0\text{ for braced frames} \end{aligned}$$



Note: There are two of these frames in the direction under consideration

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Example 5.5.1 BRBF Brace Design

$$\begin{aligned} P_{e\,story} &= R_M \frac{HL}{\Delta_H} \qquad \text{(\textit{Spec.} Eq. A-8-7)} \\ &= 1.0 \frac{(302\text{ kips})(14\text{ ft})}{(0.223\text{ in.})(1\text{ ft/12 in.})} \\ &= 228{,}000\text{ kips} \end{aligned}$$

Example 5.5.1 BRBF Brace Design

Using AISC *Specification* Equation A-8-6:

$$\alpha = 1.0$$

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{e\,story}}} \geq 1$$

$$= \frac{1}{1 - \frac{1.0(5{,}160\text{ kips})}{228{,}000\text{ kips}}} \geq 1$$

$$= 1.02$$

Example 5.5.1 BRBF Brace Design

Considering second-order effects, the required compressive and tensile strengths of the brace are:

$$P_u = T_u$$

$$= 1.02(147\text{ kips})$$

$$= 150\text{ kips}$$

Example 5.5.1 BRBF Brace Design

Determination of the brace area required to resist the required brace strength must use the specified minimum yield stress of the core material, $F_{ysc\,min}$. For the limit state of tensile or compressive yielding, set the required strength equal to AISC *Seismic Provisions* Equation F4-1 and solve for $A_{sc\,min}$:

$$\begin{aligned} A_{sc\,min} &= \frac{P_u}{\phi F_{ysc\,min}} \\ &= \frac{150\text{ kips}}{0.90(38\text{ ksi})} \\ &= 4.39\text{ in.}^2 \end{aligned}$$

Try a BRB with a core area, A_{sc}, of 4.50 in.^{2}

Example 5.5.1 BRBF Brace Design

Note that while BRB manufacturers can fabricate a BRB with the accuracy to which the core can be cut (generally ± 1/8 in. in width), it is common to round the required core area up to standard increments.

Generally, it is good practice to specify core areas in 0.25 in.^{2} increments for 0 in.^{2} $< A_{sc} \leq 5.00$ in.^{2}, in 0.50 in.^{2} increments for 5.00 in.^{2} $< A_{sc} \leq 10.0$ in.^{2}, in 1.00 in.^{2} increments for 10.0 in.^{2} $< A_{sc} \leq 20.0$ in.^{2}, and in 2.00 in.^{2} increments for $A_{sc} > 20.0$ in.^{2} (or maintaining increment amounts in the range of 5% to 10% of the total amount).

When specifying BRB area greater than required, the EOR must account for the increased demand that the specified area will place on the structure because the beams and columns are designed to be stronger than the adjusted brace strength.

Example 5.5.1 BRBF Brace Design

For LRFD, the available axial strength for the limit state of tensile yielding is:

$$\begin{aligned}\phi P_{n\,min} &= \phi F_{ysc\,min} A_{sc} \qquad \text{(from } \textit{Spec.} \text{ Eq. D2-1)}\\ &= 0.90(38\text{ ksi})(4.50\text{ in.}^2)\\ &= 154\text{ kips} > 150\text{ kips} \quad \textbf{o.k.}\end{aligned}$$

Verify with the brace manufacturer that the stiffness factor *KF* = 1.28 is acceptable for a 4.50-in.^{2} brace of this length. The remainder of the brace design is performed by the BRB manufacturer. Overstrength factors, β and ω, along with the maximum deformation capability of the brace, must be provided by the brace manufacturer in order to design the columns and beams of the BRBF and to determine the BRB applicability to the design.

Example 5.5.1 BRBF Brace Design

The final part of the brace design is establishing the expected deformation of the brace and using this deformation to determine forces that the brace imposes on the columns, beams and connections. AISC *Seismic Provisions* Section F4.2 requires consideration of deformations at the greater of 2% drift or two times the design story drift.

Example 5.5.1 BRBF Brace Design

The design story drift is defined in the AISC *Seismic Provisions* Glossary as the calculated story drift, including the effect of expected inelastic action. As given, the first-order interstory drift is Δ_H = 0.223 in. This drift does not include the redundancy factor, ρ. Note that ASCE/SEI 7, Section 12.3.4.1, permits ρ to be taken equal to 1.0 for drift calculations. The design story drift including inelastic action is:

$$\begin{aligned}\Delta &= \frac{C_d \Delta_H}{I_e} \qquad \text{(from ASCE/SEI 7, Eq. 12.8-15)}\\ &= \frac{5(0.223\text{ in.})}{1.0}\\ &= 1.12\text{ in.}\end{aligned}$$

Example 5.5.1 BRBF Brace Design

Twice the story drift including inelastic action is:

$$\begin{aligned}2\Delta &= 2(1.12\text{ in.})\\ &= 2.24\text{ in.}\end{aligned}$$

2% drift corresponds to a deflection of:

$$\begin{aligned}\Delta &= 0.02H\\ &= 0.02(14\text{ ft})\\ &= 0.280\text{ ft}\\ \Delta &= (0.280\text{ ft})(12\text{ in./ft})\\ &= 3.36\text{ in.}\end{aligned}$$

Example 5.5.1 BRBF Brace Design

In this case, 2% drift governs. The brace spans 14 ft vertically and 12.5 ft horizontally. The brace deformation can be calculated to be:

$$\Delta_{br} = \left[\sqrt{(14\text{ ft})^2 + (12.5\text{ ft} + 0.280\text{ ft})^2} - \sqrt{(14\text{ ft})^2 + (12.5\text{ ft})^2}\right](12\text{ in./ft})$$
$$= 2.25\text{ in.}$$

Consulting with the brace manufacturer, the yield length for this brace is determined to be 70% of the work-point length. The yield length is the length over which the core is expected to yield and is typically equal to the length of casing.

Example 5.5.1 BRBF Brace Design

$$L = \sqrt{(14\text{ ft})^2 + (12.5\text{ ft})^2}$$
$$= 18.8\text{ ft}$$
$$L_y = 0.7L$$
$$= 0.7(18.8\text{ ft})(12\text{ in./ft})$$
$$= 158\text{ in.}$$

The strain is therefore:

$$\varepsilon = \frac{\Delta_{br}}{L_y}$$
$$= \left(\frac{2.25\text{ in.}}{158\text{ in.}}\right)(100\%)$$
$$= 1.42\%$$

Example 5.5.1 BRBF Brace Design

Determination of the strain and the yield length is typically performed by the brace manufacturer and is shown here for illustrative purposes only.

Consulting with the brace manufacturer, the β and ω factors corresponding to this level of strain are determined to be:

$\beta = 1.1$ $\omega = 1.36$

Alternatively, according to AISC *Seismic Provisions* Section F4.2c and ASCE/SEI 7, Chapter 16, brace deformation may be determined from a nonlinear analysis in lieu of the expected deformation requirements in AISC *Seismic Provisions* Section F4.2 illustrated here.

Example 5.5.1
BRBF Brace Design

AISC | Thank you

Smarter.
Stronger.
Steel.