EARTHQUAKE RESISTANT STEEL STRUCTURES

Design of Special Moment Frames (SMF) per AISC Specifications
SMF-Working principle

- Moment resisting frames rely on the flexural rigidity of their beams, columns, and beam-to-column connections for lateral force resistance and stability.
- As the frame deforms laterally, there is a tendency for the angle between the beams and columns to change. The rigidity of the beam-column connection resists this change through development of bending moments and shears in the beams and columns.
Behavior of an MRF Under Lateral Load:
Internal Forces and Possible Plastic Hinge Locations
SMF-Working principle
Possible Plastic Hinge Locations

- **Beam (Flexural Yielding)**
- **Panel Zone (Shear Yielding)**
- **Column (Flexural & Axial Yielding)**
- **Beam (Flexural Yielding)**
Plastic Hinges In Beams
Plastic Hinges
In Column Panel Zones
Plastic Hinges in Columns:

Potential for Soft Story Collapse
Design Requirement:
Frame must develop large ductility without failure of beam-to-column connection.
Connections in moment resisting frames are designed either fully restrained or partially restrained, depending on the stiffness and strength of the connection, relative to that of beams and columns. A fully restrained connection must be capable of preserving the angle between the beam and column essentially constant, until the weaker element (beam) yields in flexure.
Basis of Design

- Special Moment Frames (SMF) are expected to provide significant inelastic deformation capacity through flexural yielding of the SMF beams and limited yielding capacity of column panel zones.

- Despite some expectations, columns shall be designed to be stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base is permitted.

- Design of connections of beam to columns, including panel zones and continuity plates, shall be based on connection tests that provide the performance required by the code, or shall be chosen from the special connection detailings provided by the code.

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The following relation should be satisfied at beam-to-column connections:

\[
\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0
\]

where

\(\sum M_{pc}^*\) = the sum of the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. \(\sum M_{pb}^*\) = the sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.
Strong Column- Weak Beam
\[ \sum M_{pb}^* = \text{the sum of the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to determine } \sum M_{pb}^* \text{ as follows:} \]

\[ \sum M_{pb}^* = \sum (1.1 R_y F_{yb} Z_b + M_{uv}) \]

- \( R_y \): Ratio of the expected yield stress to the specified minimum stress
- \( F_{yb} \): Specified minimum yield stress of beam
- \( Z_b \): Plastic section modulus of the beam
- \( M_{uv} \): Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD combinations
Expected Flexural Strength of the Beam

\[ M_{uv} = V_{beam} \times (s_h + \frac{d_{col}}{2}) \]

- \( s_h \): the distance between the possible plastic hinge location and the column face (depends on the prequalified connection type ANSI_AISC 358)
- \( V_{beam} \): the shear resulting from the factored gravity loads and the shear results from application of the required flexural strengths on the both ends of the beam segment between the possible plastic hinge points
What is $R_y$ factor??

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$R_y$: Ratio of the expected tensile strength to the minimum specified tensile strength of the material.
It is permitted to determine $\Sigma M_{pc}^*$ as follows:

$$\sum M_{pc}^* = \sum Z_c (F_{yc} - P_{uc}/A_g)$$

where

$A_g =$ gross area of column

$F_{yc} =$ specified minimum yield stress of column

$P_{uc} =$ required compressive strength using LRFD combinations, including the amplified seismic loads

$Z_c =$ plastic section modulus of the column

(Since this calculation is conservative, I advise you to use above formula compatible with AISC Specification. However, you may consider to carry the shear force from hinge location to the intersection of beam and column axes. You may assume the inflection point as the mid-height of the column in this case.)

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Must provide adequate lateral bracing of beams in SMF so that severe strength degradation due to lateral torsional buckling is delayed until sufficient ductility is achieved.

Sufficient ductility = interstory drift angle of at least 0.04 rad for prequalified connections (for connections to be tested it is 0.04 rad achieved under Appendix S loading protocol)
Lateral Torsional Buckling

Lateral torsional buckling controlled by:

\[ \frac{L_b}{r_y} \]

\( L_b = \text{distance between beam lateral braces} \)
\( r_y = \text{weak axis radius of gyration} \)
Effect of Lateral Torsional Buckling on Flexural Strength and Ductility:

Increasing $\frac{L_b}{r_y}$
Stability Bracing of Beams

Both flanges of beams shall be laterally braced, with a maximum spacing of \( L_b = 0.086 \frac{E}{F_y} r_y \)

\[
L_b \leq 0.086 \left( \frac{E}{F_y} \right) r_y \quad \left( = 50r_y \text{ for } F_y = 50 \text{ ksi} \right)
\]

Note:
For typical SMF beam: \( r_y \approx 2 \) to 2.5 inches.
and \( L_b \approx 100 \) to 125 inches (approx. 8 to 10 ft)
In addition to lateral braces provided as a maximum spacing of \( L_b = 0.086 \frac{r_y E}{F_y} \):

Lateral braces shall be placed near concentrated forces, changes in cross-section and other locations where analysis indicates that a plastic hinge will form.

The placement of lateral braces shall be consistent with that specified in ANSI/AISC 358 for a Prequalified Connection, or as otherwise determined by qualification testing.
Bolted Flange Plate Connection (BFP): For lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located a distance of \(d\) to \(1.5d\) from the bolt farthest from the face of the column.
Fig. 7.1. Bolted flange plate moment connection.
Stability Bracing of Beams

- **Welded Unreinforced Flange-Welded Web (WUF-W) Moment Connection:** for lateral bracing at plastic hinges, supplemental lateral bracing shall be provided at both the top and bottom beam flanges, and shall be located at a distance of \( d \) to \( 1.5d \) from the face of the column. No attachment of lateral bracing shall be made to the beam in the region extending from the face of the column to a distance \( d \) from the face of the column.

- **Exception:** For both SMF and IMF systems, where the beam supports a concrete structural slab that is connected along the beam span between protected zones with welded shear connectors spaced at a maximum of 12 in. (300 mm) on center, supplemental top and bottom flange bracing at plastic hinges is not required.
Fig. 8.1. WUF-W moment connection.
Beam lateral bracing requirements for the WUF-W moment connection are identical to those for the Reduced Beam Section (RBS) moment connection. The effects of beam lateral bracing on cyclic loading performance have been investigated more extensively for the RBS moment connection than for the WUF-W moment connection. However, the available data for the WUF-W moment connection suggests that beams are less prone to lateral-torsional buckling than with the RBS moment connections. Consequently, it is believed that lateral bracing requirements established for the RBS moment connection are satisfactory, and perhaps somewhat conservative, for the WUF-W moment connection.
For beams with a RBS connection, when a composite concrete floor slab is present, no additional lateral bracing is required at the RBS. When a composite concrete floor slab is not present, provide an additional lateral brace at the RBS. Attach the brace just outside of the RBS cut, at the end farthest from the column face.
Stability Bracing of Beams

- If lateral (or torsional) braces are used, they should provide an available strength of 6% of the expected capacity of the beam flange at the plastic hinge location.

- If a reduced beam section is used, the reduced flange width may be considered in calculating the bracing force.
Columns of SMF are required to be braced to prevent rotation out of the plane of the moment frame because of the anticipated inelastic behavior in, or adjacent to, the beam-to-column connection during high seismic activity.
Stability Bracing at Beam-to-column connections

(1) Braced Connections

Beam-to-column connections are usually braced laterally by the floor or roof framing. When the webs of the beams and columns are co-planar, and a column is shown to remain elastic outside of the panel zone, column flanges at beam-to-column connections shall require stability bracing only at the level of the top flanges of the beams. It shall be permitted to assume that the column remains elastic when the ratio calculated using \[ \frac{\sum M^*_{pc}}{\sum M^*_{pb}} \] is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply: (1) The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Stability bracing is permitted to be either direct or indirect. (2) Each column-flange member brace shall be designed for a required strength that is equal to 2% of the available beam flange strength \( F_y b f t_{bf} \).
Stability Bracing at Beam-to-column connections

**Direct stability bracing** of the column flange is achieved through use of member braces or other members, deck and slab, attached to the column flange at or near the desired bracing point to resist lateral buckling. **Indirect stability** bracing refers to bracing that is achieved through the stiffness of members and connections that are not directly attached to the column flanges, but rather act through the column web or stiffener plates.

The considerations about the unbraced connections can be found in the AISC Seismic Provisions E3.4c (2). We will not emphasize this topic herein, since braced case is common in multistory buildings.
Members of SMF

- Basic Requirements

Beams and columns shall satisfy Section D1.1 (compactness) for highly ductile members (unless otherwise qualified by tests).

Beams are permitted to be composite with reinforced concrete slab to resist gravity loads.
Members of SMF

- Beam Flanges

Abrupt changes in beam flange area are prohibited in plastic hinge regions. The drilling of flange holes or trimming of beam flange width shall not be permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to accommodate the required story drift angle.

The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Section K1, or in a program of qualification testing in accordance with Section K2.
Members of SMF

- Protected Zones
- Region at each end of beam subject to inelastic straining shall be designated as a protected zone.
- Extent of protected zone as designated in ANSI/AISC 358 or qualification testing.
- In general, for unreinforced connections, from face of column to one half beam depth beyond plastic hinge point.
Connections

- **Demand Critical Welds**
  - (1) Groove welds at column splices
  - (2) Welds at column-to-base plate connections (Exceptions - Section E3.6a)
  - (3) Complete-joint-penetration groove welds of beam flanges and beam webs to columns (Exceptions - Section E3.6a)
ANSI/AISC 358 describes six different connections that have been prequalified for use in both IMF and SMF systems. The prequalified connections include:

1. The reduced beam section (RBS),
2. The bolted unstiffened extended end plate (BUEEP),
3. The bolted stiffened extended end plate (BSEEP),
4. The bolted flange plate (BFP),
5. The welded unreinforced flange-welded web (WUF-W),
6. The Kaiser bolted bracket (KBB).

In a few cases, the limitations on use of the connections are less strict for IMF than for SMF, but generally, the connections are the same.
Beam-to-Column Connections

- Beam-to-column connections used in the seismic force resisting system (SFRS) shall satisfy the following requirements:

  1. The connection shall be capable of accommodating an interstory drift angle of at least 0.04 rad.

  2. The measured flexural resistance of the connection, determined at the column face, shall equal at least 0.80$M_p$ of the connected beam at an interstory drift angle of 0.04 rad.
Beam-to-Column Connections

Fig. C-E3.1. Acceptable strength degradation, per Section E3.6b.

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Required Shear Strength of the Connection

6d. **Required Shear Strength**

The required shear strength of the connection shall be based on the load combinations in the applicable building code that include the amplified seismic load. In determining the amplified seismic load the effect of horizontal forces including over-strength, $E_{mh}$, shall be taken as:

$$E_{mh} = 2(1.1R_y M_p)/L_h \quad \text{(E3-6)}$$

where

$L_h$ = distance between plastic hinge locations as defined within the test report or ANSI/AISC 358, in. (mm)

$M_p$ = nominal plastic flexural strength, kip-in. (N-mm)

$R_y$ = ratio of the expected yield stress to the specified minimum yield stress, $F_y$
12.4.3 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor applications shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.3.2 or load combinations 5 and 6 in Section 2.4.1, \( E \) shall be taken equal to \( E_m \) as determined in accordance with Eq. 12.4-5 as follows:

\[
E_m = E_{mh} + E_v
\]  
(12.4-5)

2. For use in load combination 7 in Section 2.3.2 or load combination 8 in Section 2.4.1, \( E \) shall be taken equal to \( E_m \) as determined in accordance with Eq. 12.4-6 as follows:

\[
E_m = E_{mh} - E_v
\]  
(12.4-6)

where

- \( E_m \) = seismic load effect including overstrength factor
- \( E_{mh} \) = effect of horizontal seismic forces including overstrength factor as defined in Section 12.4.3.1
- \( E_v \) = vertical seismic load effect as defined in Section 12.4.2.2

12.4.3.1 Horizontal Seismic Load Effect with Overstrength Factor

The horizontal seismic load effect with overstrength factor, \( E_{mh} \), shall be determined in accordance with Eq. 12.4-7 as follows:

\[
E_{mh} = \Omega_o Q_E
\]  
(12.4-7)

where

- \( Q_E \) = effects of horizontal seismic forces from \( V, F_{ps} \), or \( F_p \) as specified in Sections 12.8.1, 12.10, or 13.3.1. Where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.
- \( \Omega_o \) = overstrength factor

12.4.3.2 Load Combinations with Overstrength Factor

Where the seismic load effect with overstrength factor, \( E_m \), defined in Section 12.4.3, is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combination for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in either Section 2.3.2 or 2.4.1:

Basic Combinations for Strength Design with Overstrength Factor (see Sections 2.3.2 and 2.2 for notation).

- 5. \((1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S\)
- 7. \((0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H\)
Required Shear Strength of the Connection

\[
V = \max(V_{\text{beam, left}}, V_{\text{beam, right}}) = V_u = 2 \left( 1.1 R_y M_p \right) / L_h + V_{\text{gravity}}
\]

\[
L_h = (1.2 + 0.2S_{DS}) D + L(+0.2S) \quad \text{OR} \quad (0.9 - 0.2S_{DS}) D (+1.6H)
\]

\[
V_{\text{beam, left}} \quad V_{\text{beam, right}}
\]

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Panel Zone:

**Shear Strength:** The minimum required shear strength, $R_u$, of the panel zone shall be taken as the shear generated in the panel zone when plastic hinges form in the beams.

To compute panel zone shear.....

- Determine moment at beam plastic hinge locations
  \( (1.1 \, R_y \, M_p \text{ or as specified in ANSI/AISC 358}) \)
- Project moment at plastic hinge locations to the face of the column (based on beam moment gradient)
- Compute panel zone shear force.
Panel Zone Shear Strength (cont)

\[ M_{pr} = \text{expected moment at plastic hinge} = 1.1 \, R_y \, M_p \text{ or as specified in ANSI/AISC 358} \]

\[ V_{beam} = \text{beam shear (see Section 9.2a - beam required shear strength)} \]

\[ s_h = \text{distance from face of column to beam plastic hinge location (specified in ANSI/AISC 358)} \]
Panel Zone Shear Strength (cont)

$M_f = \text{moment at column face}$

$$M_f = M_{pr} + V_{beam} \times s_h$$
Panel Zone Shear Strength (cont)

Panel Zone Required Shear Strength =

\[ R_u = \frac{\sum M_f}{(d_b - t_f)} V_c \]
Panel Zone Design Requirement:

\[ R_u \leq \phi_v R_v \quad \text{where} \quad \phi_v = 1.0 \]

\( R_v \) = nominal shear strength, based on a limit state of shear yielding, as computed per Section J10.6 of the AISC Specification
To compute nominal shear strength, \( R_v \), of panel zone:

When \( P_u \leq 0.75 P_y \) in column:

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

(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
- \( t_p \) = thickness of column web including doubler plate
Panel Zone Shear Strength (cont)

To compute nominal shear strength, $R_v$, of panel zone:

When $P_u > 0.75 P_y$ in column (not recommended):

$$R_v = 0.6 F_y d_c t_p \left[ 1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_p} \right] \left[ 1.9 - \frac{1.2P_u}{P_y} \right]$$

(AISC Spec EQ J10-12)
If shear strength of panel zone is inadequate:
- Choose column section with larger web area
- Weld doubler plates to column

Options for Web Doubler Plates
9.5 Continuity Plates
9.5 Continuity Plates
Continuity plates shall be consistent with the requirements of a prequalified connection as specified in ANSI/AISC 358 (Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications) or

As determined in a program of qualification testing in accordance with Appendix S
Continuity plates are required, unless:

\[
t_{cf} \geq 0.4 \sqrt{1.8 b_{bf} t_{bf} \frac{R_{yb} F_{yb}}{R_{yc} F_{yc}}} \]

and

\[
t_{cf} \geq \frac{b_{bf}}{6}
\]

\[F_{yb} = \text{specified minimum yield stress of the beam flange, ksi (MPa)}\]
\[F_{yc} = \text{specified minimum yield stress of the column flange, ksi (MPa)}\]
\[R_{yb} = \text{ratio of the expected yield stress of the beam material to the specified minimum yield stress}\]
\[R_{yc} = \text{ratio of the expected yield stress of the column material to the specified minimum yield stress}\]
\[b_{bf} = \text{beam flange width, in. (mm)}\]
\[t_{bf} = \text{beam flange thickness, in. (mm)}\]
\[t_{cf} = \text{minimum required thickness of column flange when no continuity plates are provided, in. (mm)}\]
For Box Columns:

Continuity plates must be provided.
Required thickness of continuity plates

a) For one-sided (exterior) connections, continuity plate thickness shall be at least one-half of the thickness of the beam flange.

b) For two-sided (interior) connections, continuity plate thickness shall be at least equal to the thicker of the two beam flanges on either side of the column.

For other design, detailing and welding requirements for continuity plates - See ANSI/AISC 358
\[ t_{cp} \geq \frac{1}{2} \times t_{bf} \]
$t_{cp} \geq \text{larger of } (t_{bf-1} \text{ and } t_{bf-2})$
Common Requirements (not specific to SMFs) for all
Seismic Force Resisting Systems (SFRS)

4. Columns

Columns in moment frames, braced frames and shear walls shall satisfy the requirements of this section.

4a. Required Strength

The required strength of columns in the SFRS shall be determined from the following:

(1) The load effect resulting from the analysis requirements for the applicable system per Sections E, F, G and H.

Exception: Section D1.4a need not apply to Sections G1, H1 or H4.

(2) The compressive axial strength and tensile strength as determined using the load combinations stipulated in the applicable building code including the amplified seismic load. It is permitted to neglect applied moments in this determination unless the moment results from a load applied to the column between points of lateral support. The required axial compressive strength and tensile strength need not exceed either of the following:

(a) The maximum load transferred to the column by the system, including the effects of material overstrength and strain hardening in those members where yielding is expected.

(b) The forces corresponding to the resistance of the foundation to overturning uplift.
Common Requirements (not specific to SMFs) for all Seismic Force Resisting Systems (SFRS)

5. Column Splices

5a. Location of Splices

For all building columns, including those not designated as part of the SFRS, column splices shall be located 4 ft (1.2 m) or more away from the beam-to-column flange connections.

Exceptions:

(1) When the column clear height between beam-to-column flange connections is less than 8 ft (2.4 m), splices shall be at half the clear height

(2) Column splices with webs and flanges joined by complete-joint-penetration groove welds are permitted to be located closer to the beam-to-column flange connections, but not less than the depth of the column
5b. Required Strength

The required strength of column splices in the SFRS shall be the greater of:

(a) The required strength of the columns, including that determined from Chapters E, F, G and H and Section D1.4a; or,

(b) The required strength determined using the load combinations stipulated in the applicable building code including the amplified seismic load. The required strength need not exceed the maximum loads that can be transferred to the splice by the system.
Common Requirements (not specific to SMFs) for all Seismic Force Resisting Systems (SFRS)

Required strength of the column splices cont.

In addition, welded column splices in which any portion of the column is subject to a calculated net tensile load effect determined using the load combinations stipulated in the applicable building code, including the amplified seismic load, shall satisfy all of the following requirements:

1. The available strength of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200% of the required strength.

2. The available strength for each flange splice shall be at least equal to $0.5R_yF_yb_{tf}$, where $R_yF_y$ is the expected yield stress of the column material and $b_{tf}$ is the area of one flange of the smaller column connected.

3. Where butt joints in column splices are made with complete-joint-penetration (CJP) groove welds, when tension stress at any location in the smaller flange exceeds $0.30F_y$, tapered transitions are required between flanges of unequal thickness or width. Such transitions shall be in accordance with AWS D1.8/D1.8M clause 4.2.
5c. **Required Shear Strength**

For all building columns including those not designated as part of the SFRS, the *required shear strength* of column splices with respect to both orthogonal axes of the column shall be $M_{pc}/H$ (LRFD), where $M_{pc}$ is the lesser nominal plastic flexural strength of the column sections for the direction in question, and $H$ is the height of the story.

The required shear strength of splices of columns in the SFRS shall be the greater of the above requirement or the required shear strength determined per Section D2.5b(a) and (b).
Common Requirements (not specific to SMFs) for all Seismic Force Resisting Systems (SFRS)

Requirements for Column Bases

6a. Required Axial Strength

The required axial strength of column bases that are designated as part of the SFRS, including their attachment to the foundation, shall be the summation of the vertical components of the required connection strengths of the steel elements that are connected to the column base, but not less than the greater of:

(a) The column axial load calculated using the load combinations of the applicable building code, including the amplified seismic load

(b) The required axial strength for column splices, as prescribed in Section D2.5

User Note: The vertical components can include both the axial load from columns and the vertical component of the axial load from diagonal members framing into the column base. Section D2.5 includes references to Section D1.4a and Chapters E, F, G and H. Where diagonal braces frame to both sides of a column, the effects of compression brace buckling should be considered in the summation of vertical components. See Section F2.3.
Common Requirements (not specific to SMFs) for all Seismic Force Resisting Systems (SFRS)

Requirements for Column Bases cont.

6b. Required Shear Strength

The required shear strength of column bases, including those not designated as part of the SFRS, and their attachments to the foundations, shall be the summation of the horizontal component of the required connection strengths of the steel elements that are connected to the column base as follows:

(a) For diagonal braces, the horizontal component shall be determined from the required strength of diagonal brace connections for the SFRS.

(b) For columns, the horizontal component shall be equal to the required shear strength for column splices prescribed in Section D2.5c.
Common Requirements (not specific to SMFs) for all Seismic Force Resisting Systems (SFRS)

Requirements for Column Bases cont.

6c. Required Flexural Strength

Where column bases are designed as moment connections to the foundation, the required flexural strength of column bases that are designated as part of the SFRS, including their attachment to the foundation, shall be the summation of the required connection strengths of the steel elements that are connected to the column base as follows:

(a) For diagonal braces, the required flexural strength shall be at least equal to the required flexural strength of diagonal brace connections.

(b) For columns, the required flexural strength shall be at least equal to the lesser of the following:

(i) $1.1R_yF_yZ$

(ii) the moment calculated using the load combinations of the applicable building code, including the amplified seismic load.