Strong Frame®
Special Moment Frames

Simpson Strong-Tie® Strong Frame special moment frames provide optimal moment transfer solutions for both new and retrofit projects. Our Yield-Link® structural fuse technology ensures the resilience of the frame during seismic events.

For special moment frame offerings, design requirements and available options, visit **strongtie.com/strongframe.**

Design Requirements and Considerations

Design:

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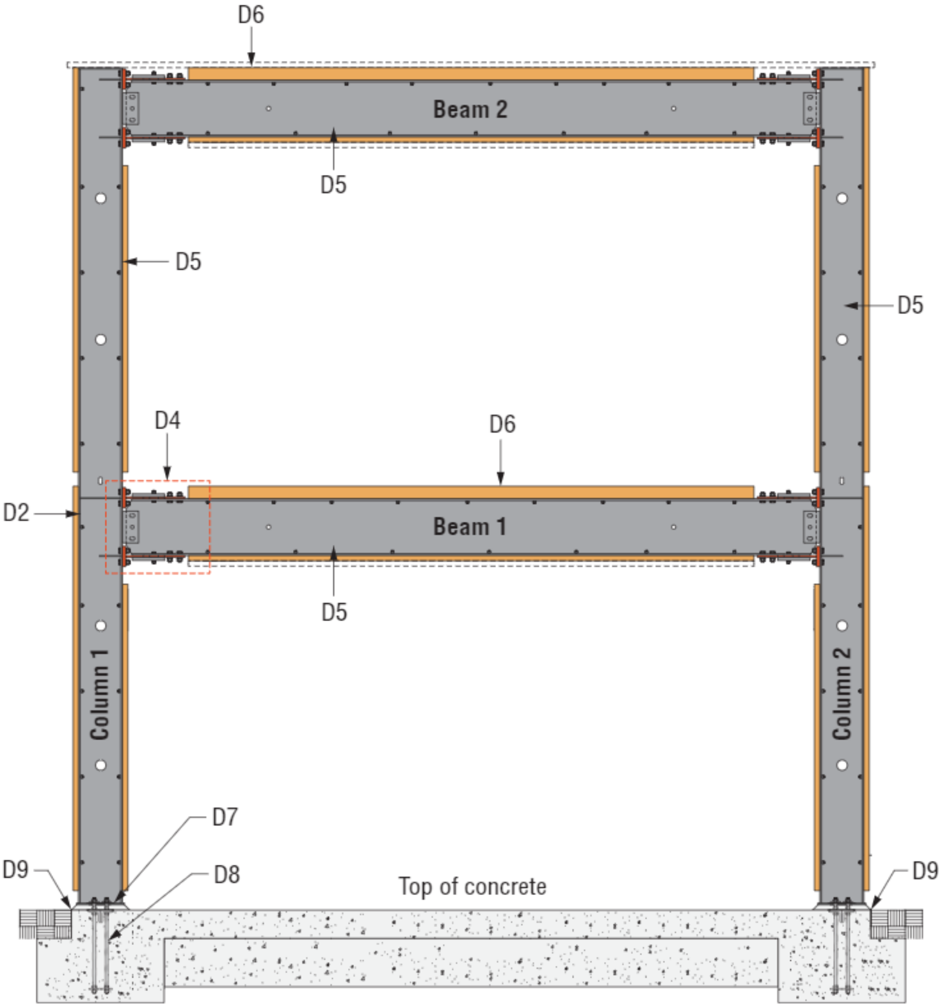


Moment Frame Design Requirements and Assumptions

D1. Drift Check

Drift Check for Seismic Loads

ASCE 7-16 Section 12.12.1 states that design story drift of a structure shall not exceed the allowable drift limit listed in Table 12.12-1. For seismic applications, the story drift limitation not only serves as a serviceably check but is an inherent ductility requirement for seismic design related to the Response Modification Coefficient (R-value) as well as structural stability.

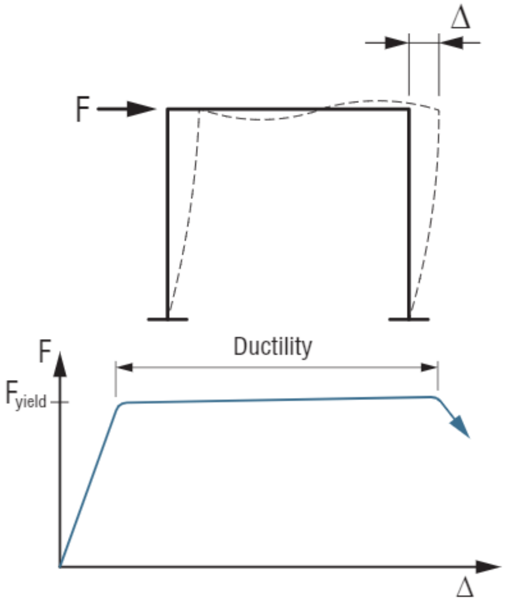


Figure D1.1 — Drift and Ductility Relationship

In the current seismic design philosophy, structures do not have to be designed for the Maximum Considered Earthquake (MCE) forces. Reduction in design forces is primarily related to the R-value in lateral force-resisting systems. The R-value for each lateral system is related to ductility and design codes have taken this into consideration when assigning higher R-values to more ductile systems. Reduced design forces used for drift check should be at strength level (LRFD) (ASCE 7-16 Section 12.8.6), and the deflection amplification factor (C_d) used shall correspond to the R-value used for the lateral force-resisting system. Please note, for drift check, ρ shall be taken as 1.0 per ASCE 7 Section 12.3.4.1. In addition, drift check need not include overstrength combinations since the ultimate displacement calculation already includes the C_d factor.

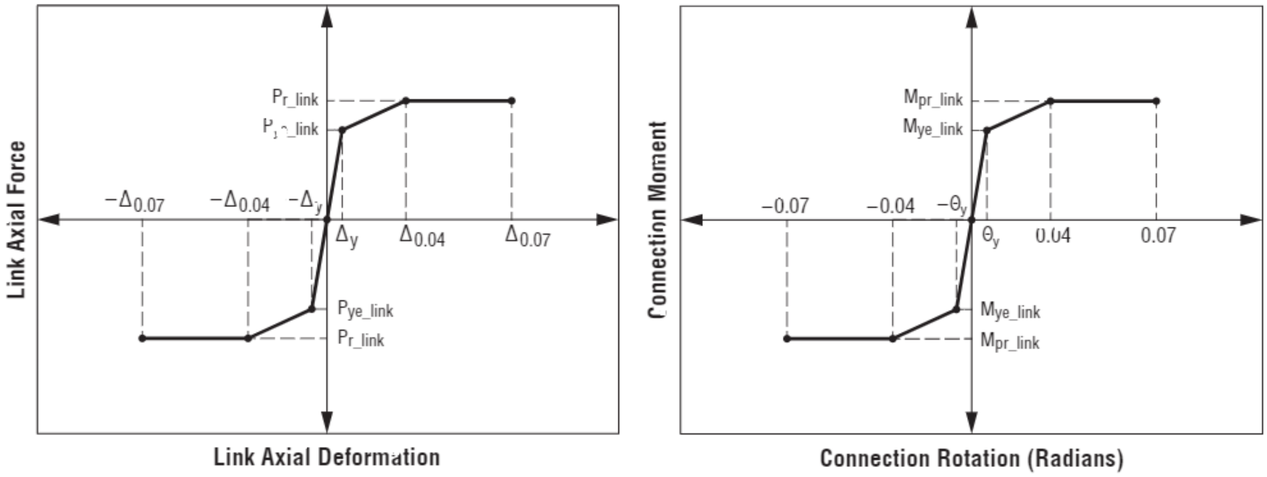
Drift Check for Wind Loads

Currently, there are no drift limit requirements for wind design. However, there are some recommendations for serviceability considerations, such as Appendix C in ASCE 7 and AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings*.

Moment Frame Design Requirements and Assumptions (cont.)

Strong Frame® Special Moment Frame Drift Check

Because the Strong Frame SMF connection is considered a partially restrained (PR) connection, modeling and analysis is more involved than for a traditional moment frame connection. When designing and analyzing PR connections, the strength and stiffness of the connection need to be considered. A detailed step-by-step procedure to calculate the axial Yield-Link® or rotational Yield-Link parameters for Strong-Frame moment connection is documented in Chapter 12 of the AISC 358-16. Once the PR connection is modeled, frame drift can be calculated similar to the traditional fully restrained (FR) connections. For pushover or nonlinear time history analysis, a full nonlinear axial Yield-Link or rotational Yield-Link model is required (see Figure D1.2). Design tools for calculating the Yield-Link parameters can be obtained from Simpson Strong-Tie at **strongtie.com**.



Case 1: Link Axial Force vs Link Axial Deformation

Case 2: Connection Moment vs. Rotation

Figure D1.2 — Simpson Strong-Tie® Yield-Link Modeling Parameters Connection Moment (Ref: AISC 358-16, Chapter 12)

Drift Check Options in Strong Frame Selector Software

The Simpson Strong-Tie Strong Frame selector is a software tool developed to assist designers to size moment frames for their projects. The adjacent table lists the various selections available within the Strong Frame selector for considerations of drift for seismic and wind design. These are provided from least restrictive to more restrictive as you move down the table. The appropriate drift selection may depend on building code and/or material requirements such as Structure Type, Risk Category, Finish Materials or various other considerations in order to accommodate the story drift. For other drift/deflection requirements not listed here, contact Simpson Strong-Tie to assist with providing a tailored design to meet your specific requirements.

Seismic Drift	Wind Drift	
0.025 hx	No Limit	Least restrictive
0.020 hx	H/175	↓
0.015 hx	H/250	
0.010 hx	H/300	
	H/350	
	H/400	More restrictive

Moment Frame Design Requirements and Assumptions (cont.)

D2. Panel Zone Check

Other than drift check, the second limit state that governs the design of a moment frame is the connection panel zone shear capacity for the column. The capacity of the panel zone depends mostly on the thickness of the column web. When design limits are exceeded, many engineers tend to increase the thickness of the column web by welding a doubler plate to increase the shear capacity. However, many fabricators are aware that increasing the column web thickness by increasing column weight approximately up to 75 plf (e.g., from a W14x74 to, say, a W14x145) can result in a less expensive frame due to the elimination of the welding cost and inspection cost of the doubler plate.

If panel zone capacity is not checked, the consequence can be column kinking due to a weak panel zone (Figure D2.1). This can lead to column flange fracture just above and below the beam flanges connecting to the column. This phenomenon has been observed after a strong seismic event (Figure D2.2) as well as reproduced in laboratory testing (Figure D2.3).

Strong Frame Special Moment Frame Panel Zone Check

For typical SMF connection design (e.g., RBS), the design shear demand on the panel zone is calculated from the summation of the moments at the face of the column by projecting the expected moment at the plastic hinge point to the column faces.

For the Strong Frame SMF, the panel zone demand is calculated from statics using the shear at the top and bottom of the beam from the Yield-Link® ultimate axial capacity ($P_{r\text{-}link}$). This demand is higher than that of a typical moment connection, where the expected moment is taken as, $M_{pe} = R_y{}^*F_y{}^*Z_x$, where $R_y = 1.1$ and $F_y = 50$ ksi for A992 steel. For the Strong Frame, $P_{r\text{-}link}$ is calculated using $R_t = 1.2$ and $F_u = 65$ ksi. On the capacity side, the Strong Frame panel zone's shear capacity is calculated assuming a $\phi = 0.9$, whereas $\phi = 1.0$ is used in the typical moment connection design. Panel zone capacity check is required by AISC 341 and is provided in the calculations supplied by Simpson Strong-Tie.

Figure D2.1 — Column Kinking Attributable to Weak Panel Zone

(Ref: Uang and Chi, SSRP-2001/05, Effect of Straightening Methods on the Cyclic Behavior of k Area in Steel Rolled Shapes)

Figure D2.2 — Fracture of Welded Beam-to-Column Connection in Northridge Earthquake

(Ref: NIST GCR 09-917-3, NEHRP Seismic Design Technical Brief No.2)

Figure D2.3 — Fracture of Welded Beam-to-Column Connection in a Laboratory Test

(Ref: Uang and Chi, SSRP-2001/05, Effect of Straightening Methods on the Cyclic Behavior of k Area in Steel Rolled Shapes)

Moment Frame Design Requirements and Assumptions (cont.)

D3. Strong Column/Weak Beam Check

The moment ratio between the columns and beams in Section E3.4a of AISC 341-16 is one of the requirements that distinguishes a steel SMF from an IMF or OMF. For SMF, plastic hinges are expected to form in the beams (Figure D3.1a). If plastic hinges occur in the columns (meaning the beams are stronger than the columns), there is a potential for the formation of a weak-story mechanism (Figure D3.1b).

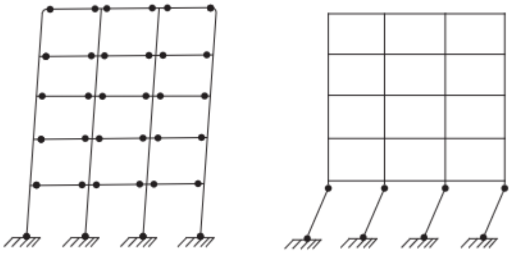

(a) (b)

Figure D3.1 — Weak-Story Mechanism

Simpson Strong-Tie® SMF Strong Column-Weak Yield-Link® Check

The Strong Frame special moment frame is unlike the typical SMF, which has either a reinforced connection (e.g., bolted flange plate connections) or weakened beam connection (e.g., RBS connections) where the plastic hinges are formed by the buckling of the beam flange and web (Figure D3.2). In the Strong Frame SMF, the stretching and shortening of the Yield-Links at the top and bottom of the Strong Frame beams are the yielding mechanisms (Figure D3.3). So instead of a strong column — weak beam check, the Strong Frame design procedure checks for a strong column — weak Yield-Link condition where the ratio of the column moments to the moment created by the Yield-Link couple is required to be greater than or equal to 1.0.

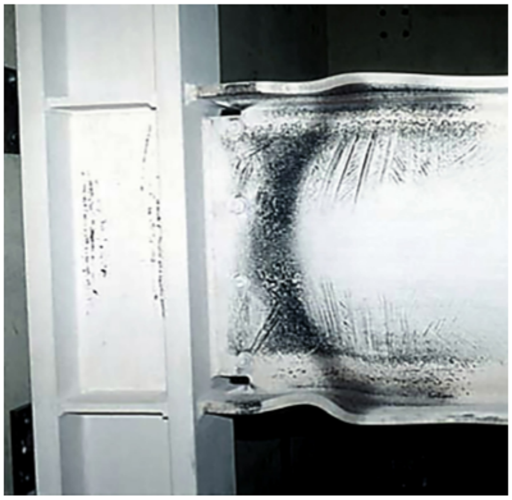


Figure D3.2 — Plastic Hinge in Beam Element for Typical SMF Connection

(Ref: NIST GCR 09-917-3, NEHRP Seismic Design Technical Brief No.2)

Figure D3.3 — Yielding in Strong Frame Yield-Links

Moment Frame Design Requirements and Assumptions (cont.)

D4a. Beam Bracing

Since special moment frames are required to have the resilience to withstand large rotation at the column-to-beam connection, the beams need to be stabilized using bracing to resist global buckling.

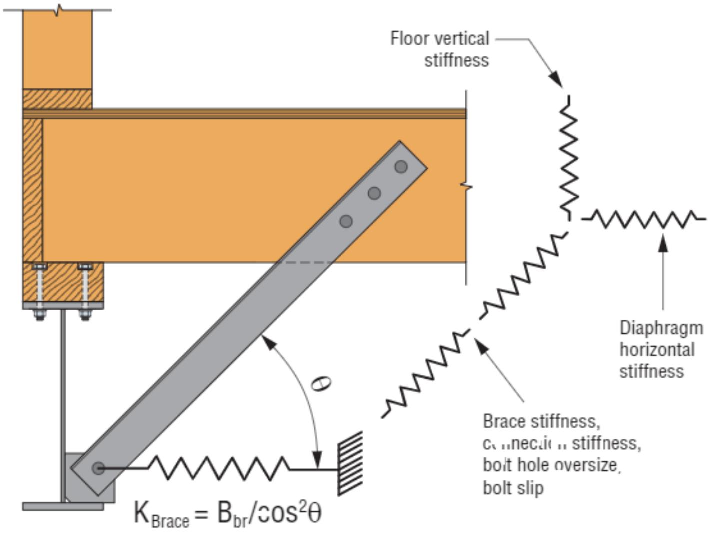
Beam Bracing Requirements

Steel special moment frame beam bracing is required by code to prevent beam torsional or flexural buckling as plastic hinges form. To preclude undesirable beam buckling failure modes that may occur during the formation of plastic hinges in the beam, Section D1.2.2b of AISC 341-16 has the following requirement for SMF for highly ductile members (i.e., beam element) with a maximum spacing of $L_b = 0.095r_yE/(F_y{}^*R_y)$.

In addition, unless justified by testing, beam bracing shall be provided near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformation of the special moment frame.

Each prequalified moment connection type has different requirements for beam bracing. For RBS connections, per AISC 358-16, supplemental lateral bracing of beams shall be provided near the reduced section. In addition, the attachment to the beam shall be located no greater than d/2 beyond the end of the reduced beam section. See AISC 358-16 for additional design guidelines.

In structural steel buildings, additional steel beams connected to full-depth shear tabs with slip-critical bolts have little difficulty in satisfying SMF bracing strength and stiffness requirements. However, meeting the code-prescribed bracing requirements is far more problematic when installing SMF in light-frame construction. There are deflections in the brace caused by oversized holes in the wood, vertical deflection of the floor beam and horizontal deflection of the floor diaphragm. Each of these sources of deflection added in sequence makes it harder to achieve the minimum bracing stiffness mandated by AISC for an SMF.



Moment Frame Design Requirements and Assumptions (cont.)

Consequences of Inadequate Bracing

Currently AISC 360-16 Appendix 6 has both strength and stiffness requirements for beam bracing. If no bracing or inadequate bracing is provided (failing either the strength or the stiffness requirements), the frame designed will not achieve the expected full capacity. The beam will either buckle in torsion (Figure D4a.1) or in flexure (Figure D4a.2) prior to the formation of the plastic hinge in the beam at the connection region.

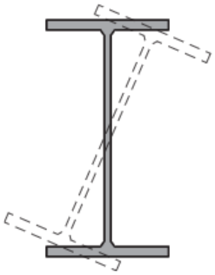


Figure D4a.1 — Beam Torsional Buckling

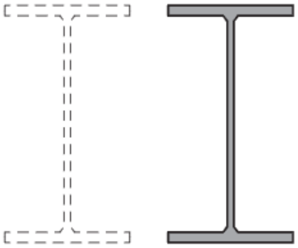


Figure D4a.2 — Beam Flexural Buckling

Ways to Brace a Beam

Per AISC 341-16, there are two methods to brace the beam: (1) lateral bracing (Figure D4a.3) and (2) torsional bracing (Figure D4a.4). Under lateral bracing, one can brace the beam at the compression flange (either top or bottom or both, depending on loading). Under torsional bracing, one is trying to prevent the section from twisting. To prevent twisting, typically a full-depth stiffener is welded to the SMF beam and connected to another beam nearby.

Figure D4a.3 — Beam Lateral Bracing (Concrete Slab at Top)

(Photo credit: NEHRP Seismic Design Technical Brief No. 2: Seismic Design of Steel Special Moment Frames: A Guide for Practicing Engineers, NIST GCR 09-917-3, June 2009.)

Stability Bracing at Beam-to-Column Connections

In addition to beam bracing, AISC 341-16 Section E3.4c requires connections to be braced at the column. When columns cannot be shown to remain elastic outside of the panel zone, column flanges shall be laterally braced at the levels of both the top and the bottom beam flanges. However, if the columns are shown to remain elastic outside of the panel zone, column flange bracing is required at the top flanges of the beams only. Each column flange brace shall be designed for a required strength that is equal to 2% of the available beam flange strength. For the Yield-Link moment connection, if the column is designed in accordance with Section 12.9 in AISC 358 (maximum nominal flexural strength is calculated using Sx, instead of Zx), only bracing at the level of the beam top flange is required.

Bracing can be either direct or indirect stability bracing. Direct bracing is achieved through the use of member braces or other members (decks, slabs, etc.) attached to the column flange at or near the bracing point. Indirect bracing is achieved through connecting through the column web or stiffener plates.

Special moment frame beam-to-column connections can be unbraced also. However, the column needs to be designed for the overall height between the adjacent brace points and the following criteria need to be applied:

1. The design strength shall be determined from the amplified seismic load combinations according to the applicable building code.
2. The L/r for the column shall not exceed 60.
3. The column's required flexural strength transverse to the seismic frame shall include moment from beam-bracing forces of 2% of the beam flange strength.

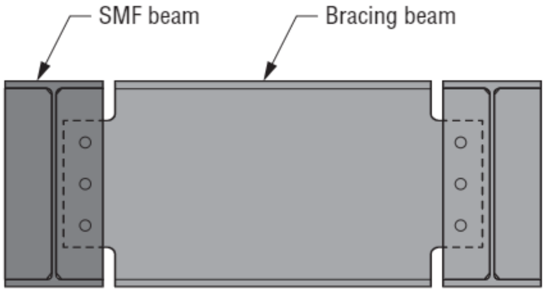


Figure D4a.4 — Torsional Bracing

Moment Frame Design Requirements and Assumptions (cont.)

Strong Frame Special Moment Frame Beam Bracing

With the introduction of the Strong Frame special moment frame, the Yield-Link® structural fuses are designed to develop plastic deformations, where beam bracing is not required. There is no inelastic lateral torsional buckling of the beam because yielding takes place at the Yield-Link structural fuses and not in the beam itself. The beam is designed to span between the supports for the maximum load the Yield-Link structural fuse system can deliver.

Figure D4a.5 below is a plot from our finite element analysis showing the equivalent plastic strain in the moment connection. All the yielding is concentrated (indicated by the green color) in the Yield-Link. The elastic beam behavior is supported by our testing as shown in Figure D4a.6. Strain gauges placed on the beam's bottom flange near the moment connection clearly show the elastic behavior in the beam. Also note the symmetry of the readings on strain gauges placed on each side of the beam. The overlapping of the red and blue lines indicate no torsional or flexural buckling occurred in the beam during testing, even at a frame drift level of 6%.

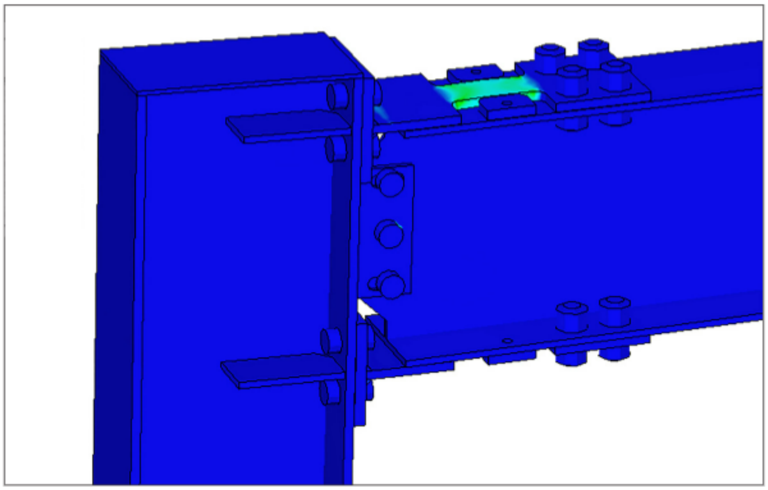


Figure D4a.5 — Equivalent Plastic Strain of Simpson Strong-Tie® Strong Frame Special Moment Frame at 0.04 Radians

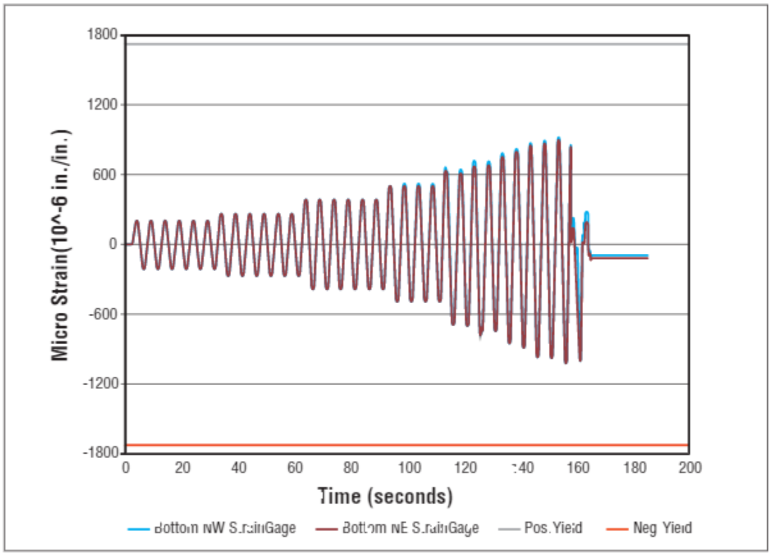


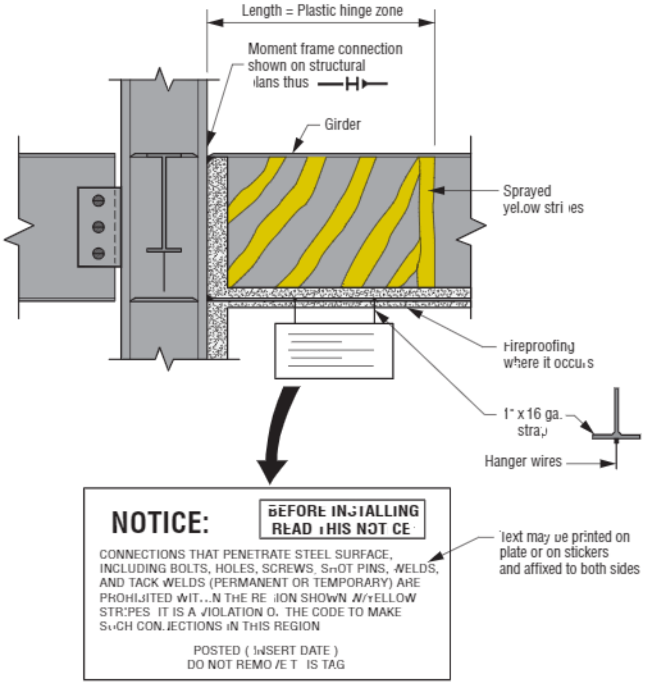
Figure D4a.6 — Measured Strain from Testing at Beam Bottom Flange

Moment Frame Design Requirements and Assumptions (cont.)

D4b. Protected Zones

According to the AISC 341-16 Section E3.5.c, the region at each end of the beam subjected to inelastic straining (plastic hinge formation) shall be designated as a Protected Zone. Each prequalified moment connection in AISC 358-16 has its own section on what is considered a Protected Zone. Figure D4b.1 shows the requirements from the Los Angeles Department of Building and Safety (LADBS). A clear marking denoting the protected zone is required, as well as a sign prohibiting penetrations and welds to this zone as it would negatively affect the performance of the moment connection. AISC Code of Standard Practice for Steel Buildings and Bridges (ANSI / AISC 303-16) also has a similar requirement where the Fabricator shall permanently mark the protected zones designated in accordance with AISC 341-16. If markings are obscured in the field after application of fire protection, then it shall be re-marked.

Figure D4b.2 shows the protected zone for an RBS connection. As can be seen, the protected zone encompasses the beam flange and the beam web, because this is the location where the expected inelastic deformation will occur. This means that during construction, the owner's designated construction representative will have to confirm with the Mechanical, Electrical and Plumbing (MEP) trades that no penetrations will be made through the beam web at these locations. In addition, someone will have to physically mark these locations on each moment connection, as seen in Figure D4b.3.



NOTICE: BEFORE INSTALLING READ THIS NOTICE

CONNECTIONS THAT PENETRATE STEEL SURFACE, INCLUDING BOLTS, HOLES, SCREWS, SHOT PINS, WELDS, AND TACK WELDS (PERMANENT OR TEMPORARY) ARE PROHIBITED WITHIN THE REGION SHOWN W/YELLOW STRIPES. IT IS A VIOLATION OF THE CODE TO MAKE SUCH CONNECTIONS IN THIS REGION

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Figure D4b.1 — LADBS Protected Zone Marking Requirements

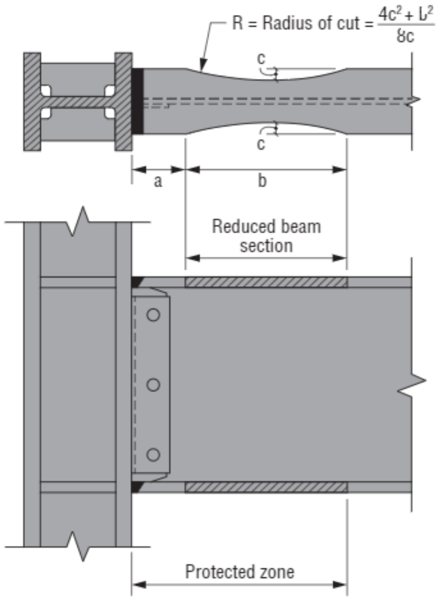


Figure D4b.2 — Protected Zone for an RBS Moment Connection

Figure D4b.3 — Protected Zone Marking for an RBS Connection in the Field (Sprayed on Top of Fireproofing)

Moment Frame Design Requirements and Assumptions (cont.)

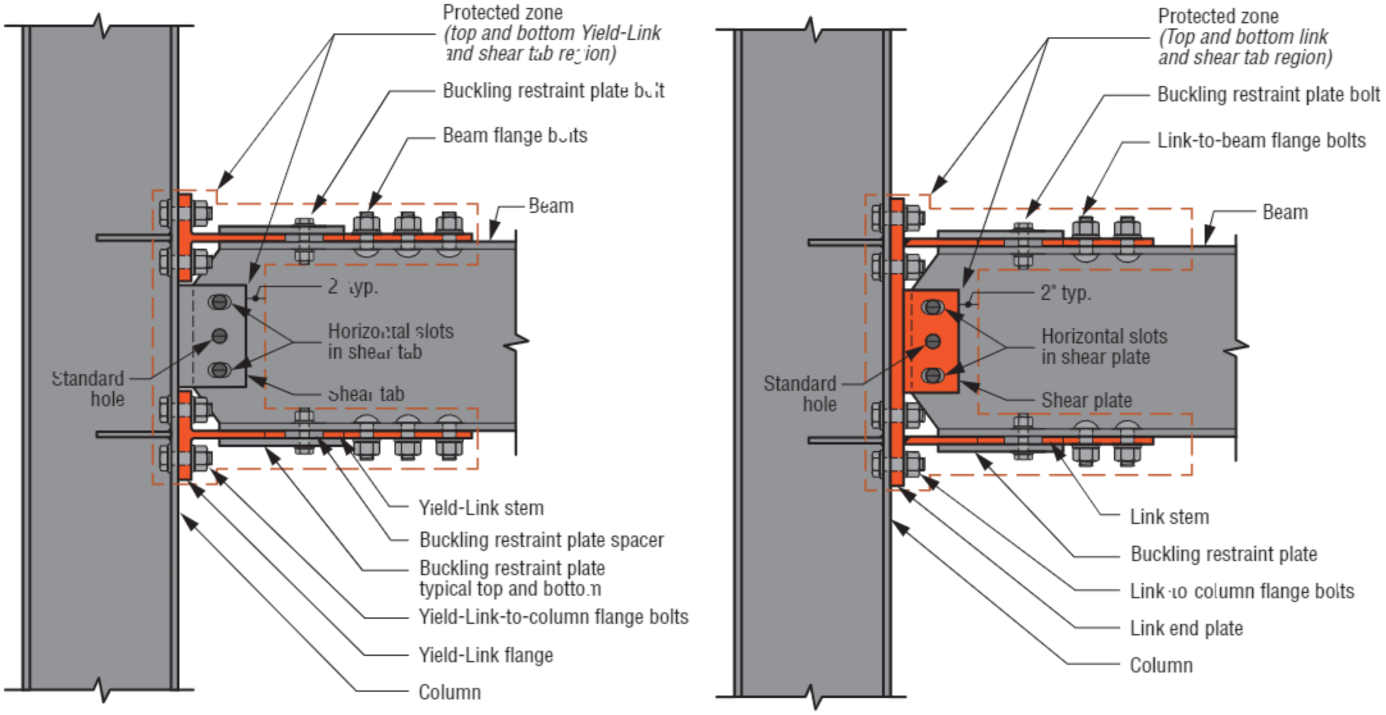
Strong Frame Special Moment Frame Protected Zone

Figure D4b.6 shows the protected zone for the Strong Frame SMF connection. Since the beam is not the yielding element, the protected zone only includes the elements in contact with the Yield-Link® at the beam flanges and shear tab at the beam web.

Note:

1. Protected zone included the following elements:
    - a. Yield-Link flange and Yield-Link stem
    - b. BRP plates
    - c. Beam flange areas connected to the Yield-Link stem
    - d. Column flange areas connected to the Yield-Link flange
    - e. Shear tab and beam web at shear tab (2" around shear plate on 3 sides)
    - f. Yield-Link-to-beam connection bolts
    - g. Yield-Link-to-column connection bolts
    - h. Shear tab-to-beam connection bolts

2. No attachment shall be made to the protected zone.



(a) T-Stub Yield-Links

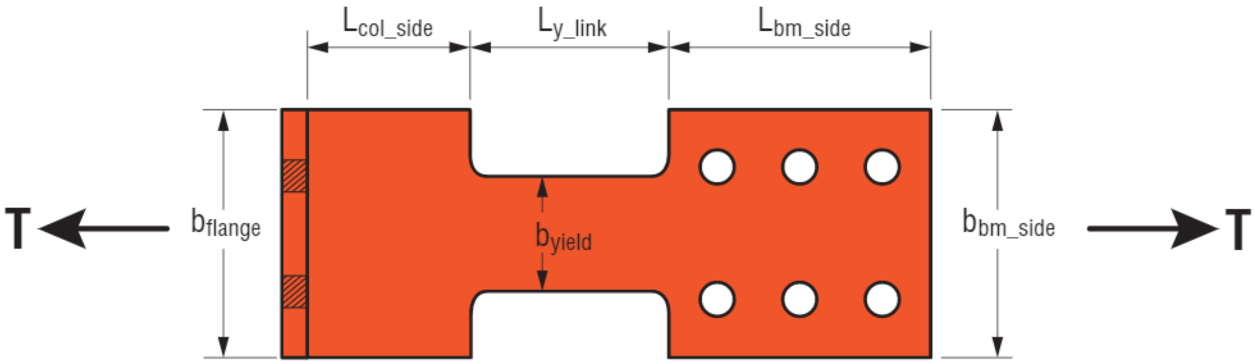
(b) End Plate Yield-Links

Figure D4b.4 — Protected Zone for Strong Frame SMF Connection

Moment Frame Design Requirements and Assumptions (cont.)

D4c. Connection Design

The Strong Frame special moment frame using the Yield-Link® structural fuse incorporates the capacity-based design approach, wherein energy dissipation is confined predominantly within the reduced region of the Yield-Link structural fuse. Member and connection design is based on the maximum probable tensile strength, $P_{r\text{-}link}$, of the reduced region of the Yield-Link (see Figure D4c.1).



(a) Design Parameters

(b) Yield-Link Stretching and Shortening from Testing

Figure D4c.1 — Yield-Link Design for Energy Dissipation

Moment Frame Design Requirements and Assumptions (cont.)

The following are steps for the Strong Frame connection design:

1. Model and analyze moment frame with Yield-Link® moment connections to get demand loads (moment, shear and axial) using code level forces.

2. Design Yield-Link yielding area to resist the maximum axial force from all the standard LRFD load combinations. This means our Yield-Links are designed to remain elastic under code force load combinations including lateral plus gravity loads.

3. Once the yielding area is known, calculate the maximum rupture strength, $P_{r\text{-link}}$, of the Yield-Link as:

   $P_{r\text{-link}} = A_{y\text{-link}} \times R_t \times F_{u\text{-link}}$

   Where:

   $A_{y\text{-link}}$ = area of reduced Yield-Link section, in.^{2}

   R_t = ratio of expected tensile strength to minimum specified tensile strength of the Yield-Link stem material, 1.2

   $F_{u\text{-link}}$ = specified minimum tensile strength of Yield-Link stem material, 65 ksi

   It is worthwhile to point out that we are using R_t and F_u for this calculation where other SMF connections typically use R_y, F_y and a C_{pr} factor that is less than or equal to 1.2. Using R_y of 1.1, R_t of 1.2, F_y of 50 ksi, F_u of 65 ksi and C_{pr} of 1.2. The difference in demand can be seen below:

   Simpson Strong-Tie® Strong Frame SMF Connection Design Demand: 1.2 x 65 ksi = 78 ksi

   Standard SMF Connection Design Demand: 1.1 x 50 ksi x 1.2 = 66 ksi

   The reason for this approach is to truly capture the ultimate strength of our Yield-Link structural fuse, since we want to make sure this is the only region where inelastic action occurs.

4. After $P_{r\text{-link}}$ has been determined, design the rest of the connection to exceed this $P_{r\text{-link}}$ demand load:

   a. Yield-Link stem-to-beam flange connection bolts
   b. Yield-Link flange-to-column flange connection bolts
   c. Yield-Link-flange thickness to prevent prying
   d. Beam-to-column shear tab connection
   e. Column panel zone
   f. Column flange thickness
   g. Stiffener/continuity plate (if required)

Moment Frame Design Requirements and Assumptions (cont.)

D5. Member Design

Similar to the connection design, members (beam and column) are designed for frame mechanism forces, assuming Yield-Links at both ends of the beam are at their maximum probable tensile strength. The beam is designed and tested as unbraced from column to column. There are no requirements for stability bracing of the beams at the Yield-Link® locations. Columns are designed so bracing is only required near the top flange of the beam. Since the frame members are not dissipating energy (i.e., beam plastic hinges do not form), members are designed in accordance with AISC Steel Construction Manual (AISC 360). This means b/t and h/t_w ratios in AISC 341 are not applicable to our beam and column members in the frame when designed using a pinned-base design. However, if the base is designed as fixed or partially fixed, i.e., so the columns may yield at the base, then AISC 341 slenderness ratios will be met for the columns at the base level. Please note, for the Strong Frame column design, the demand forces are from overstrength load combinations. This is similar to other SMF column design, however, for Strong Frame columns, axial + moment interaction check is required, whereas typical SMF column design is permitted to ignore the bending moment (unless the moment results from loads applied between points of support).

Base Plate Design

The capacity design approach also extends to the design of the column base plates. Pinned column base connection demand loads (axial and shear) are calculated from the lesser of the frame mechanism forces and the forces from code overstrength load combinations.

Design capacity for the base plate is calculated from AISC Design Guide #1 (DG #1) and Design Guide #16 (DG #16). Base plate compressive capacity is calculated per DG #1, whereas base plate tension capacity is calculated assuming two-way action using the method in DG #16. Welds in the base plate are checked for shear and tension interaction using capacity-level loads as noted above.

Oversized holes in base plates are required for erection tolerance. Per DG #1 Section 3.5, AISC recommends use of oversized holes for anchor rods. For the Strong Frame, the column base plate holes typically exceed the anchor rod diameter by ¼". When oversized holes are used for erection, considerable slip in the base plate may occur before the plate bears against the anchor rods. In addition, due to anchor placement tolerance and potential for anchor movement during concrete placement, it is not likely that all the anchor rods will resist the same load. AISC DG #1 Section 3.5.3 has two separate recommendations for shear load transfer from the base plate with oversized holes to the anchor rods:

1. Use half of all anchor rods to transfer the shear force at each column.
2. Weld a plate washer with standard oversized holes (+1⁄16")
   to the top of the base plate.

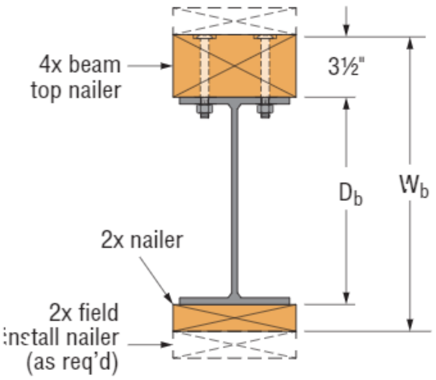
In order to minimize welding at the jobsite, Simpson Strong-Tie currently uses the first approach in our design for the anchor rods in shear. However, the designer can coordinate with Simpson Strong-Tie if they prefer to use the second method. Please note that, for this option, welding and welding inspection are required in the field. The effect of oversized holes in the frame and structural movement shall be evaluated by the designer.

Moment Frame Design Requirements and Assumptions (cont.)

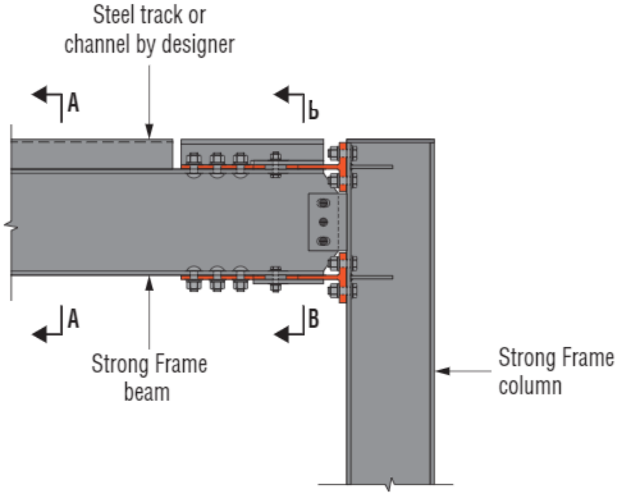
D6. Nailer to Steel Beam Connection Design

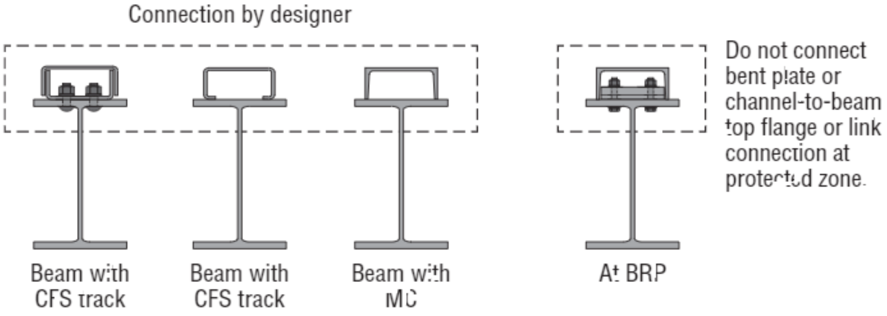
For the shear transfer from the structure to the frame, Simpson Strong-Tie typically provides a 4x wood nailer (for ½" thick Yield-Links) at the top of our steel beam. The 4x wood nailer is then connected to the steel moment frame beam top flange with A325 bolts (Figure D6.1a). Demand load for the nailer to beam top flange utilizes the amplified (Ω_0) forces to make sure adequate strength is provided. For cold-formed steel projects, the 4x nailer can be replaced with light-gauge stud tracks (Figure D6.1b) at the request of the designer. For structural steel projects, the 4x nailer at the roof level can be replaced with a bent plate or a channel section (Figure D6.1b and D6.1c) to make up the 3.5" difference between the top of the column cap plate and the top of the steel beam. For other Yield-Link® thickness models, see wood nailer and beam top flange to top of Yield-Link flange height requirements on the frame elevation drawings on **strongtie.com**.

In addition to shear transfer through the beam top flange, shear can also be transferred to the frame from the columns. A typical detail would be from a shear plate connection or a hanger welded to the face of the columns (Figure D6.2 on p. 39). Coordinate shear transfer to frame with Simpson Strong-Tie for any special requirements.



(a) Standard 4x Wood Nailer on Top of Steel Beam

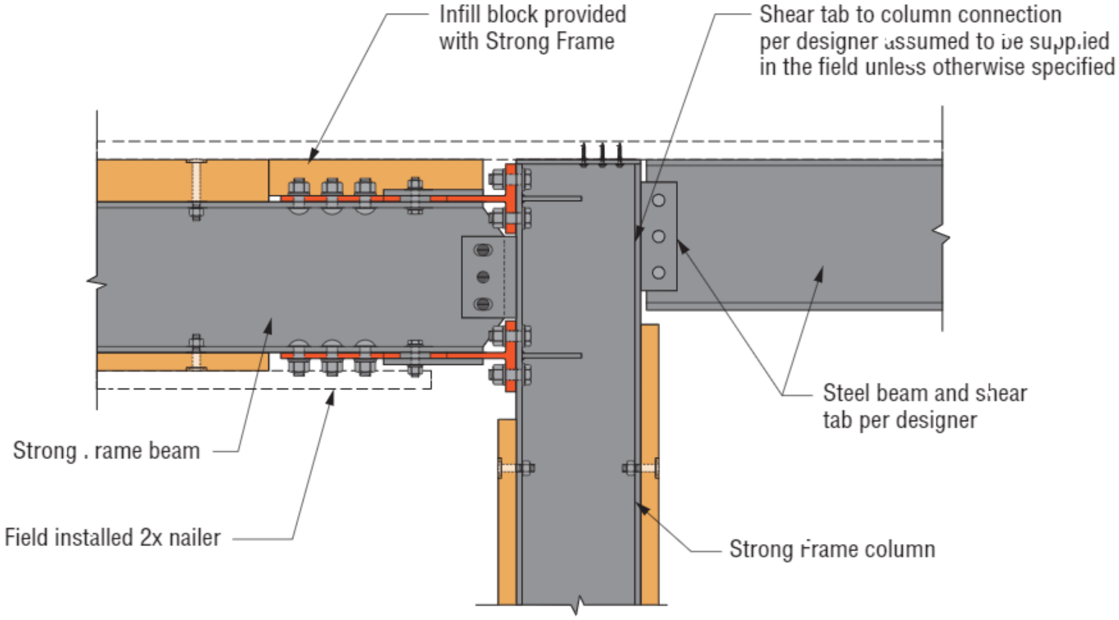




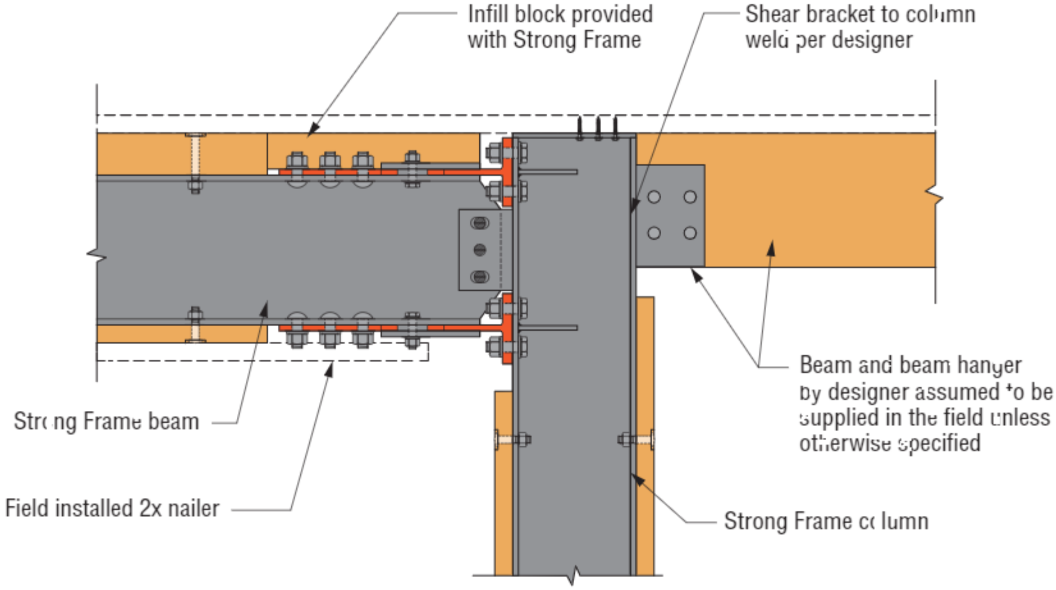
(b) Cold-Formed Steel Stud (c) Steel Bent Plate or Channel

Figure D6.1 — Shear Transfer Connection Options at Top of Steel Beam

Moment Frame Design Requirements and Assumptions (cont.)



(a) Shear Transfer to Column from Shear Plate



(b) Shear Transfer to Column from Welded Bracket

Figure D6.2 — Shear Transfer to Moment Frame Through the Column

Moment Frame Design Requirements and Assumptions (cont.)

D7. Base Fixity Design

Column base fixity has a considerable effect on the performance of moment frames. Currently, engineers assume either a fixed-base connection (Figure D7.1) or a pinned-base connection (Figure D7.4) in the analysis of moment frames. In reality, the performance of the connection is in between the two limits. Figure D7.2 shows the AISC definition of a fixed, a pinned and a partially restrained (PR) connection in a graphical format. Connections are considered fixed when the moment vs. rotation stiffness is greater than 20 EI/L of the member, whereas a connection is considered pinned (simple) when the stiffness value is less than 2 EI/L.

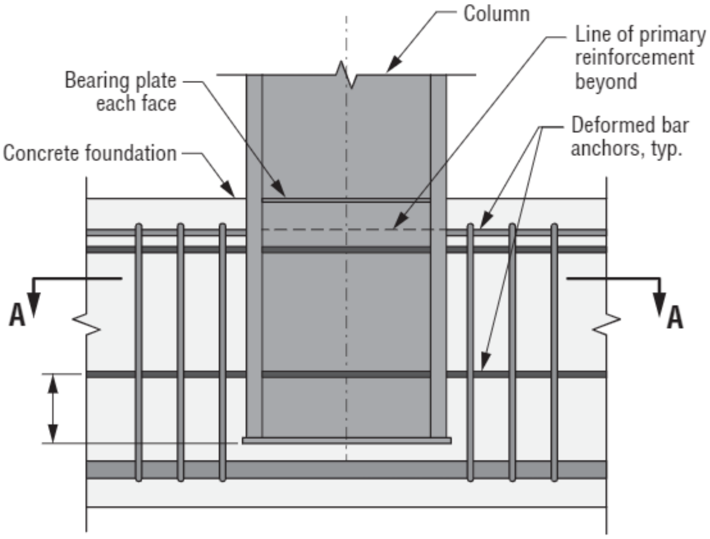


Figure D7.1 — Fixed-Base (FR) Connection in AISC Seismic Design Manual

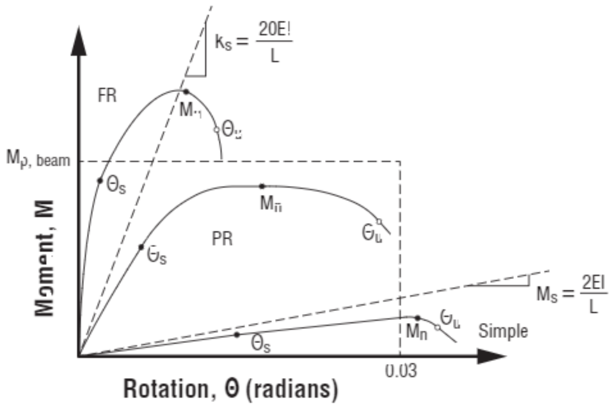


Figure D7.2 — Connection Classification per AISC 360-10

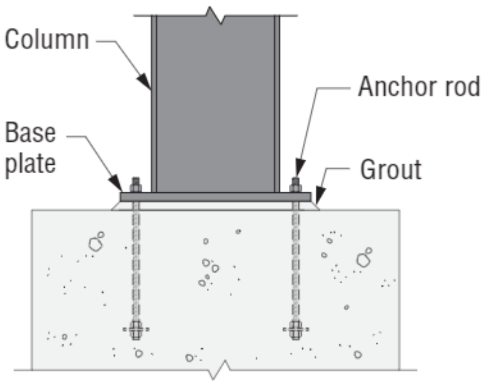


Figure D7.3 — PR Base Connection in AISC Design

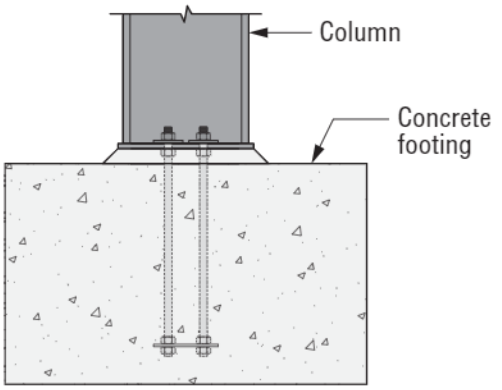


Figure D7.4 — Pinned-Base (Simple) Connection in AISC Design Guide #1

Moment Frame Design Requirements and Assumptions (cont.)

Table 1 below shows the effects of base fixity on the different performance parameters. Pinned column bases will have a higher drift and a higher k-value for column design. However, they will have lower floor accelerations than columns with a fixed-base connection. A partially restrained base will behave somewhere in between pinned and fixed bases. Compared to a frame with pinned-base connections, a frame with P.R bases will have less drift, higher base shear and higher floor accelerations.

Table 1 — Performance Effects from Different Base Fixities

Performance Parameters	Fixed	Partially Restrained	Pinned
Base Reaction	High	Medium	Low
Drift	Low	Medium	High
Floor Acceleration	High	Medium	Low
Column Design K-Value	Low	Medium	High
Beam Axial Load	High	Medium	Low

Strong Frame Special Moment Frame Base Fixity

The Strong Frame typical base fixity assumption is a pinned column base. Reactions for a pinned-base connection consist of axial and shear only. If a fixed-base connection is used, then the designer will need to address the moment in the foundation design. For fixed-base connections, we currently use the embedded column approach. Contact Simpson Strong-Tie for available non-embedded options.

D8. Anchorage Design

Anchorage to Concrete

In addition to the steel frame design, Simpson Strong-Tie also offers anchorage design. We have two solutions for anchorage of the column bases to concrete:

1. MFSL — The MSFL anchorage assembly comes with a preattached shear lug, so no field-bent ties or hairpins are required for shear capacity (see Figure D8.1).
2. MFAB — The MFAB assembly requires field-installed ties or hairpins, but also provides higher shear capacity depending on the amount of reinforcing provided (see Figure D8.2).

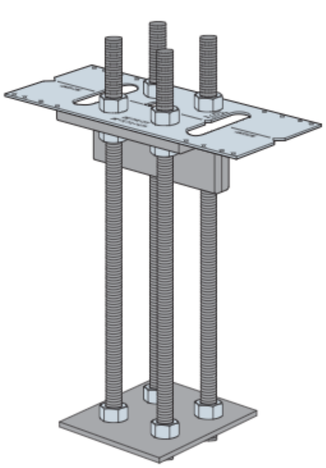
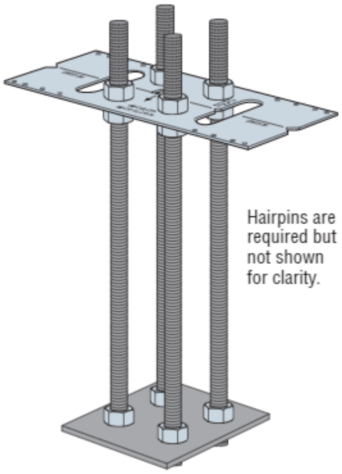


Figure D8.1 — MFSL Anchorage Assembly

US Patent 8,336,267 B2



Hairpins are required but not shown for clarity.

Figure D8.2 — MFAB Anchorage Assembly

Moment Frame Design Requirements and Assumptions (cont.)

Anchorage Design Notes

The steel-strength calculations for anchor shear and anchor tension are per ACI 318-11 (2012 IBC) and ACI 318-14 (2015/2018 IBC). Tension and shear anchorage are designed as follows:

Element	2012 IBC Code Section	2015/2018 IBC Code Section
Anchor rod steel strength in tension	ACI 318-11, D.5.1	ACI 318-14, 17.4.1
Anchor breakout strength in tension	ACI 318-11, D.5.2	ACI 318-14, 17.4.2
Anchor pullout strength in tension	ACI 318-11, D.5.3	ACI 318-14, 17.4.3
Anchor rod steel strength in shear	ACI 318-11, D.6.1	ACI 318-14, 17.5.1
Embedded plate bending strength	AISC Chapter F	AISC Chapter F
Concrete shear strength — shear lug	AISC Design Guide 1	AISC Design Guide 1
Concrete shear strength — tied anchorage	ACI 318, Chapter 10	ACI 318, Chapter 10

Anchorage designs are based on LRFD loads. For designs under the 2012 IBC, tension anchorage for seismic loads complies with ACI 318 Appendix D. The design strength is governed by the maximum tension that can be transmitted to the anchors by the frame capacity or the maximum tension obtained from design loads combinations that include E, with E increased by Ω_O. (Section D.3.3.4.3 with modifications contained in 2012 IBC section 1908.1.16.)

For designs under the 2015/2018 IBC, tension anchorage for seismic loads complies with ACI 318-14 Chapter 17. The design strength is governed by the maximum tension that can be transmitted to the anchors by the frame capacity or the maximum tension obtained from design loads combinations that include E, with E increased by Ω_O. (Section 17.2.3.4.3 with modifications contained in 2015 IBC section 1908.1.16.)

For strength calculation, strength reduction factors in tension are based on:

- Seismic Design Category, $\phi_{seismic} = 0.75$
- Crack/Uncrack Concrete factor, $\phi_{conc} = 0.70$

Strength reduction factor in shear included:

- Grout pad reduction factor = 0.8

Simpson Strong-Tie® Strong Frame Anchorage design calculates anchor bolt shear and tension interaction above the concrete using the AISC 360 bolt interaction equation. However, for capacity within the concrete, anchorage designs are based on anchor embedment into the foundation for tension, while shear design is based on the resistance within the curb or slab. The designer must consider shear and tension interaction of the concrete if failure surfaces overlap. If this failure mode occurs, we recommend providing supplemental reinforcing to transfer the shear forces into the concrete. Where a greater end distance is required, the designer should specify this on their plans. Additional studs can be specified to increase this end distance.

Calculations for the anchorage are provided and typically assume a cracked concrete design based on ACI 318 with no supplementary reinforcing and a centered square pad. Alternate design and detailing of anchorage can be specified by the designer as well.

Moment Frame Design Requirements and Assumptions (cont.)

Inspection Requirements

Since the entire Simpson Strong-Tie® Strong Frame special moment frame is designed to be field bolted, no field welding is required. Welding for the frames is performed on the premises of a fabricator registered and approved in accordance with 2015/2018 IBC Section 1704.2.5. Special inspections prescribed in IBC Section 1704 are not required for approved fabricators. Nevertheless, all Strong Frame special moment frames are inspected by a certified welding inspector. Inspection is also provided for the pretensioned bolts between the Yield-Link® stem-to-beam flange connections on top of the code inspection requirements. Welding and bolting inspection documents as well as bolt preinstallation testing records can be obtained from Simpson Strong-Tie at the request of the project designer or by scanning the QR code on the frame at the jobsite.

Even though the Strong Frame can be field bolted and all field bolting is specified as snug tight, the latest IBC code references AISC 360 and AISC 341 for bolting inspection requirements. AISC requires inspection prior to, during and after bolting similar to welding inspections, although not much is required during snug-tight bolt installation.

In addition to field-bolting inspection, different building jurisdictions might have base plate grouting inspection requirements. Please consult with your project building jurisdiction about this requirement.

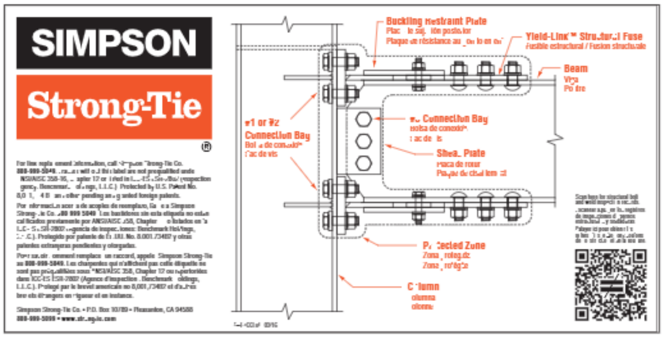
Frame Inspection

Simpson Strong-Tie Strong Frame special moment frames have had all required special inspections performed and are built in a factory environment under strict quality-control measures as required under AISC 341, AWS D1.1 and AWS D1.8.

All factory welds for the Strong Frame special moment frame are inspected and documented by a Certified Welding Inspector.

Special Moment Frame
• Column shear tab weld
• Column stiffener plate weld
• Column cap plate weld
• Column base plate weld
• Yield-Link stem-to-beam flange bolts

In addition to welding, structural ASTM A325 as well as F2280 twist-off type high-strength bolts are lot tested and stored under requirements of the Research Council on Structural Connections (RCSC). Bolting of the SMF Yield-Link® structural fuse to the beam flanges (ASTM 3125 Grade F2280 [A490-TC] Bolts) are documented.

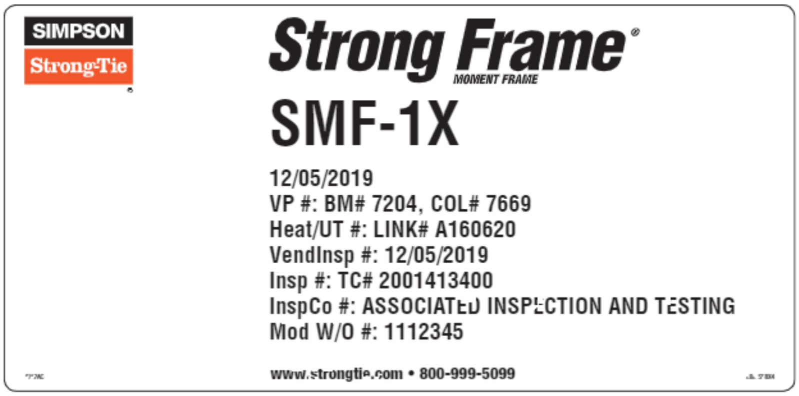


Special Moment Frame QR Code Label

Moment Frame Design Requirements and Assumptions (cont.)

Lot Inspection for Tension Controlled Bolts with DTI Washers

The structural fastener assembly lots are randomly sampled. The samples are tested to the preinstallation verification requirements for pretension bolts conforming with *AISC Steel Construction Manual 14th Edition*. Bolting and welding inspection reports and material certifications for any individual frame are available by contacting Simpson Strong-Tie with the work order number listed on the frame stickers or by scanning the QR code on the Strong Frame moment frames and entering the work order number.



SIMPSON Strong-Tie®

Strong Frame®
MOMENT FRAME

SMF-1X

12/05/2019
VP #: BM# 7204, COL# 7669
Heat/UT #: LINK# A160620
VendInsp #: 12/05/2019
Insp #: TC# 2001413400
InspCo #: ASSOCIATED INSPECTION AND TESTING
Mod W/O #: 1112345

www.strongtie.com • 800-999-5099

Special Moment Frame Label

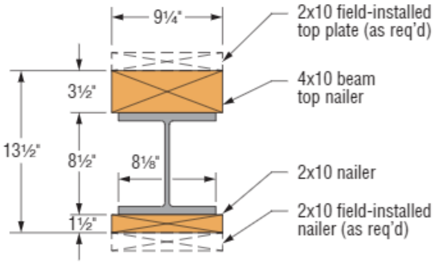
During the frame installation, some special inspections might be necessary depending on jurisdictional requirement; please contact your project's building department for specific requirements. In the table below are some of the inspections that may be required:

Special Moment Frame
• Yield-Link®-flange-to-column snug-tight bolting
• Beam web-to-column shear-plate snug-tight bolting
• Column base plate grouting
• Column splice pretensioned bolting (when used)

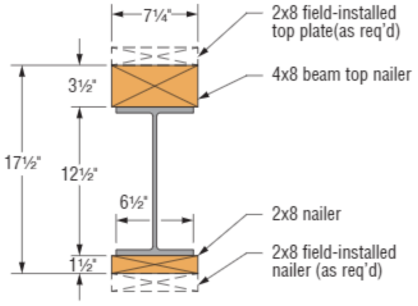
Simpson Strong-Tie® Strong Frames Special Moment Frame Product and Service Offering

1. SMF Beam Sections

Standard AISC W-Section Beams



B8



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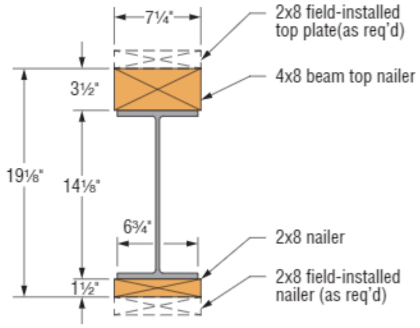

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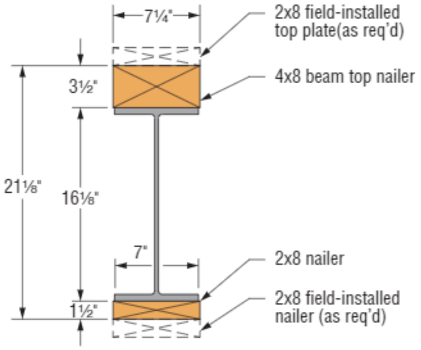


B16

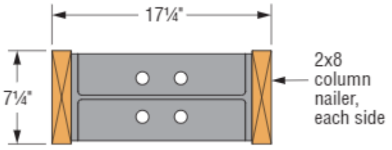
Model No.	Beam Size	Yield-Link Types	Wall Width
SMF-B12	W12x35	T-Stub	2x8
SMF-B14	W14x38	T-Stub	2x8
SMF-B16	W16x45	T-Stub	2x8
SMF-B8	W8x48	EPL	2x10

Note: Other AISC W-Section beams available. Contact Simpson Strong-Tie for more information.

Simpson Strong-Tie® Strong Frame Special Moment Frame Product and Service Offering (cont.)

2. SMF Column Sections

AISC Standard Structural Shapes W-Section Columns



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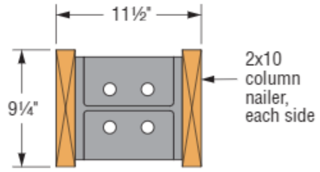

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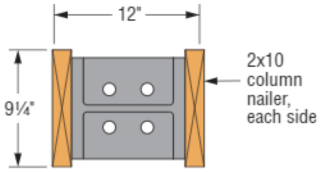

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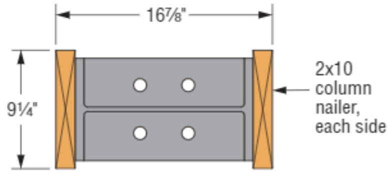

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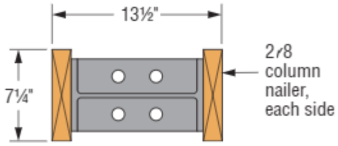

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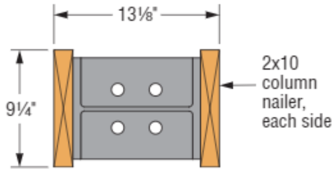

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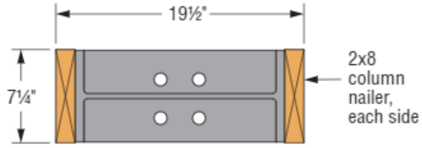

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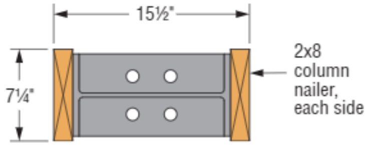

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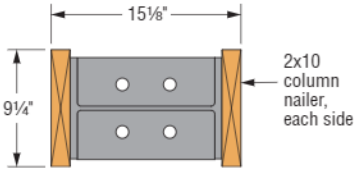

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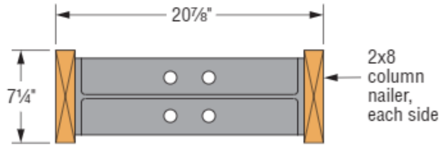

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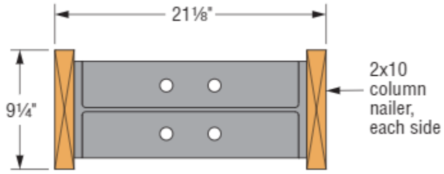


C18B

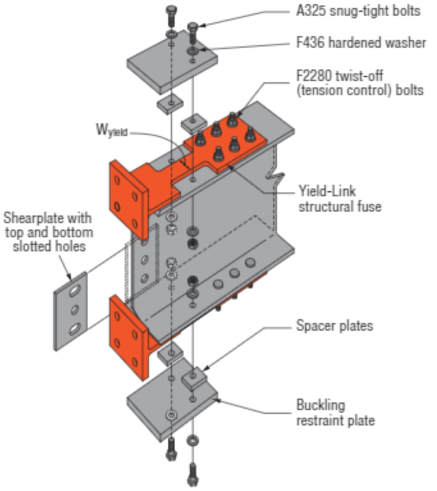
Model No.	Column Section	Yield-Link Types	Wall Width	Anchorage Type Kit
SMF-C10	W10x30	T-Stub	2x8	MFSL, MFAB
SMF-C12	W12x35	T-Stub	2x8	MFSL, MFAB
SMF-C14	W14x38	T-Stub	2x8	MFSL, MFAB
SMF-C16	W16x57	T-Stub	2x8	MFSL, MFAB
SMF-C18A	W18x40	T-Stub	2x8	MFSL, MFAB
SMF-C8A	W8x48	EPL, T-Stub	2x10	MFSL, MFAB
SMF-C8B	W8x67	EPL, T-Stub	2x10	MFSL, MFAB
SMF-C10B	W10x45	EPL, T-Stub	2x10	MFSL3.75, MFAB3.75
SMF-C12B	W12x45	EPL, T-Stub	2x10	MFSL3.75, MFAB3.75
SMF-C14B	W14x53	EPL, T-Stub	2x10	MFSL3.75, MFAB3.75
SMF-C18B	W18x55	EPL, T-Stub	2x10	MFSL3.75, MFAB3.75

Note: Other AISC W-Section beams available. Contact Simpson Strong-Tie for more information.

Simpson Strong-Tie® Strong Frame Special Moment Frame Product and Service Offering (cont.)

3. Yield-Link® Structural Fuse

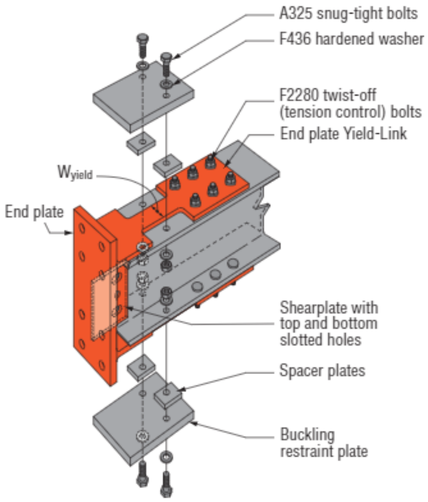
The standard Strong Frame moment connection consists of two modified T-stub Yield-Links; one on top of the beam and one on the bottom (see Figure 1a). For shallow beams with 8.5" overall steel depth, Simpson Strong-Tie developed an SMF connection with end-plate Yield-Links (see Figure 1b). Other than the design of the Yield-Link end-plate and end-plate-to-column flange bolts, the connection design procedure for the end-plate Yield-Link moment connection is identical to our existing two-piece Yield-Link moment connections.



Factory-Installed Strong Frame Yield-Link Structural Fuse Special Moment Frame Joint

Figure 1a — Two-Piece T-Stub Yield-Link

Note: Quantity of bolts will vary depending on design requirements.



Factory-Installed Strong Frame End Plate Yield-Link Special Moment Frame Joint

Figure 1b — End Plate Yield-Link

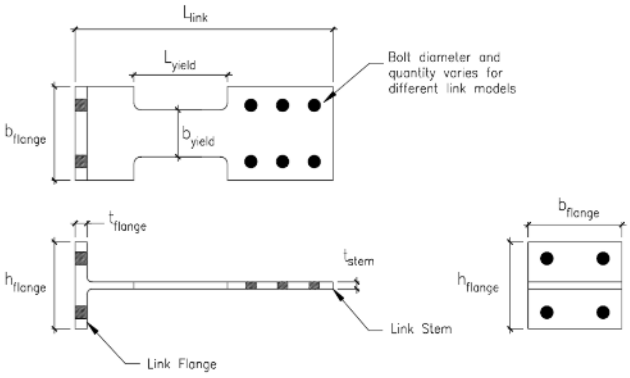
SMF Yield-Link Types

Two Piece T-Stub Yield-Link

Yield-Link ID	t_{stem}	L_{link}	b_{flange}	h_{flange}	Yield-Link Flange Bolt Size
	in.	in.	in.	in.	in.
X1.75	0.5	17.6250	6.5	5.75	0.875
X2.0					0.875
L					0.875
M					0.875
H		20.3750	7.0		0.875
MF4-2.25	0.5	18.6250	7.0	5.75	0.875
MF4-2.875					0.875
MF4-3.5		21.3750	8.0		0.875
MF4-3.75					0.875
MF4-4					0.875
MF6-3	0.75	27.5000	8.0	9.25	1.000
MF6-3.5					1.000
MF6-4					1.250

End Plate Yield-Link

Yield-Link ID	t_{stem}	L_{link}	b_{flange}	Yield-Link Flange Bolt Size
	in.	in.	in.	in.
EL	0.5	17.6250	6.5	0.875
EM				0.875
EH		20.3750	7.0	0.875

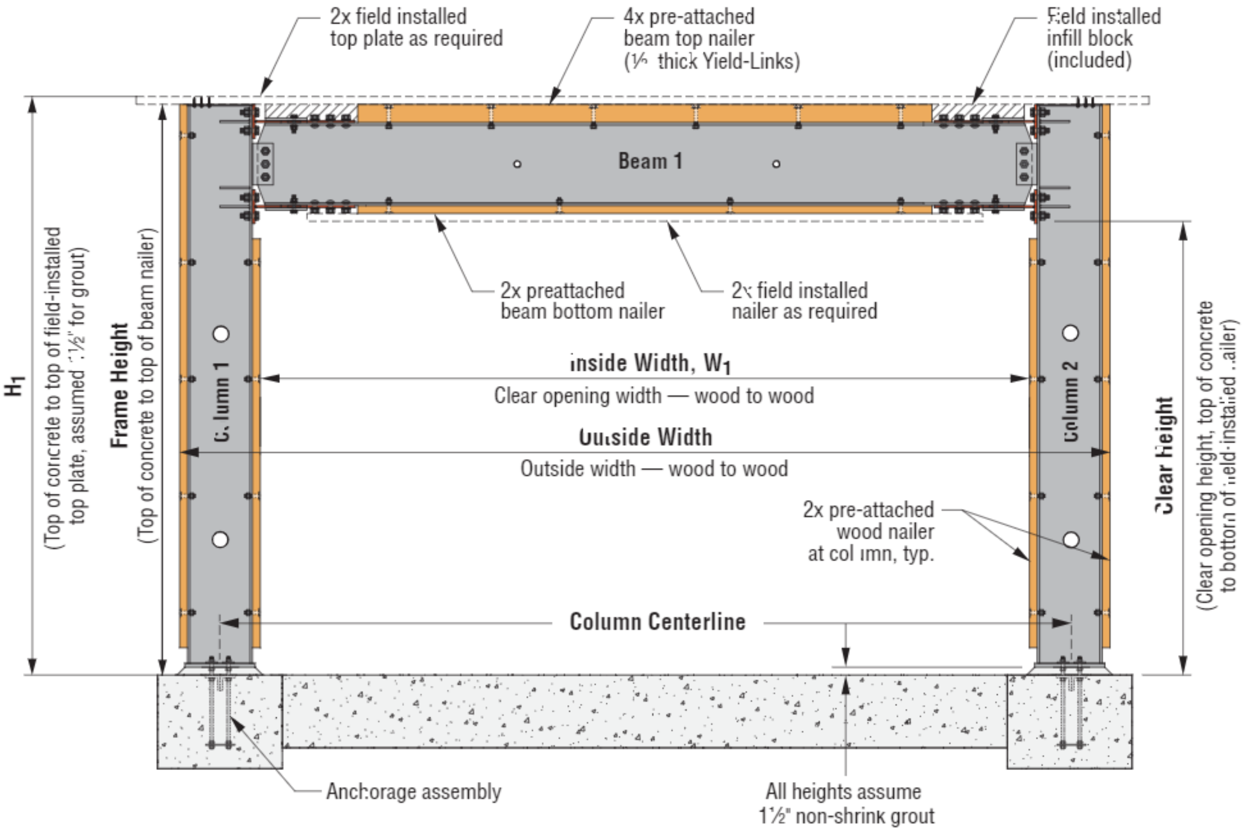


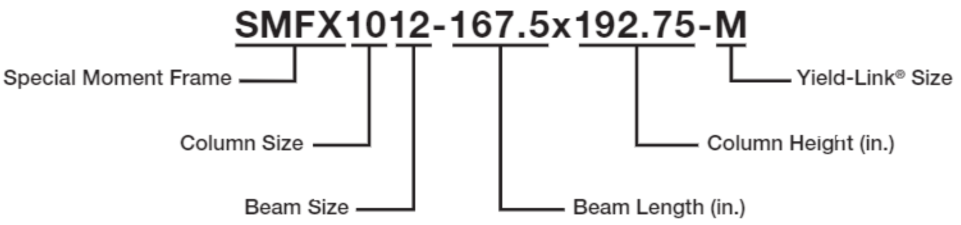
Simpson Strong-Tie® Strong Frame® Special Moment Frame Product and Service Offering (cont.)

4. Strong Frames

Combining the beam, column and Yield-Link® sections, the Strong Frame special moment frames are offered in a variety of frame combinations, ranging from one-story, one bay frames to multi-story, multi-bay frames.

4.1 One-Story x One-Bay Frames





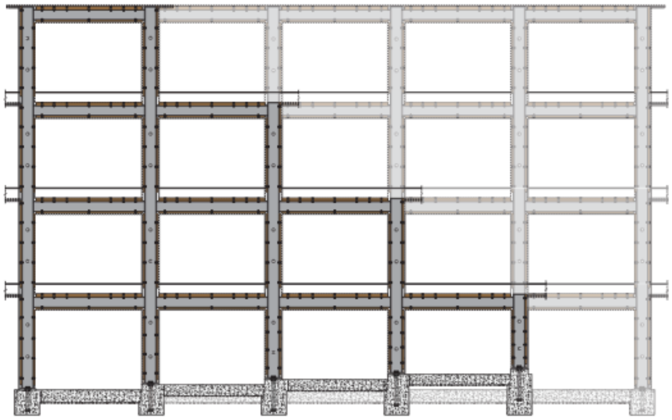
Model No. Naming Legend

Simpson Strong-Tie® Strong Frame® Special Moment Frame Product and Service Offering (cont.)

The special moment frame has proven to perform exceptionally well in structures of up to four stories. This added capability gives designers many possibilities for designing larger structures that are both structurally sound and aesthetically pleasing.

4.2 Multi-Story x Multi-Bay Frames

Four-story design



Strong Frame special moment frames can be used in varying designs, including a four-story and four-bay stepdown custom special moment frame design with first-floor uneven column height

Two-story X two-bay moment frame design

Three-bay design

Strong Frame Design Options

Fixed-Column Base Design Option

Simpson Strong-Tie offers fixed-base frame design. Moment frame performance differences between pinned column bases and fixed column bases were discussed previously in Section D7. In order to have a fixed-base connection, footing design needs to account for the added moment as well as the stiffness required to perform as a fixed base. Currently Simpson Strong-Tie uses the embedded column base approach (see Figure 1), similar to design Example 4.4.4 in the AISC Seismic Design Manual (2nd Edition). Consult with Simpson Strong-Tie for available non-embedded options (see Figure 3). By designing the column as a fixed base, designers should be aware that the fixed-base connection will be stiffer than beam-to-column moment connections, and yielding may occur at the base of the column. Once plastic hinges are formed at the column base, the frame will behave as a frame with a rotational spring base.

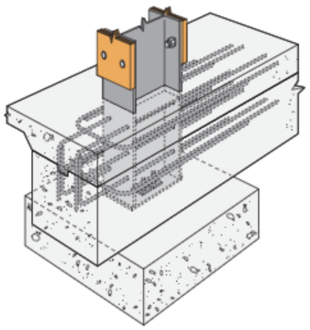


Figure 1 — Fixed-Base Connection (Embedded Option)

Spliced-Column Design Option

Designers can coordinate with Simpson Strong-Tie if column splices are required to facilitate erection. Column splice solutions offered by Simpson Strong-Tie are all field bolted. DTI washers are provided with the connection kit (see Figure 2).

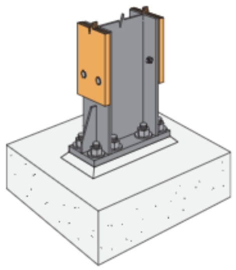


Figure 2 — Non-Embedded PR Fixed Base

Pushover Curves for FEMA P-807 or ASCE 41

The Weak Story Tool with Simpson Strong-Tie® Strong Frame Moment Frames can provide pushover (load vs. deflection) curves for one-story, one-bay frames. If pushover curves are required for other configurations, Simpson Strong-Tie can provide these at the request of the designer. More information can be found in the *Soft-Story Retrofit Design Guide* (F-L-SSRG).

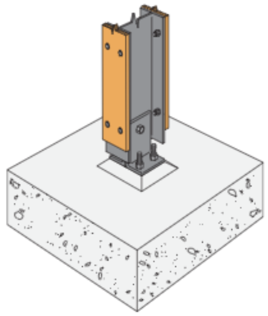
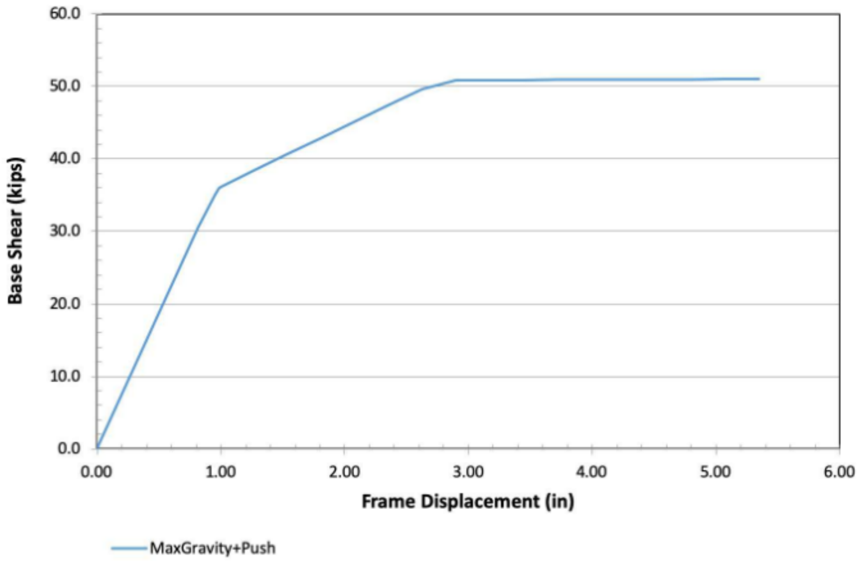


Figure 3 — Single Bolt Pinned Base



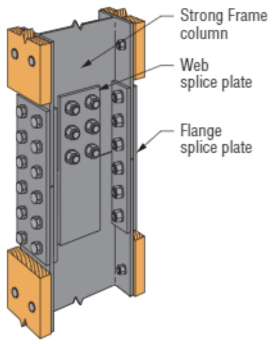


Figure 4 — Spliced-Column Design Option