

# ICC-ES Evaluation Report

ESR-2802

Reissued January 2024

This report also contains:

- LABC Supplement

Subject to renewal January 2025

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<p><b>DIVISION: 05 00 00—</b> <b>METALS</b></p> <p><b>Section: 05 12 00—</b> <b>Structural Steel Framing</b></p>	<p><b>REPORT HOLDER:</b> <b>SIMPSON STRONG-TIE</b> <b>COMPANY INC.</b></p> 	<p><b>EVALUATION SUBJECT:</b> <b>SIMPSON STRONG-TIE®</b> <b>STRONG FRAME®</b> <b>STEEL SPECIAL</b> <b>MOMENT FRAME</b> <b>CONNECTION</b></p>	
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## 1.0 EVALUATION SCOPE

### Compliance with the following codes:

2021, 2018, 2015, 2012, and 2009 [International Building Code® \(IBC\)](#)

For evaluation for compliance with codes adopted by the [Los Angeles Department of Building and Safety \(LADBS\)](#), see [ESR-2802 LABC Supplement](#).

### Property evaluated:

Structural design

## 2.0 USES

The Strong Frame® Steel Special Moment Frame Connection is used for beam-to-column moment connections in structural steel special moment frame (SMF) and steel intermediate moment frame (IMF) systems utilizing the Yield-Link™ Structural Fuse (or T-link) including two-piece T-stub Yield-Link and end-plate Yield-Link.

## 3.0 DESCRIPTION

### 3.1 General:

The Strong Frame® Steel Special Moment Frame Connection provides beam-to-column moment connections for use in steel special moment frame (SMF) and steel intermediate moment frame (IMF) systems. The Simpson Strong-Tie moment connection system satisfies all applicable requirements under the 2021, 2018, 2015, 2012 and 2009 IBC; [ANSI/AISC 341-16](#), including Section E2, Section E3, and Section K1 under the 2021 and 2018 IBC; [ANSI/AISC 341-10](#), including Section E2, Section E3, and Section K1 under the 2015 and 2012 IBC; and [ANSI/AISC 341-05](#), including Section 9, Section 10, and Appendix P under the 2009 IBC. This system also complies with the prequalification requirements under Sections 3.1 and 3.4 of the ICC-ES Acceptance Criteria for Steel Moment Frame Connection Systems (AC129), for both new and retrofit construction applications. The connection system is constructed of a single plate shear connection at the beam web and a buckling-restrained Yield-Link™ Structural Fuse yielding connection at each beam flange. Illustrative details are provided in [Figures 1](#) through [3](#).

### 3.2 Materials:

**3.2.1 Structural Shapes:** Beams and columns must be rolled wide-flange or welded built-up I-shaped and cruciform shaped members. Hot-rolled steel shapes must conform to [ASTM A992](#). Charpy V-Notch (CVN)

toughness for hot-rolled shapes with flanges 1.5 inches (38 mm) thick or thicker must conform to Section A3 of ANSI/AISC 341-16 under the 2021 and 2018 IBC or Section A3 of ANSI/AISC 341-10 under the 2015 and 2012 IBC or Section 6.3 of ANSI/AISC 341-05 under the 2009 IBC. Plates for built up I-shape material must conform to Section 3.2.2.

**3.2.2 Plates:** Plate material for the Yield-Link™ Structural Fuse stem, Yield-Link™ Structural Fuse flange or end-plate, and buckling restraint plate must conform to the requirements of [ASTM A572](#) Grade 50. For Links cut from W-sections, steel shapes must conform to Section 3.2.1. In addition, the maximum ratio of actual yield strength to actual tensile strength for I-shape material is 0.85. Plate material for the shear tab, buckling restraint spacer plates, built-up I-shape plates, stiffener plates, cap plate and base plate must conform to [ANSI/AISC 360](#) Section A3. In addition, CVN toughness of plates 2 inches (51 mm) thick or thicker must conform to Section A3 of ANSI/AISC 341-16 under the 2021 and 2018 IBC or Section A3 of ANSI/AISC 341-10 under the 2015 and 2012 IBC or Section 6.3 of ANSI/AISC 341-05 under the 2009 IBC.

**3.2.3 Bolts, Washers and Nuts:** Bolts, washers and nuts must conform to Section A3.3 of AISC 360 and [AISC 348](#). Bolts for the Yield-Link™ Structural Fuse stem-to-beam flange connection must comply as [ASTM A325](#) Type 1 or [A490](#) Type 1 structural bolts, or as [ASTM F1852](#) or [F2280](#) twist-off type structural bolt assemblies. Bolts for other connections must comply with ASTM A325 Type 1.

**3.2.4 Welds:** Welding processes for all welds must comply with Clause 6.5 of [ANSI/AWS D1.8-16](#) under the 2021 and 2018 IBC or Clause 6.2 of [ANSI/AWS D1.8-09](#) under the 2015 and 2012 IBC or Clause 6.2 of [ANSI/AWS D1.8-05](#) under the 2009 IBC. All welding must be performed using minimum E70xx electrodes. All weld filler metals must comply with Clauses 6.1, 6.2 and 6.3 of ANSI/AWS D1.8-16 under the 2021 and 2018 IBC or Clause 6.3 of ANSI/AWS D1.8-09 under the 2015 and 2012 IBC or Clause 6.3 of ANSI/AWS D1.8-05 under the 2009 IBC, along with Charpy V-Notch (CVN) toughness requirements set forth in Section A3.4 of ANSI/AISC 341-16 under the 2021 and 2018 IBC or Section A3.4 of ANSI/AISC 341-10 under the 2015 and 2012 IBC or Section 7.3, Appendix W and Appendix X of ANSI/AISC 341-05 under the 2009 IBC.

## 4.0 DESIGN AND INSTALLATION

### 4.1 Structural Design and Prequalification Limits:

Steel special moment frames using the Strong Frame® Special Moment Frame Connection must be designed and detailed in accordance with the structural *Design Procedure* contained in Annex A of this evaluation report and are subject to the limitations therein. For determining seismic loads, the system seismic performance coefficients and factors for the IBC are permitted to be as follows:

SEISMIC SYSTEM	RESPONSE MODIFICATION COEFFICIENT, R	OVERSTRENGTH FACTOR, $\Omega_0$	DEFLECTION AMPLIFICATION FACTOR, $C_d$
SMF	8	3	5 <sup>1/2</sup>
IMF	4 <sup>1/2</sup>	3	4

\*Seismic force-resisting system as defined in [ASCE/SEI 7](#), Table 12.2-1, must conform to limitations in IBC and ASCE/SEI 7, including provisions for structural system limitations including structural height noted in Table 12.2-1 of ASCE/SEI 7.

In addition, compliance with the American Welding Society Structural Welding Code—Steel ([ANSI/AWS D1.1](#)), Section 2, with modifications as set forth in AISC 360 Section J2, is required.

For the Strong Frame® Special Moment Frame Connection, a partially restrained (Type PR) connection, the seismic analysis must include the force-deformation characteristics of the specific connection in accordance with the *Design Procedure* in Annex A of this evaluation report.

#### 4.1.1 Beam Limitations: Beams must satisfy the following limitations:

1. Beams must be rolled wide-flange or welded built-up I-shaped members.
2. For used with two-piece T-stub Yield-Links, beam depth is limited to a maximum of W36 for rolled shapes and beam depth for built-up members must not exceed the maximum depth permitted for W36 shapes.
3. For use with End-plate Yield-Links, beam depth must be 8.5 in. (215.9 mm).
4. The beam flange and web width-to-thickness ratios must not exceed  $\lambda_r$  in accordance with Table B4.1 of ANSI/AISC 360. Flange thickness shall be designed per Step 11 in Annex A and must not be less than 0.40 inch (10 mm).

5. There is no limit on the weight per length (feet or meters) of beams.

#### 4.1.2 Column Limitations:

1. Columns must be rolled wide-flange or welded built-up I-shaped or cruciform shaped members.
2. The beam must be connected to the flange of the column.
3. For used with two-piece T-stub Yield-Links, column depth is limited to a maximum of W36 for rolled shapes and column depth for built-up members must not exceed the maximum depth permitted for W36 shapes.
4. For used with End-plate Yield-Links, column depth is limited to a minimum of 6 in. (152.4 mm) and a maximum of W18 for rolled shapes, and column depth for built-up members must be within the same depth range permitted for rolled shapes. Flanged cruciform columns must not have a width or depth greater than the depth allowed for rolled shapes.
5. There is no limit on the weight per length (feet or meters) of columns.
6. There are no additional requirements for flange thickness.
7. Columns with fixed base connections must satisfy the ANSI/AISC 341 limiting width-to-thickness ratio requirements for flanges and webs of columns of SMFs (i.e., column width-to-thickness ratios must satisfy ANSI/AISC 341-16 Table D1.1 for highly ductile members under the 2021 and 2018 IBC or ANSI/AISC 341-10 Table D1.1 for highly ductile members under the 2015 and 2012 IBC or ANSI/AISC 341-05 Table I-8-1 for seismically compact members under the 2009 IBC within the first story; at locations other than first story, column width-to-thickness ratios must satisfy limiting width-to-thickness ratios in Table B4.1 of ANSI/AISC 360). For columns with non-fixed base connections, the width-to-thickness ratios of flanges and webs of columns must satisfy limiting width-to-thickness ratios in Table B4.1 of ANSI/AISC 360.
8. For cruciform columns with fixed base connections, the elements of flanged cruciform columns, whether fabricated from rolled W-Sections or built-up sections using plates and W-sections, shall meet the requirements of the AISC Seismic Provisions per item 7 above.

#### 4.1.3 Buckling Restraint Assembly Requirements:

1. The Yield-Link™ Structural Fuse stem must be 1.0 in (25 mm) thick maximum and 0.5 inch (12.7 mm) thick minimum for the two-piece T-stub Yield-Link, and must be 0.5 inch (12.7 mm) thick for the end-plate Yield-Link.
2. The maximum width of the reduced portion of the T-link stem must be 3.5 inches (88.9 mm) for the end-plate Yield Link, and 6 inches (152 mm) for the two-piece T-stub Yield Link.
3. The buckling restraint plate (BRP) thickness shall be designed per Step 11 in Annex A and must be a minimum of 0.875 inch (22 mm). The BRP width must be equal to or greater than the width of the non-necked down section of the Yield-Link™ Structural Fuse stem, and must extend from the start of beam flange cope at column side to the end of the cut region on the Yield-Link™ Structural Fuse stem at the beam side. See [Figure 3](#).
4. The buckling restraint spacer plate must have the same thickness at the Yield-Link™ Structural Fuse stem. An additional configuration of Yield-Link™ with integrated spacers is available. The buckling restraint spacer plates will be attached to the non-reduced portion of the Yield-Link™ stem at the beam side for this geometry.
5. Buckling restraint bolts shall be designed per Step 11 of Annex A and must have a minimum diameter of 0.625 inch (15.9 mm).
6. Yield-Link™ Structural Fuse flange edge distance,  $L_v$  and  $L_h$  ([Figure 2c](#)), must conform to ANSI/AISC 360 Section J3.4 and Table J3.4.
7. Yield-Link™ Structural Fuse flange-to-stem connection welds must be complete joint penetration (CJP) welds that must develop the probable maximum tensile strength of the unreduced Yield-Link™ Structural Fuse stem at the column side,  $b_{col\_side}$ , and must be demand critical ([Figure 2a](#)). As an alternative, demand critical double-sided fillet welds are permitted. For Yield-Links made from W-shapes, no welding between flange and stem is required.

**4.1.4 Beam-to-Column Connection Requirements:** Standard bolt holes must be provided in the beam flanges and beam webs. Except for end-plate Yield-Link connections, where standard size holes shall be required at the column flanges; and for two-piece T-stub Yield-Link connections, oversized holes or vertical slots (maximum of 2-inch long) are permitted in the column flanges. Holes in the shear plate must be slotted to accommodate a connection rotation of at least 0.07 radians. The center/central hole in the shear plate must be a standard hole. See [Figures 1A](#) and [1B](#).

The single-shear plate must be welded to the column flange or to the end-plate, as applicable. The weld between the single-shear plate and column flange or end-plate must consist of double-sided fillet welds, partial-joint-penetration (PJP) welds or complete joint penetration (CJP) welds. CJP welds be must be demand critical.

Beam cope at yield-link must be in accordance with [Figure 3a](#).

**4.1.5 Column-beam Relationship Requirements:** Beam-to-column connections must satisfy the following requirements:

1. Panel zones must conform to the requirements of ANSI/AISC 360. The contribution of panel zone deformation to the overall story drift must be considered in accordance with [ASCE/SEI 7](#) Section 12.7.3.
2. Column-beam connection moment ratios must be limited as follows:
  - a. For SMF systems, the column-beam connection moment ratio must conform to the requirements of ANSI/AISC 341. The value of  $\sum M_{pb}^*$  must be taken equal to  $\sum(M_{pr} + M_{uv})$ , kip-in. (N-mm)

where:

- |          |   |                                                                                                                                                                                         |
|----------|---|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| $M_{pr}$ | = | Probable maximum moment capacity, computed in accordance with Eq A-9 of the <i>Design Procedure</i> , kip-in. (N-mm)                                                                    |
| $M_{uv}$ | = | Additional moment due to shear amplification from the centerline of bolts in the shear plate to the centerline of the column. $M_{uv}$ is computed as $V_u(a + d_c/2)$ , kip-in. (N-mm) |
| $V_u$    | = | Shear at the shear plate connection, computed in accordance with Step 9 of the <i>Design Procedure</i> , kips (N)                                                                       |
| $a$      | = | The distance from the centerline of bolts in shear plate to the face of the column as shown in <a href="#">Figure 3c</a> , in. (mm)                                                     |
| $d_c$    | = | The depth of the column, in. (mm).                                                                                                                                                      |

- b. For IMF systems, the column-beam moment ratio must conform to the requirements of the ANSI/AISC 341.

**4.1.6 Lateral Bracing Requirements:**

**4.1.6.1 Lateral Bracing of Beams and Joints:** There are no requirements for stability bracing of beams or joints beyond those in ANSI/AISC 360.

**Lateral Bracing of Columns:** Bracing must be provided in accordance with ANSI/AISC 341.

**Exception:** When columns are designed in accordance with the *Design Procedure* contained in this evaluation report and maximum nominal flexural strength of the column,  $M_n$ , outside the panel zone, is limited such that  $M_n \leq F_y S_x$ , where  $F_y$  is the specified minimum yield stress of the column, and  $S_x$  is the elastic section modulus of the column, bracing may be provided only at the level of the top flange of the beam.

**4.1.7 Continuity/Stiffener Plate Requirements:**

1. The need for continuity plates must be determined in accordance with the structural Design Procedure contained in this evaluation report.
2. Where required, design of continuity plates must be in accordance with ANSI/AISC 360.
3. Continuity plates may be welded to the column flange and column web with fillet welds.

**4.1.8 Bolting Requirements:**

1. The Yield-Link™ Structural Fuse stem-to-beam flange bolts must be fully pre-tensioned ASTM A325 or A490 structural bolts, or F1852 or F2280 twist-off type structural bolt assemblies complying with AISC 348. Faying surface preparation between link stem and beam flange is not required, but faying surfaces must not be painted.

2. The following connections must be made with either A325 bolts installed either as snug-tight or pre-tensioned, or A490 bolts installed as pre-tensioned, except as noted:
  - a. T-stub Yield-Link flange-to-column flange bolts or end-plate Yield-Link to column flange bolts
  - b. Buckling restraint plate bolts, which must be A325 bolts installed as snug-tight
  - c. Shear plate bolts

**4.1.9 Protected Zone:** The protected zone must consist of the Yield-Link™ Structural Fuse, the shear plate, and the portions of the beam in contact with the Yield-Link™ Structural Fuse and shear plate ([Figure 1](#) illustrates this region).

**4.1.10 Shims:** The use of finger shims at the Yield-Link™ Structural Fuse flange-to-column flange is permitted, subjected to compliance with AISC 348 and AISC 360.

**4.1.11 Column Splices:** Column splices must comply with ANSI/AISC 341.

**4.1.12 Connections Not Part of the Seismic Force Resisting System:** Beam-to-column connections, which are not part of the Seismic Force-Resisting system, but are in the same line of resistance as the Simpson Strong-Frame® steel moment frames, must be designed as simple or non-moment connections to comply with the deformation compatibility requirements of the applicable code provisions, for all applicable load combinations including drag loads, in order to minimize moment transfer and to accommodate the deformation resulting from displacement due to the design story drift calculated in accordance with the applicable building code.

## 4.2 Installation:

**4.2.1 Frame:** Strong Frame® special steel moment frames must be installed in accordance with the manufacturer's installation instructions, the applicable code, this report, and the approved construction documents prepared by a registered design professional, which must consider the effect of stiffness and strength of the supports on the structural performance of the overall structure, including lateral drift of the overall structure. Bolts connecting beams to the columns must be tightened in accordance with the manufacturer's installation instructions and this evaluation report. No field welding is required for the installation of the Strong-Frame® steel moment frames.

**4.2.2 Base Plate Grout:** Non-shrink grout complying with [ASTM C1107](#), with a minimum compressive strength of 5,000 psi (34.4 MPa) must be placed below the column base plates after the frame members are plumb and level, and all bolts are tightened. The grout pad thickness must be specified by the registered design professional and must comply with the manufacturer's installation instructions. The registered design professional may specify installation of base plates directly on concrete without grout, provided they are set level, to the correct elevation, and with full bearing.

## 4.3 Special Inspections:

Special inspections, testing and structural observations are required in accordance with [Chapter 17](#) of the IBC; Chapter N of AISC 360-16 and Chapter J of AISC 341-16 for the 2021 and 2018 IBC or Chapter N of AISC 360-10 and Chapter J of AISC 341-10 for the 2015 and 2012 IBC (or Section 18 and Appendix Q of AISC 341-05 for the 2009 IBC); applicable portions of [AISC 303-16](#) and Clause 7 of AWS D1.8-2016 for the 2021 and 2018 IBC or applicable portions of [AISC 303-10](#) and Clause 7 of AWS D1.8-2009 for the 2015 and 2012 IBC (or applicable portions of [AISC 303-05](#) and Clause 7 of AWS D1.8-2005 for the 2009 IBC) and must be specified by a registered design professional, unless the structure qualifies under the exceptions in Section [1704.2](#) of the 2021, 2018, 2015 and 2012 IBC, or in Section [1704.1](#) of the 2009 IBC, and subjected to approval of the code official. When special inspections are required, the inspections must be included in the statement of special inspections prepared by the registered design professional for submittal to the code official.

Welding is performed on the premises of a fabricator registered and approved in accordance with the requirements of 2021, 2018, 2015 and 2012 IBC Section [1704.2.5](#), and 2009 IBC Section [1704.2.2](#), for fabricator approval. Special inspection of welding required by 2021 IBC Section [1705.13](#), 2018 and 2015 IBC Section [1705.12](#), 2012 IBC Section [1705.11](#) or 2009 IBC Section [1707](#) is completed during the manufacturing process, as described in the manufacturer's quality documentation.

## 5.0 CONDITIONS OF USE:

The Simpson Strong-Tie Strong Frame® Steel Special Moment Frame Connection described in this report complies with, or is a suitable alternative to what is specified in, those codes listed in Section [1.0](#) of this report, subject to the following conditions:

- 5.1 The Simpson Strong-Tie Strong Frame® Steel Special Moment Connection design, including structural notes and details, must be in accordance with this report and the applicable code, and must be prepared by a registered design professional and subjected to approval of the code official.
- 5.2 The seismic force-resisting systems (SMFs and IMFs) utilizing the Simpson Strong-Tie Strong Frame® Steel Special Moment Connection recognized in this evaluation report must be designed by a registered design professional in accordance with the applicable code and this evaluation report, and must be subjected to approval of the code official.
- 5.3 Use of Simpson Strong-Tie Strong Frame® Steel Special Moment Connection in SMFs and IMFs with orthogonally loaded columns, as described in AISC 341-16 Sections D1.4a, E3.4a and K2.4a Must be in accordance with the requirements of AISC 341.
- 5.4 Structural design drawings and specifications must comply with Section A4 of AISC 341-16 under the 2021 and 2018 IBC or Section A4 of AISC 341-10 under the 2015 and 2012 IBC (or Section 5 of ANSI/AISC 341-05 under the 2009 IBC).
- 5.5 Installations must be in accordance with Section 4.2 of this report and the approved construction documents, as prepared by a registered design professional and approved by the code official.
- 5.6 Special inspections must be in accordance with Section 4.3 of this report and the approved construction documents.
- 5.7 The Simpson Strong-Tie Strong Frame® Steel Special Moment Frame Connection is manufactured under a quality control program with inspections by ICC-ES.

## 6.0 EVIDENCE SUBMITTED

Data in accordance with the [ICC-ES Acceptance Criteria for Steel Moment Frame Connection Systems \(AC129\)](#), dated May 2018, editorially revised February 2021.

## 7.0 IDENTIFICATION

- 7.1 The Simpson Strong-Tie Strong Frame® Steel Special Moment Frame Connection last four digits of mill certification heat number, six digit date of manufacture and Yield-Link Part ID are marked on the link-stem of each link. A label, including the ICC-ES evaluation report number (ESR-2802), must be applied near each moment connection. A Simpson Strong-Tie Strong Frame® Special Moment Frame patent label, provided by Simpson Strong-Tie Company Inc., must be applied adjacent to each moment connection.

On each sheet of structural drawing/shop detail drawing that contains technical information showing the Simpson Strong-Tie Strong Frame® Connections, the following notice of intellectual property must be affixed before release for intended use: Simpson Strong-Tie Strong Frame® Connections and Yield-Link™ Structural Fuse are protected under one or more of the following patents and applications: US patent no. 8,001,734 B2, US patent no. 8,375,652 B2, US patent publication no. 2015/0159362 and US patent publication no. 2017/0138043, and must be supplied or licensed through Simpson Strong-Tie Company Inc.

- 7.2 The report holder's contact information is the following:

**SIMPSON STRONG-TIE COMPANY INC.**  
**5956 WEST LAS POSITAS BOULEVARD**  
**PLEASANTON, CALIFORNIA 94588**  
**(800) 925-5099**  
[www.strongtie.com](http://www.strongtie.com)

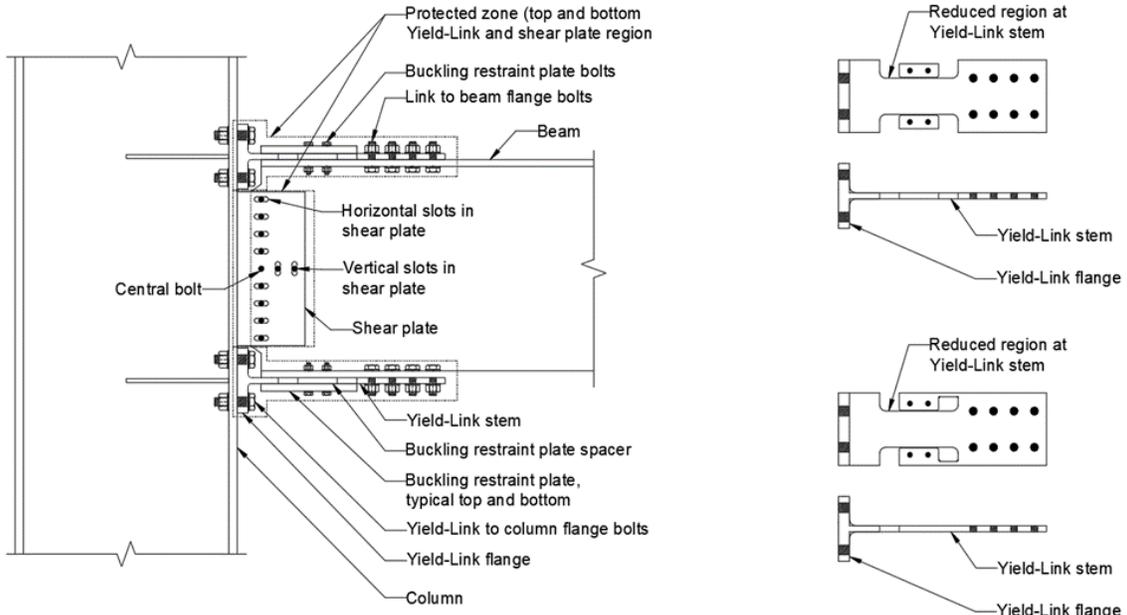


FIGURE 1A—TWO-PIECE T-STUB YIELD-LINK

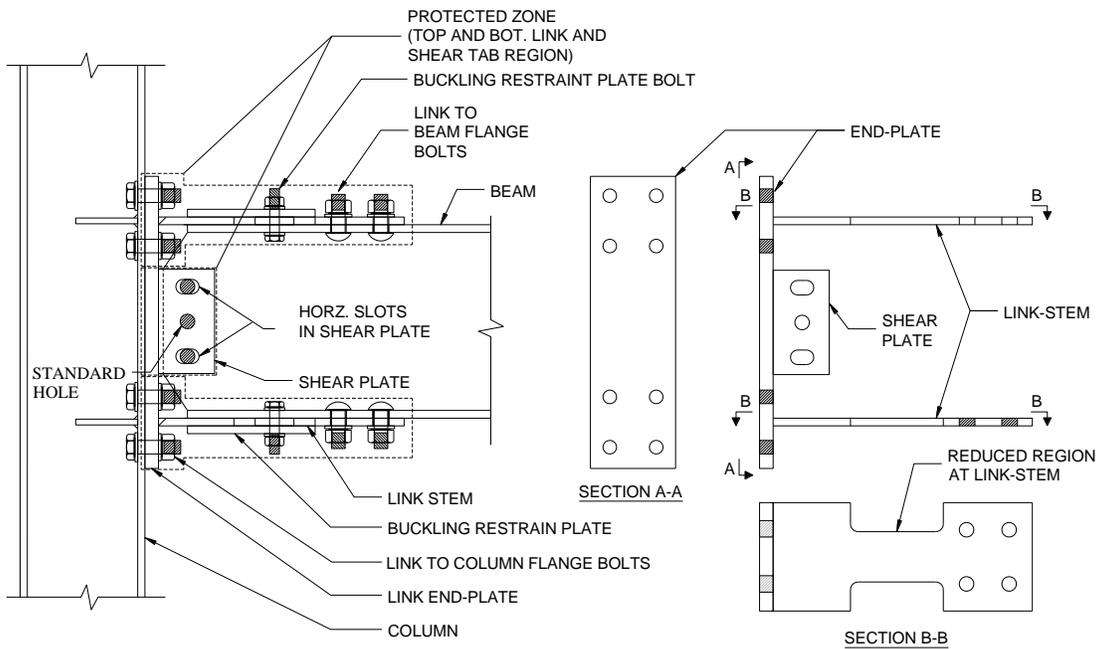
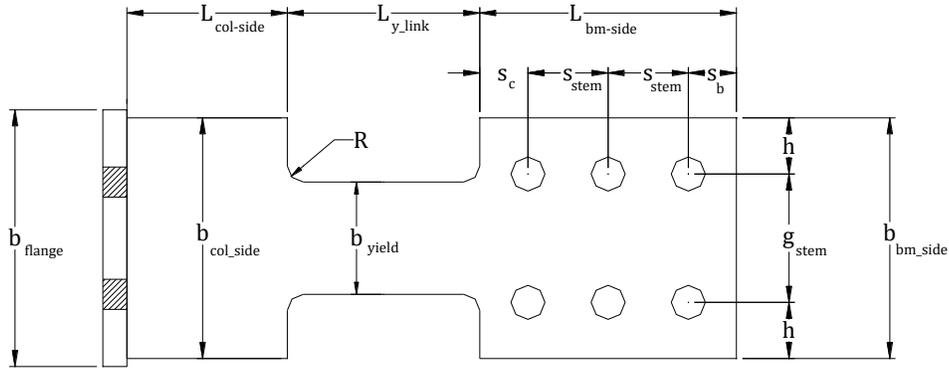
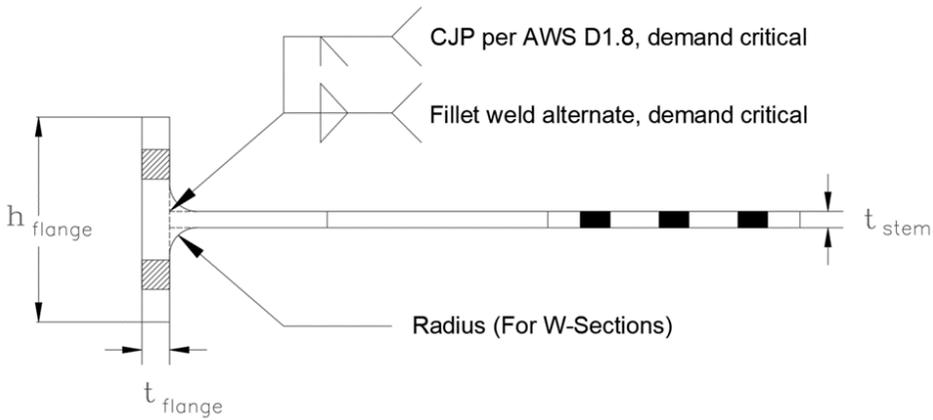


FIGURE 1B—END-PLATE YIELD-LINK

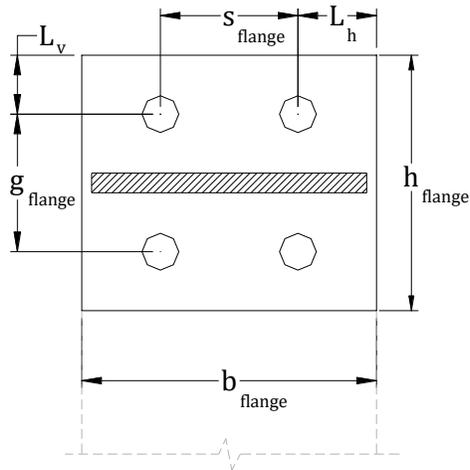
FIGURE 1—SIMPSON STRONG TIE STRONG FRAME® STEEL SPECIAL MOMENT CONNECTION



a) Yield-Link™ Structural Fuse Plan View

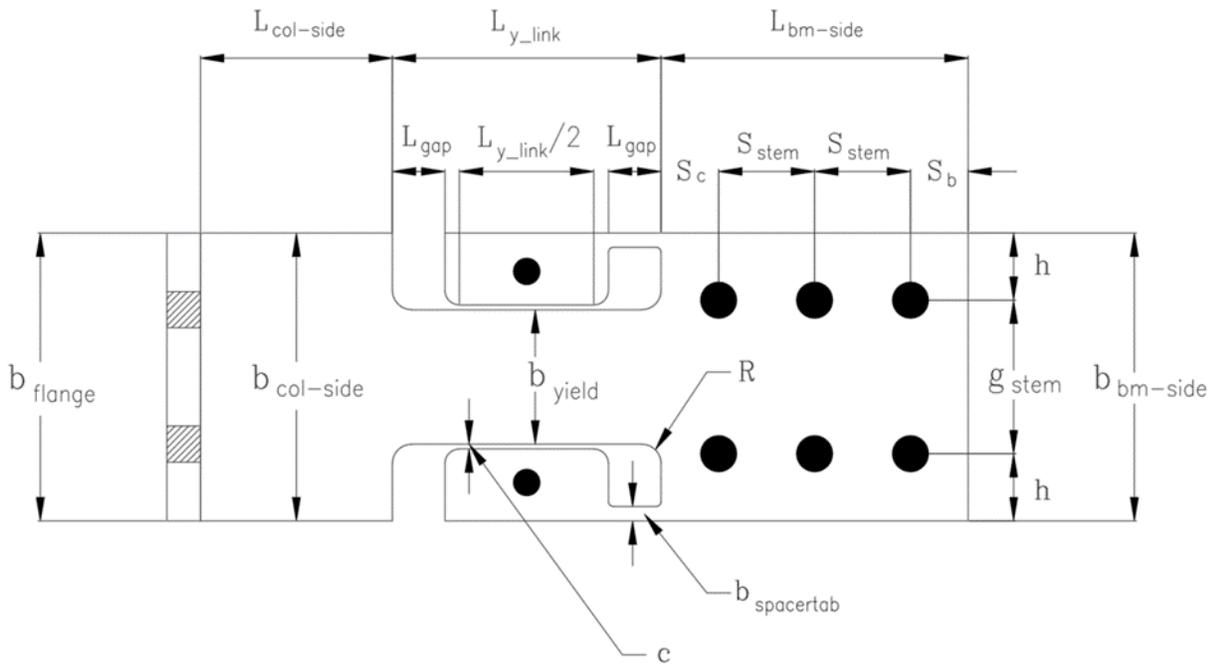


b) Yield-Link™ Structural Fuse Elevation View

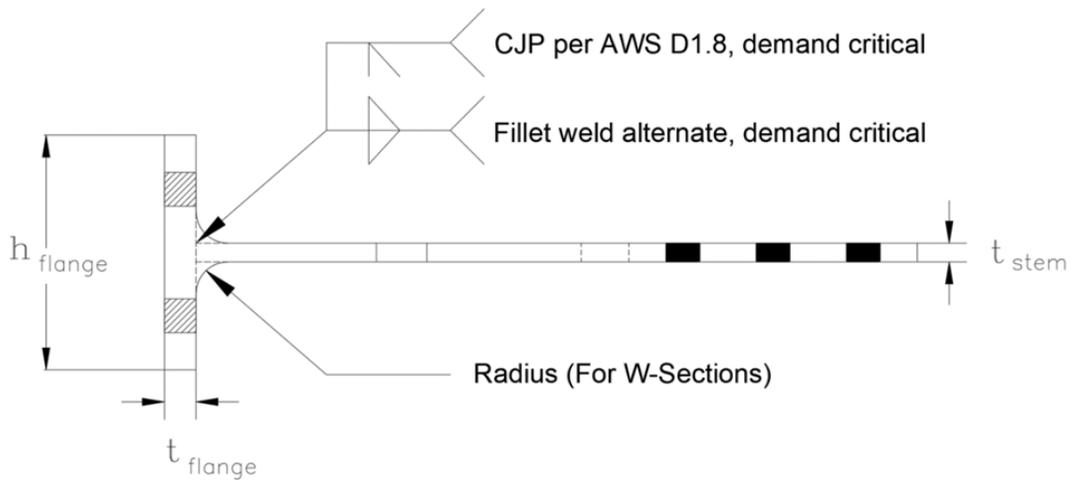


c) Link Flange View

FIGURE 2—LINK GEOMETRIES

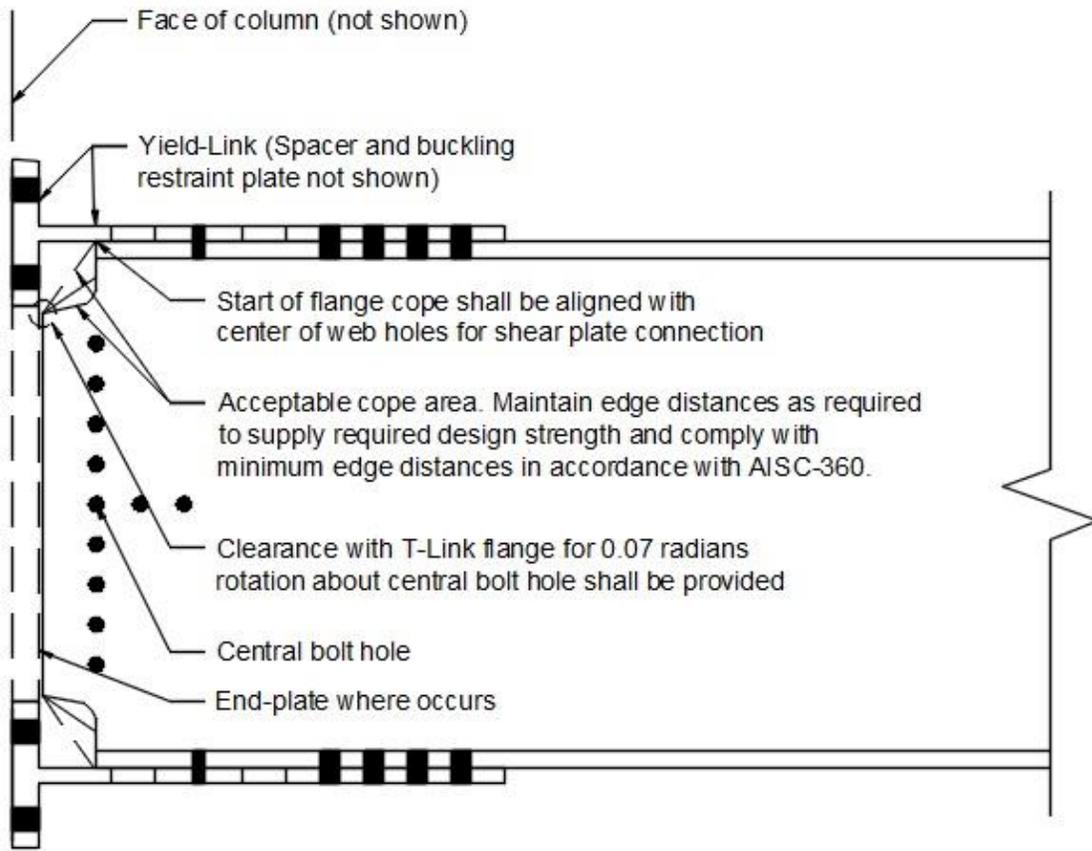


d) Yield-Link™ (with Integrated Spacers) Structural Fuse Plan View

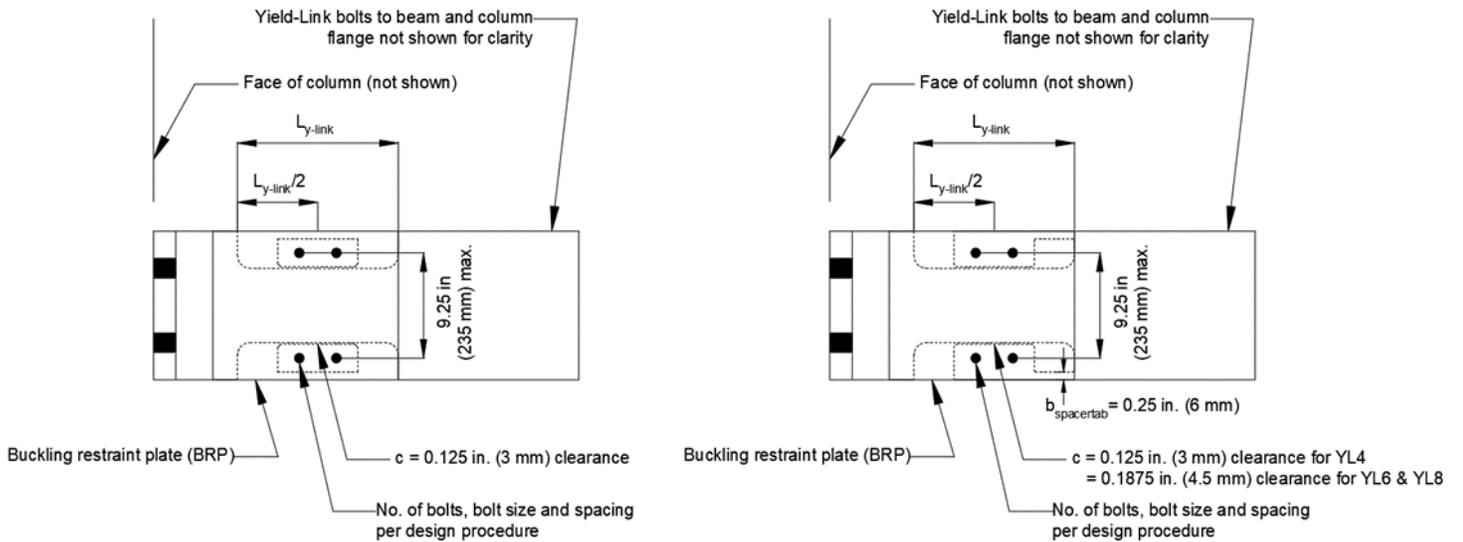


e) Yield-Link™ (with Integrated Spacers) Structural Fuse Elevation View

FIGURE 2—LINK GEOMETRIES (CONTINUED)

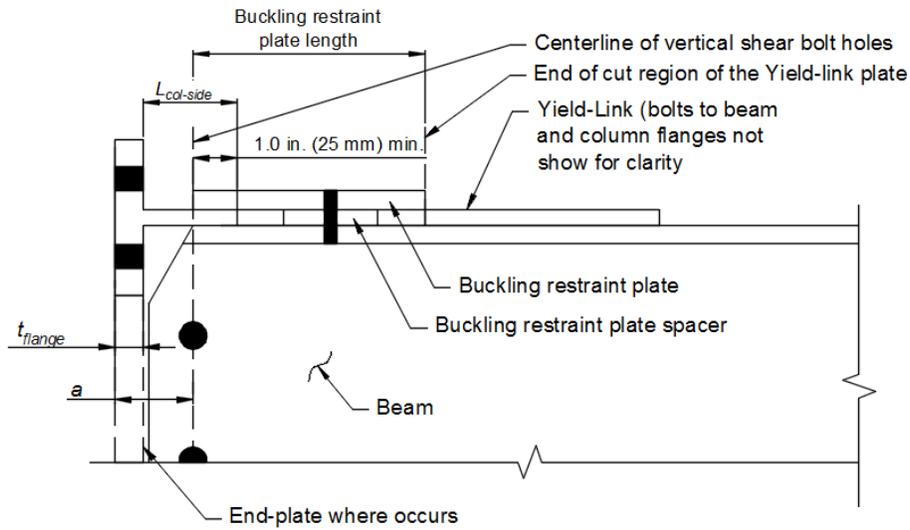


a) Beam Coping

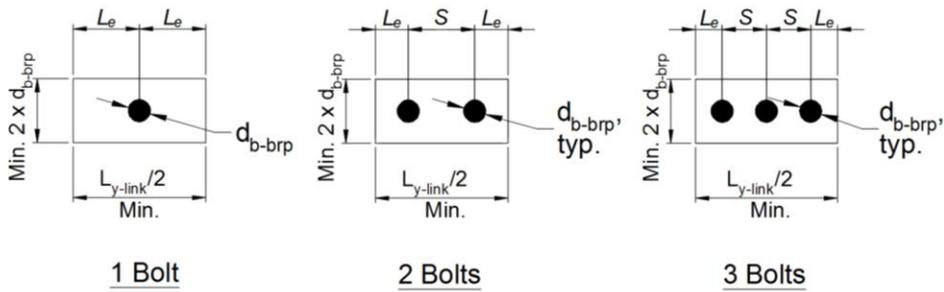


b) Buckling Restraint Spacer Plate Placement

FIGURE 3—CONNECTION DETAILING



c) Buckling Restraint Plate and Yield-Link™ Structural Fuse  $L_{col\_side}$  Limitations



d) Buckling Restraint Spacer Plate Dimensions

FIGURE 3—CONNECTION DETAILING (CONTINUED)

## Annex A: Design Procedure

**General:** Unless otherwise indicated, Step 1 through Step 19 of the design procedure are applicable to both the two piece T-stub Yield-Link and end-plate Yield-Link connection design.

- Step 1.** Choose trial values for beams and columns subject to the prequalification limits of Section 4.0, assuming fully restrained beam-to-column connections and all load combinations specified by the applicable building code. Estimate design story drift of frames with partially restrained connections for compliance with the applicable limits specified by the applicable building code as 1.2 times the calculated drift value assuming fully restrained connections.
- Step 2.** Check beam strength and deflection assuming the beam is simply supported between shear plate connections. Check beam strength using the applicable vertical load combinations of the applicable building code. Check that beam deflection under service loads is less than  $L_h/360$ , where  $L_h$  is the beam length between shear plate bolts at each end of the beam.

**User Note:** The deflection check serves to estimate beam stiffness needed to limit member end rotations. Other values may be acceptable.

- Step 3.** Estimate the required link yield strength from analysis described in Step 1.

$$P'_{y-link} = M_u / (\phi_b \times d) \quad (\text{EQ A-1})$$

$$A'_{y-link} = P'_{y-link} / F_{y-link} \quad (\text{EQ A-2})$$

Where

$M_u$  = the moment demand from elastic analysis, assuming fully restrained connections, kip-in. (N-m)

$d$  = the beam depth, in. (mm)

$\phi_b$  = 0.9

$P'_{y-link}$  = estimated required Yield-Link yield force, kips (N)

$A'_{y-link}$  = estimated required Yield-Link yield area, in.<sup>2</sup> (mm<sup>2</sup>)

$F_{y-link}$  = specified minimum yield stress of the Yield-Link stem material, ksi (MPa)

- Step 4.** Determine the non-reduced width and length of the Yield-Link stem at column side (see [Figure 2a](#)):

Step 4.1: Determine non-reduced Yield-Link stem widths:  $b_{col-side}$  and  $b_{bm-side}$ . As an initial try, let  $b_{col-side}$  and  $b_{bm-side}$  equal to the lesser of beam flange width and column flange width.

Step 4.2: Non-reduced Yield-Link stem length at column side,  $L_{col-side}$ , must have a maximum length equal to 5 inches (127 mm), and a minimum length equal to  $a - t_{flange} + 1$  inch ( $a - t_{flange} + 25$  mm). See [Figure 3c](#).

- Step 5.** Determine the reduced width of the yield section of the Yield-Link stem,  $b_{yield}$ , where thickness of link stem,  $t_{stem}$ , must be 0.5 inch (12.7 mm) for end-plate Yield Link, and must be 0.5 inch (12.7 mm) minimum and 1.0 inch (25.4 mm) maximum for two-piece T-stub Yield Link.

$$b_{yield, req'd} \geq A'_{y-link} / t_{stem} \quad (\text{EQ A-3})$$

But value of  $b_{yield, req'd}$  shall not exceed the least of 0.5  $b_{col-side}$ , 0.5  $b_{bm-side}$ , and 3.5 inches (89 mm) for the end-plate Yield Link, and 6 inches (152 mm) for the two-piece T-stub Yield Link.

- Step 6.** Determine minimum link stem yielding length such that link axial strain in the straight portion of the yield link is less than or equal to 0.085 in./in. at 0.05 radians of connection rotation.

$$L_{y-link} = \frac{0.05}{0.085} \left( \frac{d + t_{stem}}{2} \right) + 2R \quad (\text{EQ A-4})$$

Where R, the radius between the reduced width and the non-reduced width at the beam and column sides, is taken as the thickness of the link stem,  $t_{stem}$ .

Determine the length of the gap ( $L_{gap}$ ) between the spacer plate and non-reduced portion of the Yield-Link (with Integrated Spacers) stem, which is based on 0.07 radians of connection rotation, to avoid the spacer plates bearing into the non-reduced portion of the link stem at column side during compression.

$$L_{gap} \geq 0.07 \times (d / 2 + t_{stem}) \quad (\text{EQ A-4.1})$$

Where

$d$  = Beam depth, in. (mm)

$t_{stem}$  = Thickness of Yield-Link stem, in. (mm)

- Step 7.** Compute the expected yield and probable maximum tensile strength of the Yield-Link.

$$P_{ye-link} = A_{y-link} \times R_y \times F_{y-link} \quad (\text{EQ A-5})$$

$$P_{r-link} = A_{y-link} \times R_t \times F_{u-link} \quad (\text{EQ A-6})$$

Where

$A_{y-link}$	=	area of the reduced link area ( $b_{yield} \times t_{stem}$ ), in. <sup>2</sup> (mm <sup>2</sup> )
$P_{ye-link}$	=	expected yield strength of Yield-Link, kips (N)
$P_{r-link}$	=	probable maximum tensile strength of Yield-Link, kips (N)
$R_y$	=	ratio of expected yield stress to specified minimum yield stress, $F_y$ ; taken as 1.1 for Yield-Link stem material.
$R_t$	=	ratio of expected tensile strength to the specified minimum tensile strength $F_u$ ; as related to overstrength in material yield stress, $R_y$ ; taken as 1.2 for Yield-Link stem material.
$F_{u-link}$	=	specified minimum tensile strength of Yield-Link stem material, ksi (MPa)

**Step 8.** Determine the Yield Link stem non-reduced width,  $b_{bm-side}$ , and length,  $L_{bm-side}$ , at beam side of the Yield-Link using  $P_{r-link}$  from Step 7:

Step 8.1: Design bolts for shear transfer between Yield-Link™ Structural Fuse stem and beam flange (i.e. to resist  $P_{r-link}$  determined per Step 7) per ANSI/AISC 360 (AISC Specification) and determine bolt diameter,  $d_{b-stem}$ .

Step 8.2: Determine the non-reduced width of the Yield-Link stem,  $b_{bm-side}$ . See Step 4.1.

Step 8.3: Determine the non-reduced length of the Yield-Link stem at beam side,  $L_{bm-side}$ ,

$$L_{bm-side} = S_c + [(n_{rows} - 1) \times S_{stem}] + S_b \quad (\text{EQ A-7})$$

Where

$S_c$	=	distance from reduced section of Yield-Link to center of first row of bolt holes = minimum of $1.5d_{b-stem}$ , in. (mm)
$S_b$	=	distance from center of last row of bolt holes to beam-side end of Yield-Link stem, from Table J3.4 of ANSI/AISC 360, in. (mm)
$S_{stem}$	=	spacing between rows of bolt holes for Yield-Link stem to beam flange connection, ≥ <b>minimum of</b> $2 \frac{2}{3}d_{b-stem}$ , in. (mm)
$n_{rows}$	=	number of rows of bolt holes determined in Step 8.1.

Step 8.4: To resist  $P_{r-link}$  determined per Step 7, check link stem at beam side for tensile yielding, tensile rupture, block shear rupture, and bolt bearing (where deformation at the hole is a design consideration) in accordance with ANSI/AISC 360; and check the beam flange for bolt bearing (where deformation at the bolt hole is a design consideration) and block shear rupture in accordance with ANSI/AISC 360.

**Step 9.** Determine the required shear strength,  $V_u$ , of the beam and beam web-to-column flange (or beam web-to-end plate) connections:

$$V_u = \frac{2M_{pr}}{L_h} + V_{gravity} \quad (\text{EQ A-8})$$

Where

$$M_{pr} = P_{r-link} \times (d + t_{stem}) \quad (\text{EQ A-9})$$

$V_u$	=	required shear strength of beam, beam web-to-column flange, and beam web-to-end plate connections, kips (N)
$L_h$	=	horizontal distance between centerlines of bolts in shear plate at each end of the beam, in. (mm)
$V_{gravity}$	=	shear force in the beam, kips (N), resulting from $1.2D + f_1L + 0.2S$ (where $f_1$ is the load factor determined by the applicable building code for live loads, but not less than 0.5). The shear force at the shear plate connection must be determined from a free body diagram of the portion of the beam between the shear plate connections.

**User Note:** The load combination of  $1.2D + f_1L + 0.2S$  is in conformance with ASCE/SEI 7. When using the International Building Code, a factor of 0.7 must be used in lieu of the factor of 0.2 for S (snow) when the roof configuration is such that it does not shed snow off the structure.

**Step 10.** Design Yield-Link flange-to-column flange connection to resist  $P_{r-link}$  from Step 7, and  $M_{pr}$  and  $V_u$  from Step 9:

Step 10.1: Design bolts for tension transfer,  $r_t$ , between the Yield-Link flange (or end-plate) and the column flange per ANSI/AISC 360 and determine bolt diameter,  $d_{b-flange}$ . The required tension force per bolt in the Yield-Link flange (or end-plate) to column flange connection,  $r_t$ , kips (N) is:

For T-stub Yield-Links:

$$r_t = \frac{P_{r-link}}{4} \quad (\text{EQ A-10})$$

For end-plate Yield-Links:

$$r_t = \frac{M_{pr}}{2(h_o+h_1)} + \frac{V_u \times a}{2h_1} \quad (\text{EQ A-11})$$

Where:

$$d_{b-flange \text{ req'd}} = \sqrt{\frac{4 \times r_t}{\pi \times \phi_n \times F_{nt}}} \quad (\text{EQ A-12})$$

$M_{pr}$  = Probable maximum moment capacity, kip-in (kN-m), per Step 9

$V_u$  = Shear force at the end of the beam, kips (N), per Step 9

$a$  = Horizontal distance from centerline of bolt holes in shear plate to face of the column, in. (mm).

See [Figure 3c](#).

$h_o$  = End-Plate geometry, as shown in [Table A3](#), in. (mm)

$h_1$  = End-Plate geometry, as shown in [Table A3](#), in. (mm)

$\pi$  = 3.14159

$\phi_n$  = 0.9

$F_{nt}$  = nominal tensile strength of bolt from the AISC *Specification*, ksi (MPa)

Step 10.1a: For end plate Yield-Link connections, check bolt shear rupture strength of the connection provided by bolts at the compression flange only:

$$V_u \leq \phi_n \times n_b \times F_{nv} \times A_b \quad (\text{EQ-A13})$$

Where:

$V_u$  = Shear force at the end of the beam, kips (N), per Step 9

$n_b$  = 4 for the 4 compression bolts

$F_{nv}$  = Nominal shear strength of bolt from the AISC *Specification*, ksi (MPa)

$A_b$  = Nominal gross area of each bolt, in.<sup>2</sup> (mm<sup>2</sup>)

Step 10.2: Determine Yield-link flange/end plate thickness required to prevent prying action.

$$t_{flange} = \sqrt{\frac{4 \times r_t \times b'}{p \times \phi_d \times F_u}} \quad (\text{EQ A-14})$$

$$b' = (b - d_{b-flange} / 2) \quad (\text{EQ A-15})$$

Where

$b$  = vertical distance between centerline of top row bolts (or bottom row of bolts) in Yield-Link to the corresponding face of Yield-Link stem, in. (mm)

$p$  = minimum of  $b_{flange}/2$  and  $s_{flange}$ , in. (mm), as shown in [Figure 2c](#).

$d_{b-flange}$  = diameter of bolt connecting Yield-Link flange/end-plate and column flange, in. (mm)

$r_t$  = Required tension force per bolt in the Yield-Link flange (or end-plate) to column flange connection,  $r_t$ , kips (N), per Step 10.1

$\phi_d$  = 1.0

Width of end-plate shall be no less than  $b_{col-side}$  determined from Step 4 above.

Step 10.3: Check thickness of the Yield-Link flange,  $t_{flange}$ , for shear yielding and shear rupture per ANSI/AISC 360 for resisting bolt tension forces determined per Step 10.1.

Step 10.3a: For end-plate Yield-Link connections, check shear yielding and shear rupture of the extended portion of the end-plate:

$$F_{pf}/2 \leq \phi_d(0.6)F_{yp}b_p t_p \quad (\text{EQ A-16})$$

$$F_{pf}/2 \leq \phi_n(0.6)F_{up}A_n \quad (\text{EQ A-17})$$

Where:

$$\begin{aligned}
 F_{pf} &= P_{r-link} \text{ from Equation A-6 in Step 7, kips (N)} \\
 b_p &= \text{Width of the End-plate, in. (mm)} \\
 t_p &= \text{Thickness of the End-plate provided, in. (mm)} \\
 F_{yp} &= \text{Specified minimum yield stress of End-plate, ksi (MPa)} \\
 F_{up} &= \text{Specified minimum tensile stress of End-plate, ksi (MPa)} \\
 A_n &= t_p [b_p - 2(d_b + 1/8)], \text{ in.}^2 \\
 &= t_p [b_p - 2(d_b + 3)], \text{ mm}^2 \\
 d_b &= \text{bolt diameter for the End-plate to column flange connection, in. (mm)}
 \end{aligned}$$

Step 10.3b: For end-plate Yield-Link connections, check bolt bearing/tear-out failure of the end-plate and column flange:

$$V_u \leq \phi_n \times (n_i \times r_{ni} + n_o \times r_{no}) \quad (\text{EQ-A-18})$$

Where:

$$\begin{aligned}
 V_u &= \text{Shear force at the end of the beam, kips (N), per Step 9} \\
 n_i &= 2, \text{ the number of inner bolts} \\
 n_o &= 2, \text{ the number of outer bolts} \\
 r_{ni} &= 1.2 \times L_c \times t \times F_u < 2.4 \times d_b \times t \times F_u, \text{ kips (N)} \\
 r_{no} &= 1.2 \times L_c \times t \times F_u < 2.4 \times d_b \times t \times F_u, \text{ kips (N)} \\
 L_c &= \text{Clear distance, in the direction of force, between the edge of the hole and the edge of the adjacent hole or edge of connected steel elements, in. (mm)} \\
 d_b &= \text{Diameter of the bolt, in. (mm)} \\
 t &= \text{End-plate or column flange thickness, in. (mm)} \\
 F_u &= \text{Specified minimum tension strength of End-plate or column flange material, ksi (MPa)}
 \end{aligned}$$

Step 10.4 Design the stem to flange weld of the T-stub Yield Link and end-plate Yield Link as either a CJP weld or a double-sided fillet weld that will develop the probable maximum tensile strength of non-reduced stem widths  $b_{col-side}$  at the column side,  $P_{r-weld}$ , as:

$$P_{r-weld} = b_{col-side} \times t_{stem} \times R_t \times F_{u-link} \quad (\text{EQ A-19})$$

### Step 11. Design the Buckling Restraint Assembly

Step 11.1 Determine the minimum thickness of the Buckling Restraint Plate (BRP) to prevent yielding during compression of the link stem, minimum BRP thickness shall not be less than 0.875 inch (22 mm):

$$t_{brp\_min} = 0.51 \sqrt{\frac{L_{cant} \times P_{r-link}}{F_{y\_BRP} R_{y\_BRP} b_n}} \quad (\text{EQ A-20})$$

Where:

$L_{cant}$  = Lever arm from start of reduced region to edge of first spacer plate bolt hole, plus plate stretch length due to a 0.05 radian of rotation, in (mm)

$b_n$  = Net width of the BRP, in (mm)

$F_{y\_BRP}$  = Specified minimum yield strength of BRP material, ksi (MPa)

$R_{y\_BRP}$  = Ratio of the expected yield stress to specified minimum yield stress,  $F_{y\_BRP}$ , taken as 1.1 for BRP material

Step 11.2 Determine beam minimum flange thickness to prevent yielding and BRP bolt induced prying, minimum flange thickness shall not be less than 0.4 in (10 mm):

$$t_{bf\_min} = \sqrt{\frac{4b'T}{\phi_d p_e F_{u,bm}}} \quad (\text{EQ A-21})$$

Where:

$b'$  = Distance from the BRP bolt centerline to the face of beam web, in (mm)

$p_e$  = Effective (tributary) length per bolt from the yield line pattern, in (mm)

$T$  = Tension force for each BRP bolt, ( $T_{ux} / N_{BRP\_bolts}$ ), kips (kN)

$N_{BRP\_bolts}$  = Total number of BRP bolts per one Yield-Link.

$T_{ux}$  = Total inelastic vertical thrust force on the beam flange, kips (kN)

$$T_{ux} = N_{design} \times Q_i \quad (\text{EQ A-22})$$

$$N = \left\lfloor \frac{L_{y-link}}{L_o} \right\rfloor \quad (\text{EQ A-23})$$

$N$  = Number of buckling wave crests

$N_{design}$  = Number of contact points between reduced region of link-stem and BRP or beam flange,  $(N/2)$ , rounded to the nearest integer

$L_o$  = The effective buckling wave length, in (mm):

$$L_o = \sqrt{\frac{\pi^2 E_t I_y}{P_{r-link}} \left[ 1 + \left( \frac{b_{yield}}{2g} + 1.013 \right)^{-1} \right]} \quad (\text{EQ A-24})$$

Where:

$E_t$  = Yield-link tangent modulus, 193 ksi (1327 MPa)

$I_y$  = Weak axis moment of inertia of reduced link stem region, in<sup>4</sup> (mm<sup>4</sup>)

$$g = 0.25 \varepsilon_{0.04} \times t_{stem} \quad (\text{EQ-A25})$$

$$\varepsilon_{0.04} = \frac{(0.04 \times \frac{d+t_{stem}}{2})}{L_{y-link} - 2R} \quad (\text{EQ A-26})$$

$$Q_i = \frac{4gP_{r-link}}{L_o} \quad (\text{EQ A-27})$$

Step 11.3 Determine the BRP bolt size and quantity

$$V_{uy} = \frac{4cP_{r-link}}{\sqrt{\frac{\pi^2 E_t I_x}{P_{r-link}} \left[ 1 + \left( \frac{t_{stem}}{2c} + 1.013 \right)^{-1} \right]}} \quad (\text{EQ A-28})$$

Where:

$c$  = gap between spacer plate and reduced section of the Yield-Link stem, 0.125 in (3.175 mm).  $c = 0.1875$  in (4.76 mm) for Yield-Links (with Integrated Spacers) with stem thickness = 0.75 in (19 mm) or 1 in (25.4 mm).

$V_{uy}$  = Total in-plane thrust force exerted on each spacer plate, kip (kN)

$I_x$  = Strong axis moment of inertia of reduced link stem region, in<sup>4</sup> (mm<sup>4</sup>)

Design BRP bolts per AISC 360 for the following load conditions: (1) combined tension and shear interaction due to out-of-plane thrust  $T_{ux}$ , where tension  $T_{ux}$  is from Step 11.2 and shear  $V_{ux} = 0.3T_{ux}$ . and (2) shear due to in-plane thrust force  $V_{uy}$ , where  $V_{uy}$  is derived from EQ A-28 above.

**Step 12.** Check frame drift and connection moment demand by accounting for actual connection stiffness.

Step 12.1: Elastic (or strength level seismic force) panel zone deformation contributions to story drift must be included as required by ASCE/SEI 7 Section 12.7.3.

Step 12.2: Model the connection using a pair of nonlinear axial links or a nonlinear rotational spring at each connection, determined from the following properties:

$K_1$  is the elastic axial stiffness contribution due to bending stiffness of Yield-Link/end-plate flange, kip/in. (N/mm)

$$K_1 = \frac{0.75 \times 192 \times E_s \times \left( \frac{b_{col-side} \times t_{flange}^3}{12} \right)}{g_{flange}^3} \quad (\text{EQ A-29})$$

Where:  $E_s$  = Modulus of elasticity of steel, ksi (MPa)

$K_2$  is the elastic axial stiffness contribution due to non-yielding section of Yield-Link, kip/in. (N/mm)

$$K_2 = \frac{t_{stem} \times b_{col-side} \times E_s}{L_{col-side} + S_c + l_v} \quad (\text{EQ A-30})$$

Where:

$l_v$  = 0 when 4 or fewer bolts are used at the Yield Link -to-beam connection; or

$l_v$  =  $S_{stem} / 2$  when more than four bolts are used at the Yield Link -to-beam connection.

$K_3$  is the elastic axial stiffness contribution due to yielding section of Yield-Link, kip/in. (N/mm)

$$K_3 = \frac{t_{stem} \times b_{yield} \times E_s}{L_{y-link}} \quad (\text{EQ A-31})$$

$K_{eff}$  = effective elastic axial stiffness of Yield-Link, k/in. (N/mm)

$$K_{eff} = \frac{K_1 K_2 K_3}{(K_1 \times K_2 + K_2 \times K_3 + K_1 \times K_3)} \quad (\text{EQ A-32})$$

$M_{ye-link}$  = expected yield moment capacity of Yield-Link, kip-in. (N-mm)

$$M_{ye-link} = P_{ye-link} \times (d + t_{stem}) \quad (\text{EQ A-33})$$

$M_{pr-link}$  = probable maximum moment capacity of Yield-Link, equals to  $M_{pr}$  determined per Step 9, kip-in. (N-mm)

$\Delta_y$  = axial deformation in Yield-Link at Yield-Link expected yield strength, in. (mm)

$$\Delta_y = \frac{P_{ye-link}}{K_{eff}} \quad (\text{EQ A-34})$$

$\Delta_{0.04}$  = axial deformation in Yield-Link at a connection rotation of 0.04 rad, in. (mm)

$$\Delta_{0.04} = \frac{0.04 \times (d + t_{stem})}{2} \quad (\text{EQ A-35})$$

$\Delta_{0.07}$  = axial deformation in Yield-Link at a connection rotation of 0.07 rad, in. (mm)

$$\Delta_{0.07} = \frac{0.07 \times (d + t_{stem})}{2} \quad (\text{EQ A-36})$$

$\theta_y$  = connection rotation at Yield-Link expected yield strength, rad

$$\theta_y = \frac{\Delta_y}{0.5 \times (d + t_{stem})} \quad (\text{EQ A-37})$$

All other terms were previously defined or shown in [Figure A1](#). Refer to [Figure A1.a](#) for a plot of Yield-Link axial force vs. Yield-Link axial deformation, and [Figure A1.b](#) for a plot of the Yield-Link moment vs. Yield-Link rotation, which are the relationships required for the analysis and modeling of the Simpson Strong-Tie moment connection.

Step 12.3: Considering the applicable drift limit and all applicable load combinations specified by the applicable building code, without the amplified seismic load, verify that:

- (a) The connection moment demand,  $M_u$ , is less than or equal to the connection design moment capacity,  $\phi M_n$ , taking  $\phi=0.9$  and  $M_n$  as  $M_{ye-link}/R_y$ ;
- (b) The drift complies with applicable limits;

Adjust link size and/or number of connections as needed to comply.

**Step 13:** Verify beam and column sizes selected in Step 1.

Step 13.1: Beams must satisfy ANSI/AISC 360, considering:

- (a) Gravity load from all applicable load combinations.
- (b) Axial force due to seismic effects determined as the lesser of a) the maximum force that the system can deliver, including the effects of the probable maximum tensile strength of the Yield-Link; and b) the amplified seismic load.
- (c) The application of  $M_{pr}$  and  $V_u$ , determined per Step 9, at each end of the beam as required.

Step 13.2: Columns strength must satisfy ANSI/AISC 360 considering loads from all applicable load combinations in the IBC, where the seismic effects are determined from the lesser of a) the maximum force that the system can deliver, including the effects of the probable maximum tensile strength of the Yield-Link; and b) the amplified seismic loads. Column splices must satisfy Section 4.1.11. Per Section 4.1.6, if column bracing is only provided at the level of the top flange of the beam, in addition to the requirements of

ANSI/AISC 360, the maximum design flexural strength of the column outside the panel zone,  $\phi_b M_n$ , must be taken as  $\phi_b M_n \leq \phi_b F_y S_x$ ,

where

$$\phi_b = 0.9$$

$F_y$  = Specified minimum yield stress for column material, ksi (MPa)

$S_x$  = Elastic section modulus of column about the x-axis, in.<sup>3</sup> (mm<sup>3</sup>)

- Step 13.3: For built-up beams and built-up columns, the strength of web-to-flange welds must equal to or exceed the shear flow demand at the web-to-flange interface, accounting for applicable loads described in Steps 13.1 and 13.2. In addition, web-to-flange connections must have a minimum of  $3/16$  in. continuous fillet welds on one side of the web-to-flange connections.
- Step 13.4: For built-up beams, in addition to requirements in Steps 13.1 and 13.3, at the link stem-to-beam flange connection region, double-sided fillet welds shall be used for web-to-flange connection, and the minimum length of the double-sided fillet welds shall cover the distance from the end of the beam to the end of the link stem connecting to the beam flange. Minimum weld size must satisfy AISC *Specification* Table J2.4.
- Step 13.5: For built-up I-shaped columns, in addition to the requirements in Steps 13.2 and 13.3, double-sided fillet welds shall be used to connect the column web-to-column flanges at the panel zone region. The minimum length of double-sided fillet welds shall extend 6 inches beyond the top and bottom of the beam flanges. Minimum weld size shall satisfy AISC *Specification* Table J2.4.
- Step 13.6: For built-up flanged cruciform columns, the web of the tee-shaped section(s) shall be welded to the web of the continuous I-shaped section with double-sided fillet welds. Fillet welds shall be sized to develop the minimum of 1) the shear strength of the column section per AISC 360 Equation G2-1, or 2) the maximum shear that can be developed in the column when  $M_{pr\_link}$  is developed in the moment connection(s). The minimum fillet weld size shall satisfy AISC *Specification* Table J2.4.

**Step 14.** Check the column-beam relationship limitations in accordance with Section 4.1.5.

**Step 15.** Design beam web-to-column flange or beam web-to-end plate connection for the following required strengths

$V_u$  = shear demand, as determined in Step 9.  
 $P_{u-sp}$  = required axial strength of the connection, which must be taken as the lesser of the following:

- 1) Maximum axial force the system can deliver, including the effects of the probable maximum tensile strength of the Yield-Link, and
- 2) The axial force calculated using the load combinations of the applicable building code, including the amplified seismic load.

$M_{u-sp}$  = moment in the shear plate at the column face, kip-in. (N-mm)  
 =  $V_u \times a$

- Step 15.1: (a) Calculate maximum shear plate bolt shear by sizing the shear plate central bolt to resist axial load from the beam and its share of the vertical loads,  $V_{u-bolt}$ , kips (N):

$$V_{u-bolt} = \sqrt{\left(\frac{P_{u-sp}}{n_{bolt-sp-horz}}\right)^2 + \left(\frac{V_u}{n_{bolt-sp-vert}}\right)^2} \quad (\text{EQ A-38})$$

Where

$n_{bolt-sp-vert}$  = total number of bolts in the shear plate resisting gravity load in the beam.

$n_{bolt-sp-horz}$  = total number of bolts in the shear plate in line with the central bolt resisting axial loads in the beam.

- (b) Based on bolt shear determined in Step 15.1(a), select a bolt size,  $d_{b-sp}$ , for all bolts, which complies with ANSI/AISC 360.

Step 15.2: Determine shear plate geometry to accommodate a connection rotation of  $\pm 0.07$  radians

$$L_{slot\_borz} = d_{b-sp} + \frac{1}{8} \text{ in.} + 0.14 S_{vert} \left(\frac{n_{bolt-sp-vert}-1}{2}\right) \quad (\text{EQ A-39})$$

$$L_{slot\_horz} = d_{b-sp} + 3 \text{ mm} + 0.14 S_{vert} \left(\frac{n_{bolt-sp-vert}-1}{2}\right) \quad (\text{EQ A-39M})$$

$$L_{slot\_vert} = d_{b-sp} + \frac{1}{8} \text{ in.} + 0.14 S_{horz} (n_{bolt\_sp\_horz} - 1) \quad (\text{EQ A-40})$$

$$L_{slot\_vert} = d_{b-sp} + 3 \text{ mm} + 0.14 S_{horz}(n_{bolt\_sp\_horz} - 1) \quad (\text{EQ A-40M})$$

Where

- $d_{b-sp}$  = diameter of bolts in shear plate, in. (mm)
- $S_{vert}$  = vertical shear plate bolt spacing, in. (mm)
- $S_{horz}$  = horizontal shear plate bolt spacing, in. (mm)
- $L_{slot\_horz}$  = Horizontal dimension of horizontal slots in shear plate bolts (other than center for bolt), see [Figure 1A](#), in. (mm)
- $L_{slot\_vert}$  = Vertical dimension of vertical slots in shear plate bolts, see [Figure 1A](#), in. (mm)

- Step 15.3: For tension and shear load resistance, check shear plate for tension and shear yielding, tension and shear rupture, block shear for tension and shear loads, combined tension and bending yielding at the column face, and bolt bearing, where deformation at the bolt hole is a design consideration, per ANSI/AISC 360.
- Step 15.4: Size the weld at the shear plate to column face or shear plate to end plate joint to develop the plate in shear, tension and bending. For double-sided fillet welds, the minimum leg size shall be  $\frac{5}{8} t_{sp}$ , where  $t_{sp}$  is the shear plate thickness.
- Step 15.5: For tension and shear load resistance, check beam web for tension and shear yielding, tension and shear rupture, block shear for tension and shear loads, and bolt bearing, where deformation at the bolt hole is a design consideration, per ANSI/AISC 360.
- Step 15.6: Detail the beam flange and web cope such that the flange begins at a point aligned with the centerline of bolts in shear plate. Check entering and tightening clearances as appropriate. See [Figure 3a](#).

**User Note:** Checking the beam web for flexure at the cope is not required since the flange copes do not extend beyond the centerline of the beam shear plate connection.

- Step 16.** Check the column panel zone shear strength per ANSI/AISC 360. The required shear strength must be determined from the summation of link probable maximum axial strengths determined in Step 7. Doubler plates must be used as required.
- Step 17.** Check column web for concentrated force(s) of  $P_{r-link}$ , in accordance with ANSI/AISC 360.
- Step 18.** Check minimum column flange thickness for flexural yielding:

$$t_{cf\_min} = \sqrt{\frac{1.11M_{pr}}{\phi_d F_{yc} Y_c}} \quad (\text{EQ A-41})$$

Where

- $F_{yc}$  = the specified minimum yield strength of the column flange material, ksi (MPa)
- $Y_c$  = column flange yield line mechanism parameter from [Table A1](#) or [A2](#) (Note: When using [Table A2](#)  $Y_p$  substitutes for  $Y_c$ ). For connections away from column ends, [Table A1](#) must be used. For connections at or near column ends, [Table A2](#) (stiffened) must be used. An unstiffened column flange connection at the end of a column may be used when a rational analysis demonstrates that that the unstiffened column flange design moment strength, as controlled by flexural yielding of the column flange, meets or exceeds the connection moment demand,  $M_{pr-link}$ .

If stiffeners are required for column flange flexural yielding, column flange flexural strength is:

$$\phi_d M_{cf} = \phi_d F_{yc} Y_c t_{cf}^2 \quad (\text{EQ A-42})$$

where  $Y_c$  is the unstiffened column yield line mechanism parameter from [Table A1](#). Therefore, the equivalent column flange design force is

$$\phi_d R_n = \frac{\phi_d M_{cf}}{(d+t_{stem})} \quad (\text{EQ A-43})$$

- Step 19.** If a continuity or stiffener plate is required for any of the column limit states in Steps 17 and 18, the required strength  $F_{su}$ , is:

$$F_{su} = P_{r-link} - \text{minimum } (\phi R_n) \quad (\text{EQ A-44})$$

Where

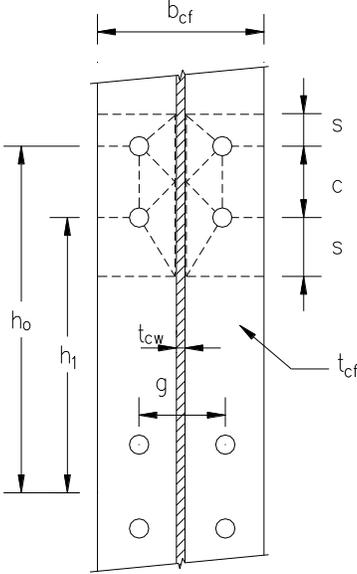
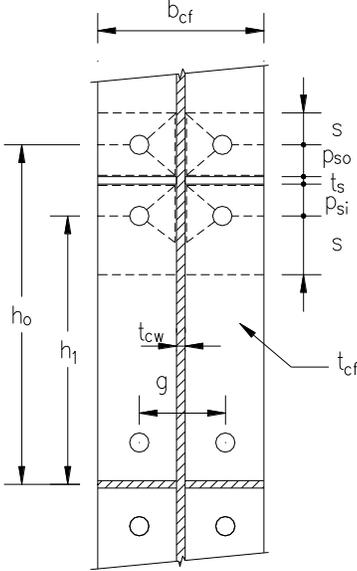
- $\phi R_n$  = the design strengths from Steps 17 and 18, kips, (N)

- Step 19.1: Design continuity or stiffener plate per ANSI/AISC 360.
- Step 19.2: Design stiffener-to-column web weld and stiffener to-column flange weld per ANSI/AISC 360.

The continuity or stiffener plate must conform to Section J10.8 of ANSI/AISC 360, with a minimum thickness of  $\frac{1}{4}$  in. (6 mm). For cruciform columns, the continuity or stiffener plate must be a minimum thickness of  $\frac{1}{2}$  of the thickness of the Yield-Link stem thickness and must be in each quadrant of the column. Fillet welds, partial joint penetration (PJP)

welds, or complete joint penetration (CJP) welds shall be used for the connection between the continuity plate to the cruciform column flange and web sections.

TABLE A1—SUMMARY OF FOUR-BOLT EXTENDED COLUMN FLANGE YIELD LINE MECHANISM PARAMETER, WHERE CONNECTION IS AWAY FROM COLUMN END

Unstiffened Column Flange Geometry And Yield Line Pattern	Stiffened Column Flange Geometry And Yield Line Pattern
	
<p><b>Unstiffened Column Flange</b></p>	$Y_c = \frac{b_{cf}}{2} \left[ h_1 \left( \frac{1}{s} \right) + h_0 \left( \frac{1}{s} \right) \right] + \frac{2}{g} \left[ h_1 \left( s + \frac{3c}{4} \right) + h_0 \left( s + \frac{c}{4} \right) + \frac{c^2}{2} \right] + \frac{g}{2}$ $s = \frac{1}{2} \sqrt{b_{cf} g}$
<p><b>Stiffened Column Flange</b></p>	$Y_c = \frac{b_{cf}}{2} \left[ h_1 \left( \frac{1}{s} + \frac{1}{p_{si}} \right) + h_0 \left( \frac{1}{s} + \frac{1}{p_{so}} \right) \right] + \frac{2}{g} \left[ h_1 (s + p_{si}) + h_0 (s + p_{so}) \right]$ $s = \frac{1}{2} \sqrt{b_{cf} g} \quad \text{Note: If } p_{si} > s, \text{ use } p_{si} = s$

Note 1: Minimum continuity plate size is required as set forth in Step 19.1 of the design procedure. The designer must determine whether continuity effectively stiffens the column flange and select the appropriate value of  $Y_c$  from the table.

Note 2: This table is based on Table 6.5 of AISC 358-16 and -10, and  $t_s$  corresponds to continuity/stiffener plate thickness.

TABLE A2—SUMMARY OF FOUR-BOLT EXTENDED STIFFENED COLUMN FLANGE YIELD LINE MECHANISM PARAMETER, WHERE CONNECTION IS AT OR NEAR COLUMN END

Stiffened Column Flange Geometry and Yield Line Pattern		Bolt Force Model
Case 1 ( $d_e \leq s$ )	Case 2 ( $d_e > s$ )	
Case 1 ( $d_e \leq s$ )	$Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{fo}} + \frac{1}{2s} \right) \right] + \frac{2}{g} [h_1(p_{fi} + s) + h_0(d_e + p_{fo})]$	
Case 2 ( $d_e > s$ )	$Y_p = \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{s} + \frac{1}{p_{fo}} \right) \right] + \frac{2}{g} [h_1(p_{fi} + s) + h_0(s + p_{fo})]$	
$s = \frac{1}{2} \sqrt{b_p g}$ Note: If $p_{fi} > s$ , use $p_{fi} = s$ .		

Note 1: Minimum continuity plate size is required as set forth in Step 19.1 of the design procedure. The designer must determine whether continuity plates effectively stiffen the column flange and select the appropriate value of  $Y_c$  from the table.

Note 2: This table is based on Table 6.3 of AISC 358-16 and -10. Within the graphs for Case 1 and Case 2,  $b_p$ ,  $t_p$ , and  $t_{bw}$  correspond to the column flange width, column flange thickness, and column web thickness, respectively; and  $t_{bf}$  corresponds to continuity/stiffener plate thickness.

TABLE A3—SUMMARY OF FOUR-BOLT UNSTIFFENED END-PLATE GEOMETRY AND YIELD LINE MECHANISM PARAMETER FOR END-PLATE LINK DESIGN

End-Plate Geometry and Yield Line Pattern	Bolt Force Model

Note 1: This table is based on Table 6.2 of AISC 358-16 and -10. Within the graph for end-plate geometry,  $t_r$  corresponds to link stem thickness.

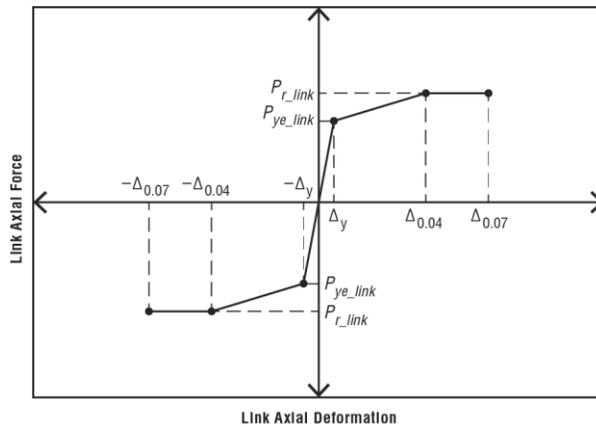


FIGURE A1A—LINK AXIAL FORCE VS. LINK AXIAL DEFORMATION

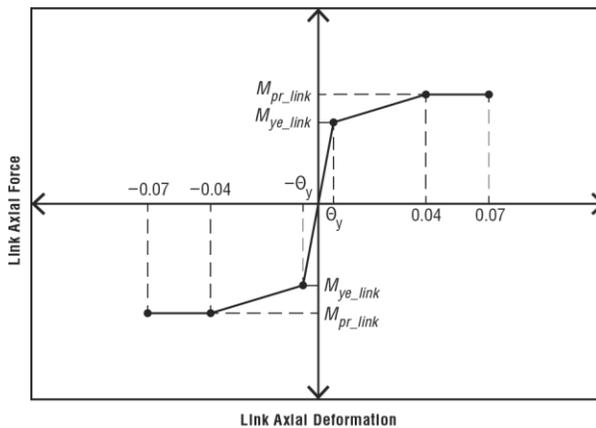


FIGURE A1B—CONNECTION MOMENT VS. ROTATION

FIGURE A1—SST MOMENT CONNECTION MODELING PARAMETERS

**DIVISION: 05 00 00—METALS**  
**Section: 05 12 00—Structural Steel Framing**

**REPORT HOLDER:**

**SIMPSON STRONG-TIE COMPANY INC.**

**EVALUATION SUBJECT:**

**SIMPSON STRONG-TIE® STRONG FRAME® STEEL SPECIAL MOMENT FRAME CONNECTION**

**1.0 REPORT PURPOSE AND SCOPE****Purpose:**

The purpose of this evaluation report supplement is to indicate that the Simpson Strong-Tie® Strong Frame® Steel Special Moment Frame Connection, described in ICC-ES evaluation report [ESR-2802](#), has also been evaluated for compliance with the code noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

**Applicable code edition:**

2020 *City of Los Angeles Building Code* (LABC)

**2.0 CONCLUSIONS**

The Simpson Strong-Tie® Strong Frame® Steel Special Moment Frame Connection, described in Sections 2.0 through 7.0 of the evaluation report [ESR-2802](#), complies with LABC Chapter 22, and is subject to the conditions of use described in this supplement.

**3.0 CONDITIONS OF USE**

The Simpson Strong-Tie® Strong Frame® Steel Special Moment Frame Connection described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report [ESR-2802](#).
- The design, installation, conditions of use and identification of the Simpson Strong-Tie® Strong Frame® Steel Special Moment Frame Connection are in accordance with the 2018 *International Building Code*® (IBC) provisions noted in the evaluation report [ESR-2802](#).
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.

This supplement expires concurrently with the evaluation report, reissued January 2024.