ICC EVALUATION SERVICE®

ICC-ES Evaluation Report

ESR-2802

Reissued January 2024	*This report also contains:*
Revised September 2024	**- LABC Supplement**
Subject to renewal January 2025	**- CBC Supplement**

DIVISION: 05 00 00—METALS Section: 05 12 00—Structural Steel Framing	REPORT HOLDER: SIMPSON STRONG-TIE COMPANY INC. SIMPSON Strong-Tie®	EVALUATION SUBJECT: SIMPSON STRONG-TIE® YIELD-LINK® MOMENT CONNECTION FOR STEEL MOMENT FRAME SYSTEMS	

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021, 2018, 2015 and 2012 *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 Yield-Link® moment 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 including two-piece T-stub Yield-Link and End-plate Yield-Link.

3.0 DESCRIPTION

3.1 General:

The Yield-Link moment 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 Yield-Link moment connection system satisfies all applicable requirements under the 2021, 2018, 2015 and 2012 IBC; ANSI/AISC 341-16, including Section E2, Section E3, and Section K1 under the 2021 and 2018 IBC; and ANSI/AISC 341-10, including Section E2, Section E3, and Section K1 under the 2015 and 2012 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- or double-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 in. (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. 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 structural fuses 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 in. (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.

3.2.3 Bolts, Washers and Nuts: Bolts, washers and nuts must conform to Section A3.3 of ANSI/AISC 360 and ANSI/AISC 348 (RCSC *Specifications*). Bolts for the Yield-Link structural fuse stem-to-beam flange connection must comply with ASTM A325 Type 1, A490 Type 1 or ASTM F3111 structural bolts, or with ASTM F1852, ASTM F2280, ASTM F3148 Type 1 or ASTM F3043 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. 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, 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.

4.0 DESIGN AND INSTALLATION

4.1 Structural Design and Prequalification Limits:

Steel special moment frames using the Yield-Link moment 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, Ω_0	DEFLECTION AMPLIFICATION FACTOR, C_d
SMF	8	3	$5^1/_2$
IMF	$4^1/_2$	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 such as 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 ANSI/AISC 360 Section J2, is required.

For the Yield-Link moment 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 use with two-piece T-stub Yield-Link, beam depth is limited to a maximum of W44 for rolled shapes and beam depth for built-up members must not exceed the maximum depth of 60 in. (1524 mm).
3. For use with End-plate Yield-Link, beam depth must be 8.5 in. (215.9 mm).
4. The beam flange and web width-to-thickness ratios must not exceed λ_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 in. (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 use with two-piece T-stub Yield-Link®, column depth is limited to a maximum of W44 for rolled shapes and column depth for built-up members must not exceed the maximum depth of 60 in. (1524 mm).
4. For used with End-plate Yield-Link, 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 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, must meet the requirements of the ANSI/AISC *Seismic Provisions* per item 7 above.

4.1.3 Buckling-Restraint Assembly Requirements:

1. The Yield-Link structural fuse stem must be 1.5 in (38 mm) thick maximum and 0.5 in. (12.7 mm) thick minimum for the two-piece T-stub Yield-Link, and must be 0.5 in. (12.7 mm) thick for the End-plate Yield-Link.
2. The maximum width of the reduced portion of the Yield-Link stem must be 3.5 in. (88.9 mm) for the End-plate Yield-Link, and 8 in. (203 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 in. (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 the 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 as 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 in. (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-Link 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 in. long) are permitted in the column flanges. Holes in the shear plate(s) must be slotted to accommodate a connection rotation of at least 0.07 radians. The width of the slot must be limited to 1/16 in. (1.59 mm) greater than the shear plate bolt diameter. The diameter of the center/central hole in the shear plate and beam web must be limited to 1/16 in. (1.59 mm) greater than the shear plate bolt diameter. The holes in the Yield-Link stem must be limited to 1/16 in. (1.59 mm) greater than the Yield-Link stem-to-beam flange bolt diameter. See Figures 1A and 1B.

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

Beam copes at the Yield-Link® structural fuses 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 $\sum \boldsymbol{M}_{\boldsymbol{pb}}^{*}$ must be taken equal to $\sum(\boldsymbol{M}_{\boldsymbol{pr}} + \boldsymbol{M}_{\boldsymbol{uv}})$, kip-in. (N-mm)

   where:

   M_{pr} = Probable maximum moment capacity, computed in accordance with EQ A-9 of the *Design Procedure*, 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 $\boldsymbol{V}_{\boldsymbol{u}}(\boldsymbol{a} + \boldsymbol{d}_{\boldsymbol{c}}/\boldsymbol{2})$, kip-in. (N-mm).

   V_u = Shear at the shear plate connection, computed in accordance with Step 9 of the *Design Procedure*, kips (N).

   a = The distance from the centerline of bolts in the shear plate to the face of the column as shown in Figure 3c, 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 W36 and smaller 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 pretensioned ASTM A325, ASTM A490 or ASTM F3111 structural bolts, or ASTM F1852 or ASTM F2280, ASTM F3148 Type 1 or ASTM F3043 twist-off type structural bolt assemblies complying with ANSI/AISC 348 (RCSC *Specifications*). 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 either with A325 bolts installed either as snug-tight or pretensioned, or with A490 bolts installed as pretensioned, 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 ANSI/AISC 348 (RCSC *Specifications*) and ANSI/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 Yield-Link® moment connection 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: Yield-Link moment connections 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 Yield-Link moment connections.

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 ANSI/AISC 360-16 and Chapter J of ANSI/AISC 341-16 for the 2021 and 2018 IBC or Chapter N of ANSI/AISC 360-10 and Chapter J of ANSI/AISC 341-10 for the 2015 and 2012 IBC; applicable portions of ANSI/AISC 303-16 and Clause 7 of ANSI/AWS D1.8-2016 for the 2021 and 2018 IBC or applicable portions of ANSI/ AISC 303-10 and Clause 7 of ANSI/AWS D1.8-2009 for the 2015 and 2012 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, 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 must be 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, for fabricator approval. Special inspection of welding required by 2021 IBC Section 1705.13, 2018 and 2015 IBC Section 1705.12 or 2012 IBC Section 1705.11 must be completed during the manufacturing process, as described in the manufacturer's quality documentation.

5.0 CONDITIONS OF USE:

The Simpson Strong-Tie Yield-Link moment connection described in this report complies with, or is a suitable alternative to what is specified in, the codes listed in Section 1.0 of this report, subject to the following conditions:

5.1 The Simpson Strong-Tie Yield-Link 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 Yield-Link 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 Yield-Link moment connection in SMFs and IMFs with orthogonally loaded columns, as described in ANSI/AISC 341-16 Sections D1.4a, E3.4a, and K2.4a must be in accordance with the requirements of ANSI/AISC 341.

5.4 Structural design drawings and specifications must comply with Section A4 of ANSI/AISC 341-16 under the 2021 and 2018 IBC or Section A4 of ANSI/AISC 341-10 under the 2015 and 2012 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 Yield-Link® moment 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 Yield-Link moment connection last four digits of mill certification heat number, six-digit date of manufacture and Yield-Link Part ID are marked on the stem of each Yield-Link structural fuse. A label, including the ICC-ES evaluation report number (ESR-2802), must be applied near each moment connection. A Simpson Strong-Tie Yield-Link moment connection 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 Yield-Link moment connections, the following notice of intellectual property must be affixed before release for intended use: Simpson Strong-Tie Yield-Link moment connections and Yield-Link structural fuse are protected under one or more of the following patents and applications: US patent no. 11,346,102 B2, US patent no. 11,299,880 B2, US patent no. 11,203,870 B2 and US patent publication no. 11,795,721 B2, 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.
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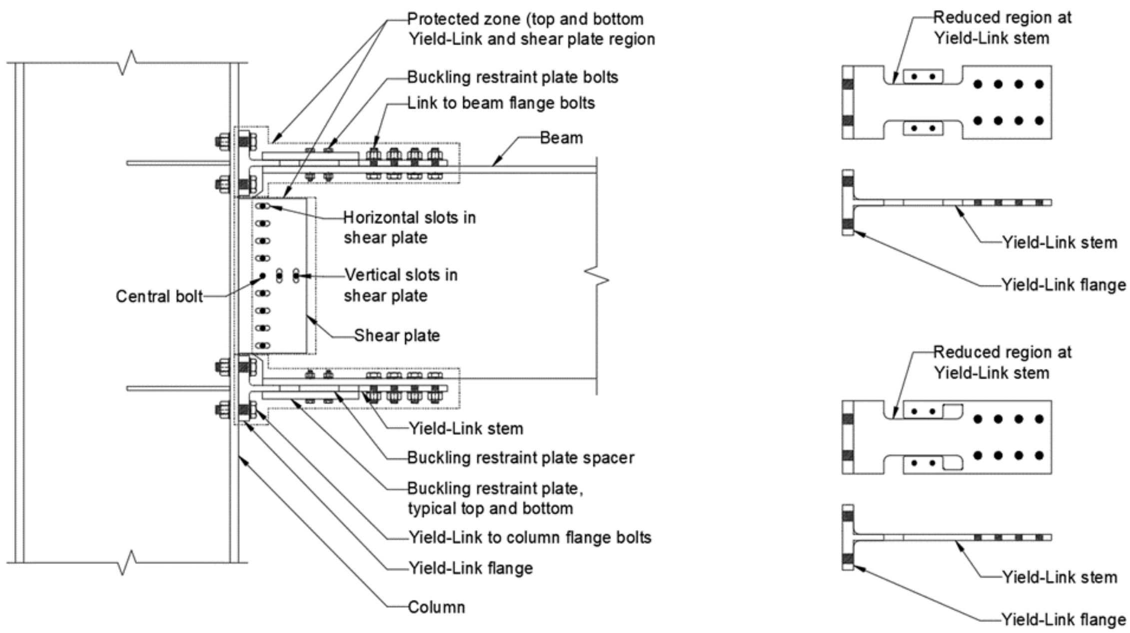


FIGURE 1A — TWO-PIECE T-STUB YIELD-LINK®

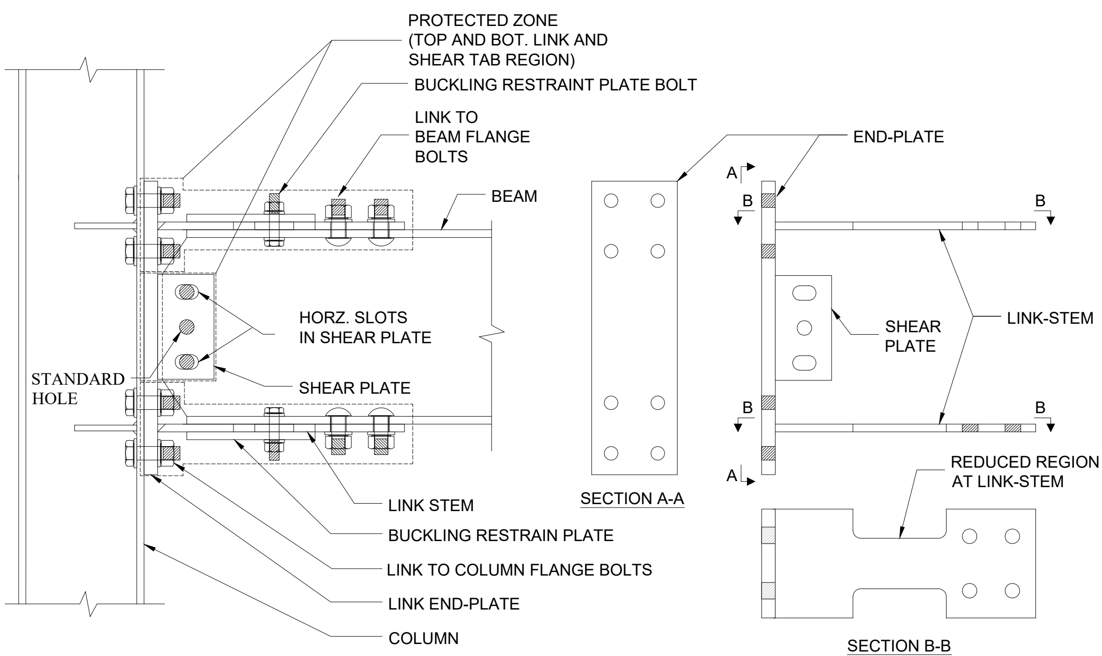
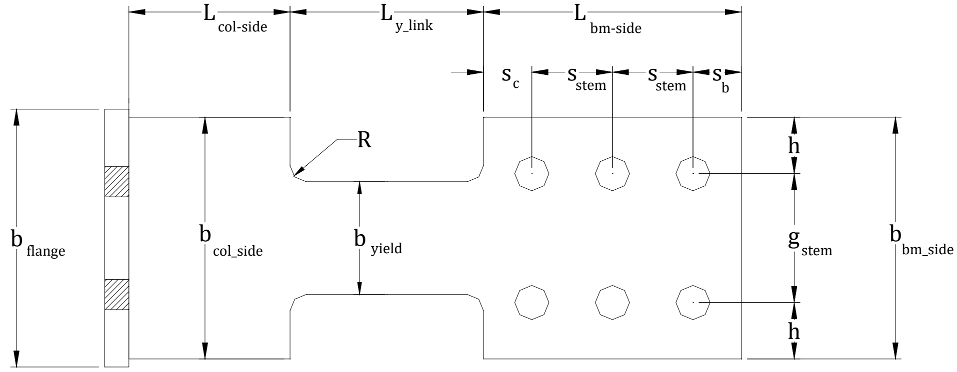
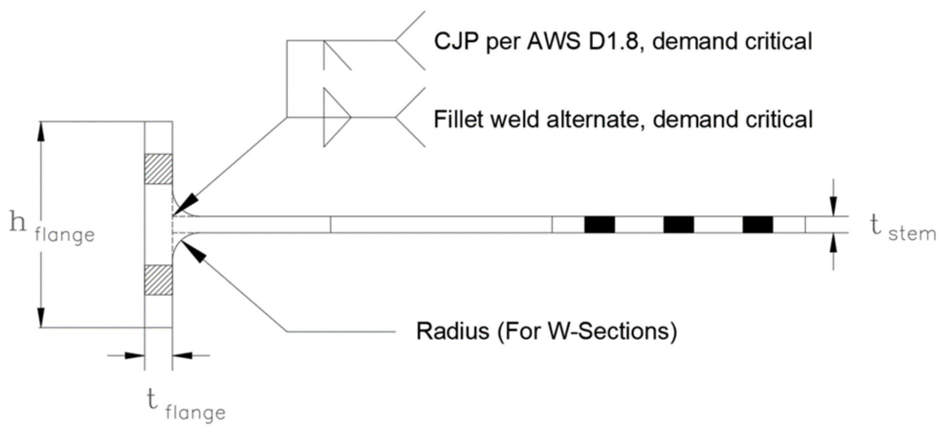


FIGURE 1B — END-PLATE YIELD-LINK

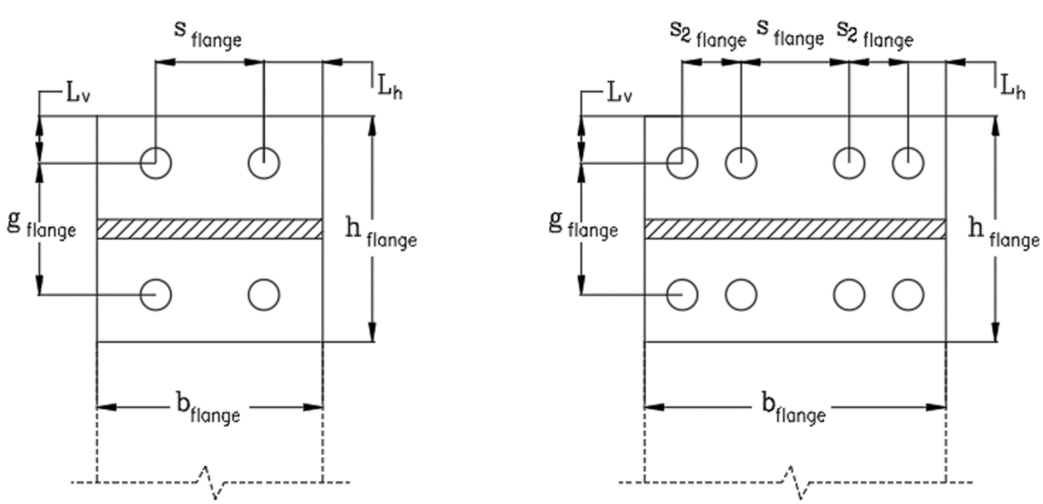
FIGURE 1—SIMPSON STRONG-TIE® YIELD-LINK® MOMENT CONNECTION



a) Yield-Link Structural Fuse Plan View

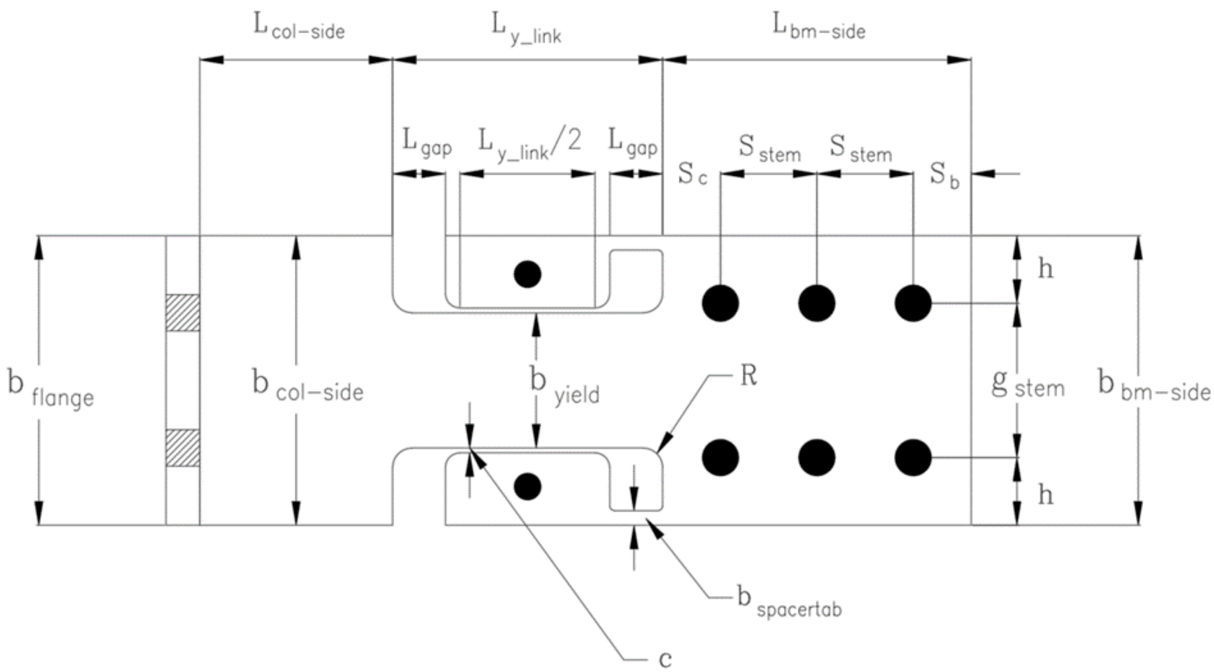


b) Yield-Link Structural Fuse Elevation View

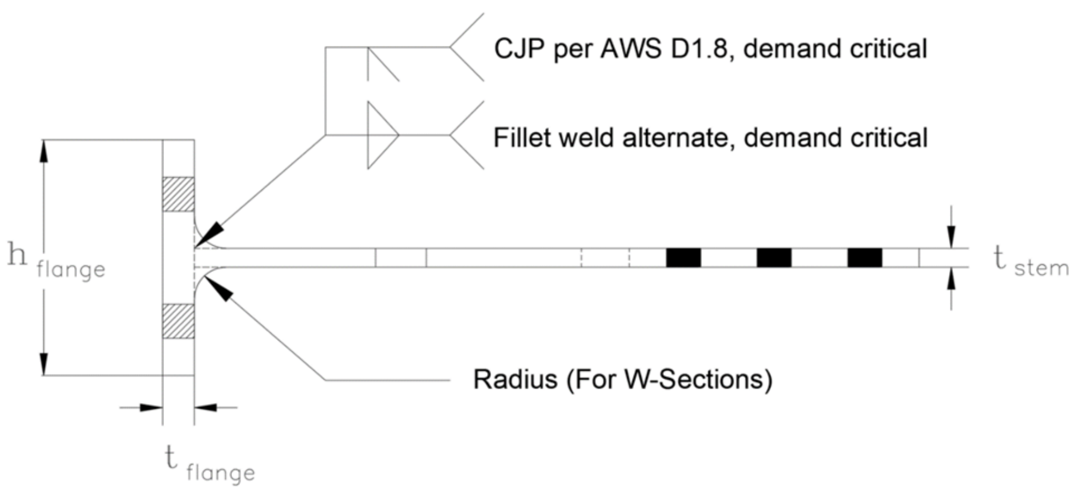


(c) 1.Yield-Link Flange View (4 Bolts) & 2. Yield-Link Flange View (8 Bolts)

FIGURE 2 — YIELD-LINK® STRUCTURAL FUSE GEOMETRIES

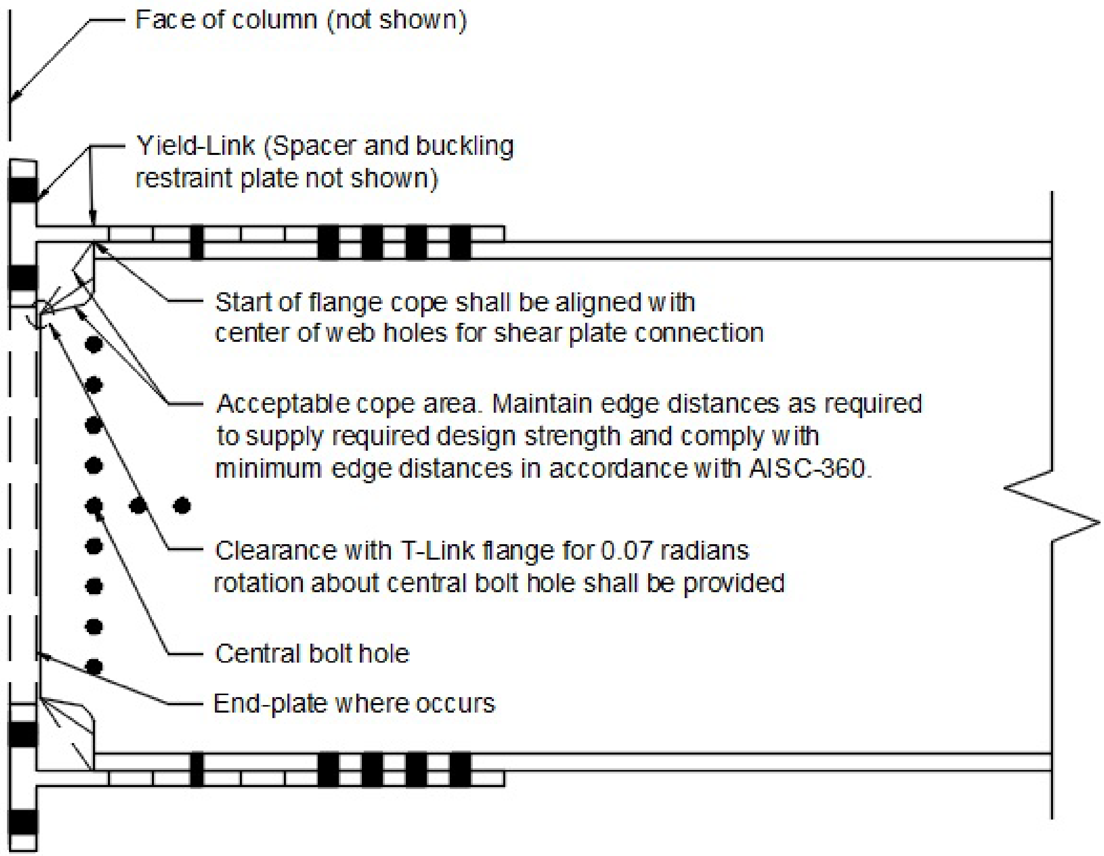


c) Yield-Link (with Integrated Spacers) Structural Fuse Plan View

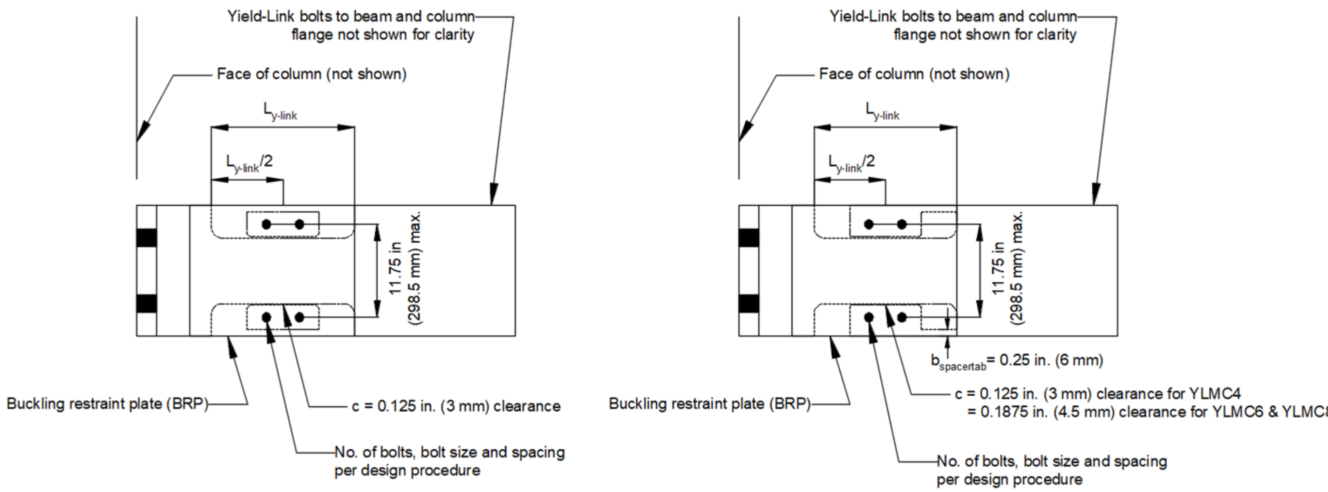


d) Yield-Link (with Integrated Spacers) Structural Fuse Elevation View

FIGURE 2 — YIELD-LINK® STRUCTURAL FUSE GEOMETRIES (CONTINUED)

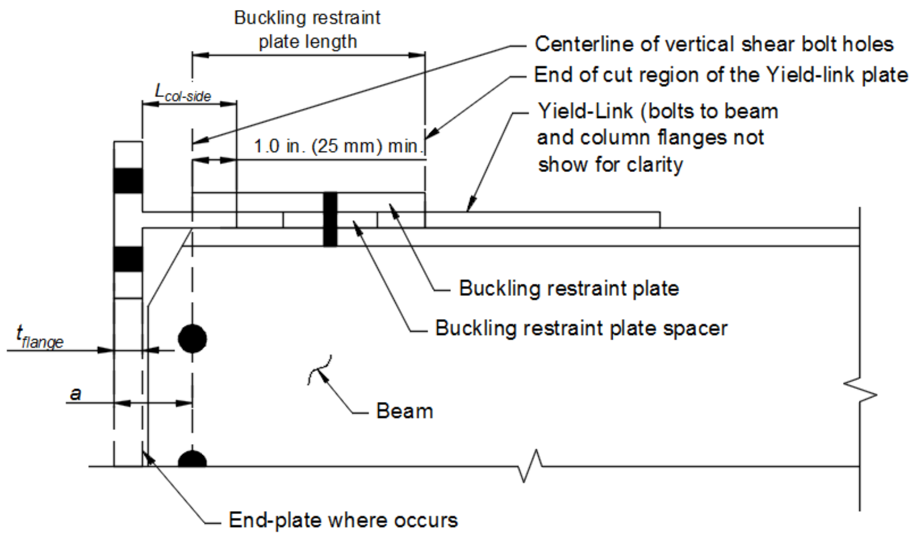


a) Beam Coping

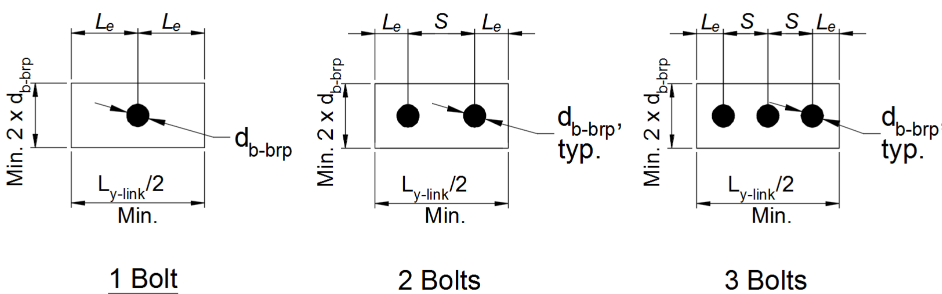


b) Buckling-Restraint Spacer Plate Placement

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 the 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 yield strength of the Yield-Link structural fuse from analysis described in Step 1.

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

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

Where

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

d = the beam depth, in. (mm)

ϕ_b = 0.9

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

$A'_{y\text{-}link}$ = estimated required Yield-Link yield area, in.^{2} (mm^2)

$F_{y\text{-}ink}$ = 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 the column side (see Figure 2a):

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

Step 4.2: Non-reduced Yield-Link stem length at column side, $L_{col\text{-}side}$, must have a maximum length equal to 5 in. (127 mm), and a minimum length equal to a - t_{flange}+ 1 in. (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 the stem, t_{stem}, must be 0.5 in. (12.7 mm) for End-plate Yield-Link, and must be 0.5 in. (12.7 mm) minimum and 1.5 in. (38 mm) maximum for the two-piece T-stub Yield-Link.

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

But the value of $b_{yield,req'd}$ must not exceed the least of 0.5 $b_{col\text{-}side}$, 0.5 $b_{bm\text{-}side}$, and 3.5 in. (89 mm) for the End-plate Yield-Link, and 8 in. (203 mm) for the two-piece T-stub Yield-Link.

Step 6. Determine the minimum Yield-Link stem yielding length such that the axial strain in the straight portion of the stem 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 the 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.

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

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

Where

$A_{y\text{-}link}$ = area of the reduced link area (b_{yield} × t_{stem}), in.^{2} (mm^2)

$P_{ye\text{-}link}$ = expected yield strength of Yield-Link®, kips (N)

$P_{r\text{-}link}$ = 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.

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

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

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

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

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

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

Where

s_c = distance from reduced section of Yield-Link to center of first row of bolt holes

= minimum of 1.5$d_{b\text{-}stem}$, 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,

≥ **minimum of** 2 ⅔ $d_{b\text{-}stem}$, in. (mm)

n_{rows} = number of rows of bolt holes determined in Step 8.1.

Step 8.4: To resist $P_{r\text{-}link}$ 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:

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

Where

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

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

n_{side} = number of sides of the beam with moment 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 $P_{r\text{-}link}$ 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, $d_{b\text{-}flange}$. 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:

$$r_t = \frac{P_{r-link}}{n_{bolt-Link-flg}} \quad \text{(EQ A-10)}$$

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:

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

$n_{bolt\text{-}Link\text{-}flg}$ = number of bolts in Yield-Link flange-to-column flange connection.
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_0 = End-plate geometry, as shown in Table A3, in. (mm)
h_1 = End-plate geometry, as shown in Table A3, in. (mm)
π = 3.14159
ϕ_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.^{2} (mm^2)

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

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

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

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_{nbolts} = either p_{4bolts} or p_{8bolts} for four bolts or eight bolts at the tension flange, respectively

where

p_{4bolts} = min (L_h, 1.75b) + min ($\frac{s_{flange}}{2}$, 1.75b) in. (mm), as shown in Figure 2c.1

p_{8bolts} = min. of [$b_{flange}/4$; s_{flange}; $s2_{flange}$; (min (L_h, 1.75b) + $\frac{s2_{flange}}{2}$); 3.5b], in. (mm), as shown in Figure 2c.2

$d_{b\text{-}flange}$ = 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

ϕ_d = 1.0

Width of End-plate must be no less than $b_{col\text{-}side}$ 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:

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

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

Where:

F_{pf} = $P_{r\text{-}link}$ from Equation A-6 in Step 7, kips (N).
b_p = width of the End-plate, in. (mm)
t_p = thickness of the End-plate provided, in. (mm)
F_{yp} = specified minimum yield stress of End-plate, ksi (MPa).
F_{up} = specified minimum tensile stress of End-plate, ksi (MPa).
A_n = $t_p\,[b_p - 2(d_b + {}^1/_8)]$, in.^{2}
= $t_p\,[b_p - 2(d_b + 3)]$, mm^2
d_b = bolt diameter for the End-plate-to-column-flange connection, in. (mm)

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

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

Where:
V_u = shear force at the end of the beam, kips (N), per Step 9
n_i = 2, the number of inner bolts
n_o = 2, the number of outer bolts
r_{ni} = $1.2 \times L_c \times t \times F_u < 2.4 \times d_b \times t \times F_u$, kips (N).
r_{no} = $1.2 \times L_c \times t \times F_u < 2.4 \times d_b \times t \times F_u$, kips (N).
L_c = 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 = diameter of the bolt, in. (mm)
t = End-plate or column flange thickness, in. (mm)
F_u = specified minimum tension strength of End-plate or column flange material, ksi (MPa).

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 $b_{col\text{-}side}$ at the column side, $P_{r\text{-}weld}$, as:

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

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 must not be less than 0.875 in. (22 mm):

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

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 must not be less than 0.4 in. (10 mm):

$$t_{bf\_min} = \sqrt{\frac{4b'T}{\emptyset_d p_e F_{u\_bm}}} \qquad \text{(EQ A-21)}$$

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).

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

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

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 = 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.^{4} (mm^4)

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

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

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

Step 11.3 Determine the BRP bolt size and quantity

$$V_{uy} = \frac{4cP_{r-link}}{\sqrt{\frac{\pi^2 E_t I_x}{P_{r-link}}\left[1 + \left(\frac{t_{stem}}{2c} + 1.013\right)^{-1}\right]}} \quad \text{(EQ-A28)}$$

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^4 (mm^4).

Design BRP bolts per ANSI/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 stem, kip/in. (N/mm)

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

Where:

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

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

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

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

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

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

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

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

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

For Beams deeper than W36,

$M_{pr\text{-}link} = [P_{r\text{-}link} \times (d \times t_{stem})] + M_{YLf}$ (EQ A-33a)

Where:

$M_{YLf} = T_{UX} \times N_{BRP\_bolts} \times 0.3 \times (d + t_{stem})$ (EQ A-33b)

Δ_y = axial deformation in Yield-Link structural fuse at 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 strucutural fuse 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 structural fuse at a connection rotation of 0.07 rad., in. (mm)

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

θ_y = connection rotation at 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® Yield-Link 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, ϕM_n, taking $\phi = 0.9$ and M_n as $M_{ye\text{-}link}/R_y$

(b) The drift complies with applicable limits;

Adjust the Yield-Link moment connection 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

ϕ_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.^{3} (mm^3)

Step 13.3: For built-up beams and built-up columns, the strength of web-to-flange welds must equal 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. Minimum weld size must satisfy AISC *Specification* Table J2.4.

Step 13.4: For built-up beams, in addition to requirements in Steps 13.1 and 13.3, at the Yield-Link® stem-to-beam flange connection region, double-sided fillet welds must be used for web-to-flange connection, and the minimum length of the double-sided fillet welds must cover the distance from the end of the beam to the end of the Yield-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 must be used to connect the column web-to-column flanges at the panel zone region. The minimum length of double-sided fillet welds must extend 6 in. (152.4 mm) above the top Yield-Link flange and 6 in. (152.4 mm) below the bottom Yield-Link flange. 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) must be welded to the web of the continuous I-shaped section with double-sided fillet welds. Fillet welds must be sized to develop the minimum of (1) the shear strength of the column section per ANSI/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 must 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.

$P_{u\text{-}sp}$ = 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
2) The axial force calculated using the load combinations of the applicable building code, including the amplified seismic load

For moment connection at one-end of the beams deeper than W36 only, required axial strength shall be determined as follows:

$$P_{u\text{-}sp\text{-}one\text{-}sided} = \left(P_{u\text{-}sp} + \frac{M_{YLf}}{d/2}\right)$$

$M_{u\text{-}sp}$ = 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, $V_{u\text{-}bolt}$, kips (N):

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

Where

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

$n_{bolt\text{-}sp\text{-}horz}$ = 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, $d_{b\text{-}sp}$, for all bolts, which complies with ANSI/AISC 360.

Step 15.2: Determine shear plate geometry to accommodate a connection rotation of ±0.07 radians

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

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

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

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

Where

$d_{b\text{-}sp}$ = 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 single-shear plate or first shear plate of the double-shear plate to column face or End-plate joint to develop the plate in shear, tension and bending. For double-sided fillet welds, the minimum leg size shall be $^5/_8$ t_{sp}, where t_{sp} is the shear plate thickness. Design PJP weld for required demand load for connecting second shear plate of double-shear plate connection. Design welds for the second shear plate to resist minimum of 50% of moment, shear and axial force determined in Step 15.

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 the 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 Yield-Link® structural fuse probable maximum axial strengths determined in Step 7. The requirements of ANSI/AISC 341 Sections E3.6e.1 and E3.6e.2 need not apply. Doubler plates must be used as required. Check doubler plate shear strength in accordance with AISC *Specification* Section G2.1, where the doubler plate shear strength shall be proportioned such that C_{v1} = 1.0. Doubler plate detailing shall comply with ANSI/AISC 341-16 Section E3.6e.3, except Equation E3-7 need not apply.

Step 17. Check column web for concentrated force(s) of $P_{r\text{-}link}$, in accordance with ANSI/AISC 360.

Step 18. Check minimum column flange thickness for flexural yielding:

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

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, $M_{pr\text{-}link}$

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

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

where Y_c is the unstiffened column yield line mechanism parameter from Table A1 or A4. Therefore, the equivalent column flange design force is

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

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:

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

Where

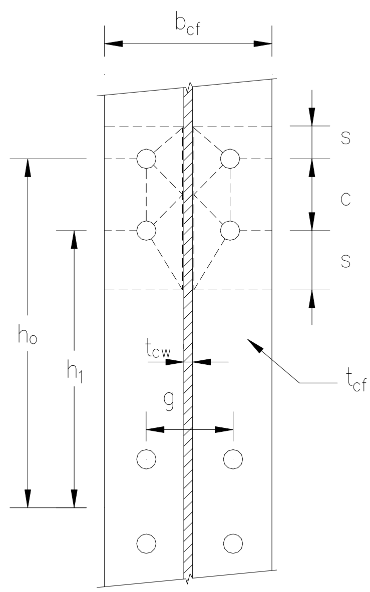
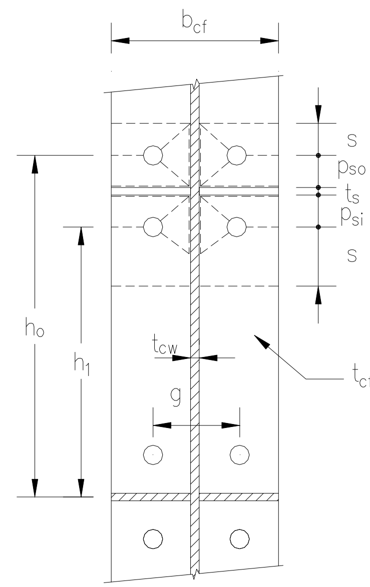
ϕR_n = the design strengths from Steps 17 and 18, kips, (N)

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.

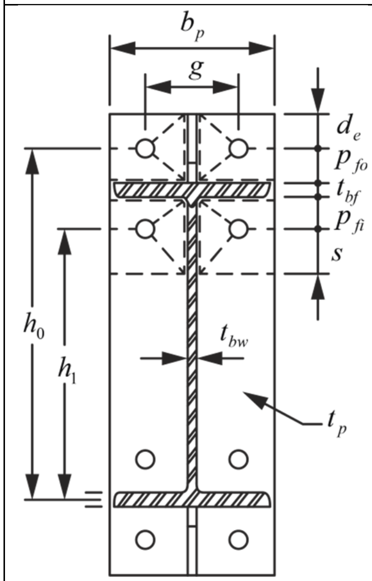
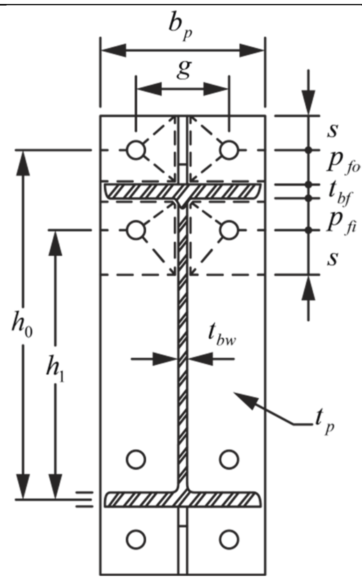
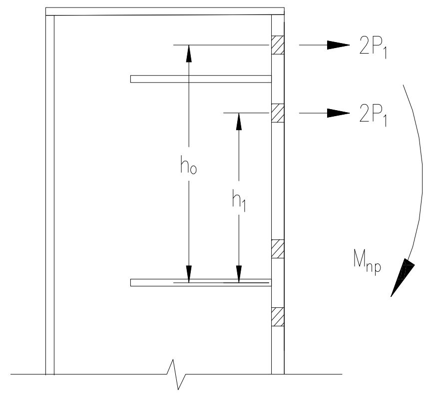
The continuity or stiffener plate must conform to Section J10.8 of ANSI/AISC 360, with a minimum thickness of $^1/_4$ in. (6 mm). For flanged cruciform columns, the continuity or stiffener plate must be a minimum thickness of ½ of the Yield-Link stem thickness and must be in each quadrant of the column. Fillet welds, partial joint penetration (PJP) welds, or complete joint penetration (CJP) welds shall be used for the connection between the continuity plate and 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**
	
Unstiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_0\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$ $s=\frac{1}{2}\sqrt{b_{cf}g}$
Stiffened Column Flange	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}+\frac{1}{p_{si}}\right)+h_0\left(\frac{1}{s}+\frac{1}{p_{so}}\right)\right]+\frac{2}{g}\left[h_1\left(s+p_{si}\right)+h_0\left(s+p_{so}\right)\right]$ $s=\frac{1}{2}\sqrt{b_{cf}g}$ Note: If $p_{si}>s$, 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 ANSI/AISC 358-16 and -10, and t_s corresponds to continuity/stiffener plate thickness.

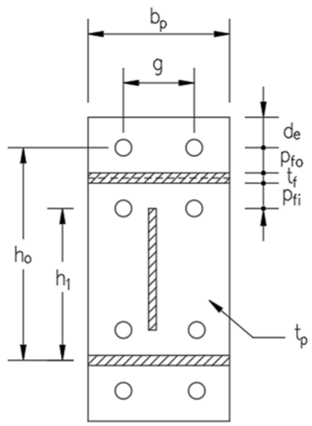
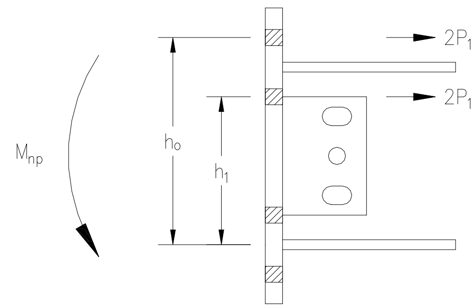
TABLE A2—SUMMARY OF FOUR-BOLT EXTENDED STIFFENED COLUMN FLANGE YIELD LINE MECHANISM PARAMETER, WHERE CONNECTION IS AT OR NEAR COLUMN END

<table>
<tr><th colspan="2">Stiffened Column Flange Geometry and Yield Line Pattern</th><th rowspan="2">Bolt Force Model</th></tr>
<tr><th>Case1 $\left(d_e \le s\right)$</th><th>Case 2 $\left(d_e > s\right)$</th></tr>
<tr><td>
</td><td>
</td><td>
</td></tr>
<tr><td>Case1 $\left(d_e \le s\right)$</td><td colspan="2">$$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+h_0\left(\frac{1}{p_{fo}}+\frac{1}{2s}\right)\right]+\frac{2}{g}\left[h_1\left(p_{fi}+s\right)+h_0\left(d_e+p_{fo}\right)\right]$$</td></tr>
<tr><td>Case2 $\left(d_e > s\right)$</td><td colspan="2">$$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)+h_0\left(\frac{1}{s}+\frac{1}{p_{fo}}\right)\right]+\frac{2}{g}\left[h_1\left(p_{fi}+s\right)+h_0\left(s+p_{fo}\right)\right]$$</td></tr>
<tr><td colspan="3">$s = \frac{1}{2}\sqrt{b_p g}$ Note: If $p_{fi} > s$, use $p_{fi} = s$.</td></tr>
</table>

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 ANSI/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 PARAMETERFOR 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 ANSI/AISC 358-16 and -10. Within the graph for End-plate geometry, t_f corresponds to link stem thickness.

TABLE A4 – SUMMARY OF EIGHT-BOLT EXTENDED STIFFENED COLUMN FLANGE YIELD LINE MECHANISM PARAMETER FOR T-STUB YIELD-LINK® DESIGN

Configuration	Yield Line Parameter
Continuous unstiffened column [Fig. A2 (a)]	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_2\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(s+\frac{3}{4}c\right)+h_2\left(s+\frac{1}{4}c\right)\right]+\frac{g}{2}$
Continuous column stiffened between the bolts [Fig. A2 (b)]	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}+\frac{1}{p_{so}}\right)+h_2\left(\frac{1}{p_{si}}+\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(s+p_{so}\right)+h_2\left(p_{si}+s\right)\right]$ Note: Use $p_{si}=s$ if $p_{si} > s$ Use $p_{so}=s$ if $p_{so} > s$
Top of column unstiffened [Fig. A2 (c)]	$Y_c = \frac{b_{cf}}{2}\left[h_2\left(\frac{1}{s}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(d_e+\frac{3}{4}c\right)+h_2\left(s+\frac{1}{4}c\right)\right]+\frac{3g}{4}$
Top of column with cap plate [Fig. A2 (d)]	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{p_{cp}}\right)+h_2\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(p_{cp}+\frac{3}{4}c\right)+h_2\left(s+\frac{1}{4}c\right)\right]+\frac{g}{2}$ Note: Use $p_{cp}=s$ if $p_{cp} > s$
Top of column stiffened between the bolts [Fig. A2 (e)]	$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{p_{so}}\right)+h_2\left(\frac{1}{p_{si}}+\frac{1}{s}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(p_{so}+d_e\right)+h_2\left(p_{si}+s\right)\right]+\frac{g}{4}$ Note: Use $p_{si}=s$ if $p_{si} > s$ Use $p_{so}=s$ if $p_{so} > s$

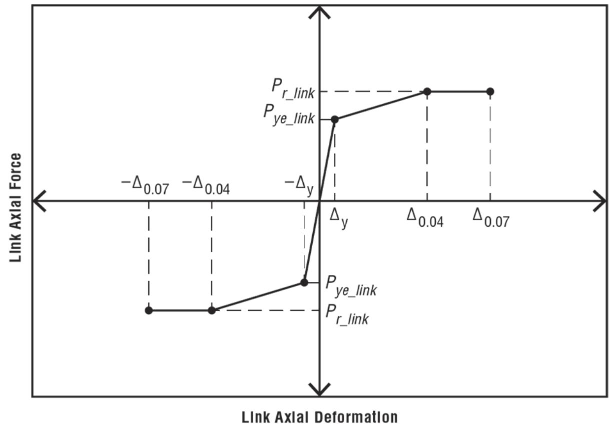


FIGURE A1A—LINK AXIAL FORCE VS. LINK AXIAL DEFORMATION

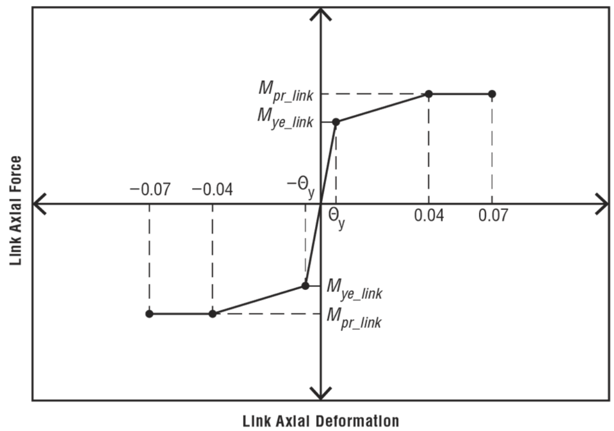


FIGURE A1B—CONNECTION MOMENT VS. ROTATION

FIGURE A1 — SIMPSON STRONG-TIE® YIELD-LINK® MOMENT CONNECTION MODELING PARAMETERS

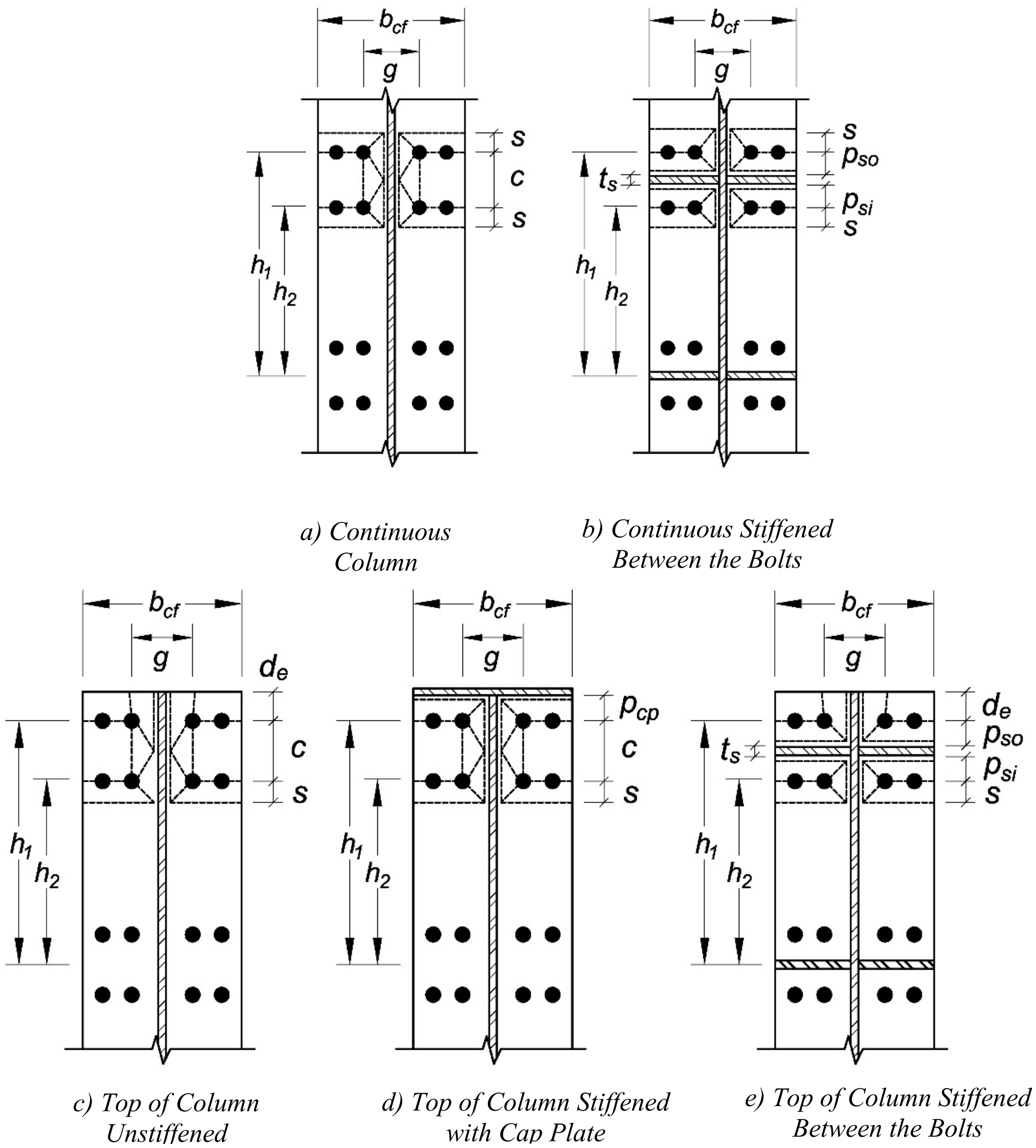


FIGURE A2 – SUMMARY OF EIGHT-BOLT EXTENDED STIFFENED COLUMN FLANGE YIELD LINE MECHANISM PARAMETER FOR T-STUB YIELD-LINK® DESIGN

ICC-ES Evaluation Report

ESR-2802 LABC Supplement

Reissued January 2024

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

REPORT HOLDER:

SIMPSON STRONG-TIE COMPANY INC.

EVALUATION SUBJECT:

SIMPSON STRONG-TIE® YIELD-LINK® MOMENT CONNECTION FOR STEEL MOMENT FRAME SYSTEMS

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Simpson Strong-Tie® Yield-Link® moment 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:

- 2023 *City of Los Angeles Building Code* (LABC)

2.0 CONCLUSIONS

The Simpson Strong-Tie Yield-Link moment 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 Yield-Link moment 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 Yield-Link moment connection are in accordance with the 2021 *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.
- The steel moment frame system shall be produced in the shop of an approved City of Los Angeles Fabricator, in accordance with LABC Section 91.200 “Fabricated Item” and LABC Section 96.203.

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

ICC-ES Evaluation Report

ESR-2802 CBC Supplement

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Applicable code edition:

- 2022 *California Building Code* (CBC)

For evaluation of applicable chapters adopted by the California Office of Statewide Health Planning and Development (OSHPD) AKA: California Department of Health Care Access and Information (HCAI) and the Division of State Architect (DSA), see Sections 2.1.1 and 2.1.2 below.

2.0 CONCLUSIONS

2.1 CBC:

Simpson Strong-Tie Yield-Link moment connection, described in Sections 2.0 through 7.0 of the evaluation report ESR-2802, complies with CBC Chapter 22, provided the design and installation are in accordance with the 2021 *International Building Code®* (IBC) provisions noted in the evaluation report and the additional requirements of CBC Chapters 16 and 17, as applicable.

2.1.1 OSHPD:

The applicable OSHPD Sections of the CBC are beyond the scope of this supplement.

2.1.2 DSA:

The applicable DSA Sections of the CBC are beyond the scope of this supplement.

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