ICC-ES Evaluation Report

ESR-4342

Reissued March 2024 | *This report also contains:*

- LABC Supplement

Subject to renewal March 2025 | **- CBC Supplement**

DIVISION: 05 00 00—METALS **Section: 05 12 00—Structural Steel Framing**	**REPORT HOLDER: SIMPSON STRONG-TIE COMPANY INC**	**EVALUATION SUBJECT: YIELD-LINK® BRACE CONNECTION STEEL BRACED FRAME SYSTEM WITH FUSE PLATES**	

1.0 EVALUATION SCOPE

Compliance with the following codes:

- 2021 and 2018 *International Building Code®* (IBC)
- 2021 and 2018 *International Existing Building Code®* (IEBC)

For evaluation for compliance with codes adopted by the *Los Angeles Department of Building and Safety (LADBS)*, see ESR-4342 LABC Supplement.

Property evaluated:

- Structural

2.0 USES

The Yield-Link® Brace Connection (YLBC) steel braced frame system utilizing steel Yield-Link® Fuse Plates is recognized for use as the main lateral force-resisting system to resist wind and seismic loads.

3.0 DESCRIPTION

3.1 General:

The Yield-Link® Brace Connection steel braced frame is a lateral load resisting system consisting of Yield-Link® Fuse Configurations that connect the ends of structural steel brace members to gusset plates via bolted connections. Fuse Configurations consist of Fuse Plates of unique geometry, U-shims and cover plates to confine the Fuse Plates and limit their buckling behavior.

The Yield-Link® Fuse Plates are specifically designed to accommodate large inelastic deformations and are expected to be the primary source of energy dissipation during a moderate to severe seismic event. See Figure 2 for illustrative details of Fuse Plates. The Yield-Link® Fuse Plates may be replaced if damaged in a significant seismic event.

Pairs of Fuse Plates may be combined on each side of the gusset plate in a brace-to-gusset connection to incrementally increase the capacity. The predefined combinations of Fuse Plate pairs, U-shims, and cover plates are referred to as Fuse Configurations as detailed in Figures 3 and 4. Each Fuse Configuration identification of "YLBCX-XXX-X", correlates to a design strength shown in Table 1. The strength and stiffness parameters associated with each Fuse Configuration in the elastic and inelastic range are shown in Table 1 and Figure 1. A maximum of two Fuse Configurations may be used with a single diagonal brace, one at each end. YLBC may utilize brace configurations in accordance with Section 3.2.

The YLBC is evaluated as an alternative structural system in accordance with Section 12.2.1.1 of ASCE 7-16 and ICC-ES AC494 Annex B.

3.2 Yield-Link® Brace Connection Steel Braced Frame System Configurations:

The YLBC is permitted for use with single diagonal, V-, inverted V-, and X-braced configurations. K-braced frames are not permitted.

The angle of brace members measured from horizontal must not be less than 20° and must not be greater than 70° except for rooftop structures not exceeding two stories in height and 10% of the total structure weight or other supported structural systems with a weight equal to or less than 10% of the weight of the structure. Horizontal spacing between column centerlines at each end of brace bays must not exceed 45 feet (13.7 m) for single diagonal and X- braced configurations and must not exceed 60 feet (18.3 m) for V- and inverted V- configurations. See table below.

Brace Configuration	Min. angle[1] (degrees)	Max. angle[1] (degrees)	Max. column spacing (ft)
Single Diagonal	20	70	45
X-	20	70	45
V- and Inverted V-	20	70	60

[1] Angle measured from horizontal. Minimum and maximum angles may be exceeded for rooftop structures not exceeding two stories in height and 10% of the total structure weight or other supported structural systems with a weight equal to or less than 10% of the weight of the structure.

Fuse Configurations and braces may be concentrically or eccentrically connected to beams and columns provided eccentricities do not exceed the beam depth. Member and connection forces resulting from eccentric connections must be addressed in member design while maintaining the Fuse Plates as the expected source of inelastic deformation.

A multi-tiered braced frame is defined as a braced frame configuration with two or more levels of bracing between diaphragm levels or locations of out-of-plane bracing. The YLBC may be used in multi-tiered braced frames provided connections of horizontal struts to columns at each tier provide torsional bracing of the column in accordance with Appendix 6 of AISC 360-16.

Beams, columns, and struts in the YLBC system must be rolled wide-flange members or hollow-structural sections (HSS) only for new construction. Diagonal braces must be rolled wide-flange members for new and existing construction. Diagonal brace depths must be limited to W12 and W14 members for YLBC6 Fuse Configurations, and W14 members for YLBC8 Fuse Configurations.

3.3 Material:

3.3.1 Structural Shapes: Hot-rolled wide flange shapes must conform to ASTM A992. HSS shapes must conform to ASTM A500 or A1085. WT members used in the brace to gusset connections without Fuse Plates must conform to ASTM A992. When the YLBC system is designed and detailed in accordance with AISC 341-16 (R=8), Charpy V-Notch (CVN) toughness for hot-rolled shapes with flanges 1.5 inches (38.1 mm) thick or thicker must conform to Section A3 of AISC 341-16.

3.3.2 Plates: Plates used for YLBC Fuse Plates are ¾-inch (19 mm) thick and must conform to ASTM A572 Grade 50 with a minimum average value of 50 ft-lb absorbed energy from Charpy V-Notch Impact Test results at temperature of +70°F (+21°C). The maximum variation in yield strength, Fy, from mill certification reports or coupon tests of the Fuse Plates connected to each end of a single brace member shall be 10%.

Plates used for U-shims are 10 gage and must conform to ASTM A1011 Grade 33. Plates used for cover plates are ¾-inch (19 mm) thick and must conform to ASTM A572 Grade 50.

Plates used for gussets, slotted flange plates connected to brace flanges, filler shims, or other connection material must conform to ASTM A36 or ASTM A572 Grade 50. When the YLBC system is designed and detailed in accordance with AISC 341-16 (R=8), CVN toughness of plates 2 inches (50 mm) thick or thicker must conform to Section A3 of AISC 341-16.

3.3.3 Bolts, Threaded Rods, Washers and Nuts: Structural bolts must comply with ASTM F3125 Gr. A325 Type 1 or ASTM F3125 Gr. A490 Type 1, twist-off type structural bolt assemblies must comply with ASTM F3125 Gr. F1852 or ASTM F3125 Gr. F2280, and threaded rods must comply with ASTM A449 or ASTM A354 Gr. BD. Heavy-hex carbon-steel nuts must comply with ASTM A563 Grade C or DH and hardened carbon-steel washers must comply with ASTM F436 Type 1. Direct Tension Indicators must comply with ASTM F959 Type 325-1 or ASTM F959 Type 490-1. All bolts, threaded rods, washers and nuts must have a plain finish or must have a hot-dip zinc coating where indicated to be galvanized.

3.3.4 Welds: All welds must comply with the requirements of AISC 341-16 and must be performed using minimum E70xx electrodes.

4.0 DESIGN AND INSTALLATION

4.1 Design:

4.1.1 General: The seismic design coefficients and factors of the YLBC system for use with provisions of the ASCE 7-16 under the IBC are consistent with those of a Buckling-Restrained Braced Frame (BRBF) system and are as follows:

Response Modification Coefficient	R	=	8
Overstrength Factor	Ω0	=	2½
Deflection Amplification Factor	Cd	=	5

The component capacity modification factors for linear analysis procedures of the YLBC system for use with provisions of ASCE 41-17 under the IEBC are consistent with those of a Buckling-Restrained Braced Frame (BRBF) system for braces subject to an axial force and are as follows for the given performance levels:

Immediate occupancy (IO) m = 2.3

Primary Component

Life safety (LS) m = 5.6

Collapse prevention (CP) m = 7.5

Secondary Component

Life safety (LS) m = 7.5

Collapse prevention (CP) m = 10

Unless otherwise noted in this evaluation report, the parameters, factors and structural system limitations for the YLBC system must be consistent with those of the BRBF system noted in Table 12.2-1 of ASCE 7-16 and in accordance with the IBC and IEBC, as applicable.

Required strength, determination of seismic design category (SDC), risk category, and other design requirements must be performed in accordance with the provisions of the IBC and IEBC. This includes the drift and deformation requirements in ASCE 7-16 Section 12.12 and Table 12.12-1.

The response modification coefficient, R, designated for “steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in Table 12.2-1 of ASCE 7-16 shall be permitted for the YLBC system designed and detailed in accordance with AISC 360-16 and need not be detailed in accordance with AISC 341-16.

4.1.2 Members Design: The required strength of columns, beams, braces, and connections in the YLBC steel braced frame are to be determined from the load combinations of the IBC and IEBC. Seismic load effects must be included as defined in Section 12.4 of ASCE 7-16.

When the YLBC system is designed and detailed in accordance with AISC 341-16 (R=8), the columns, beams, struts, braces and connections must conform to the following:

1. The required strength of columns, beams, struts, gusset-to-column, gusset-to-beam, and gusset-to-base plate connections in the YLBC frame must be determined using the capacity-limited horizontal seismic load effect. The capacity-limited horizontal seismic load effect, E_{cl}, should be taken as the forces developed in the member assuming the forces in all Fuse Configurations correspond to their adjusted fuse configuration strength as defined in Section 4.1.4.3. Analysis must consider both directions of frame loading and braces must be determined to be in compression or tension neglecting the effects of gravity loads. Exceptions shall be in accordance with the exceptions of Section F4.3 of AISC 341-16.

2. The required strength of braces must be the greater of the maximum load determined from the horizontal seismic load effect including overstrength and the capacity-limited horizontal seismic load effect.

3. Columns, beams, and braces must comply with the requirements for moderately ductile members as defined in Section D1.1 of AISC 341-16.

4. Columns must be designed in accordance with the strength requirements of Section D1.4 of AISC 341-16.

5. Beams used in V- and inverted V- braced configurations must comply with the requirements of Section F.4.4a of AISC 341-16.

6. Unless a rational analysis demonstrates otherwise, braces in X-braced configurations shall have one continuous brace and one discontinuous cross brace which starts and stops at the continuous brace. The discontinuous brace must have axially rigid and rotational unrestrained connections to the top and bottom flange of the continuous brace. The continuous brace must be designed for the required axial strength in compression considering the unbraced length measured between work points of beam/column intersections at each of the brace. The continuous brace must have sufficient out-of-plane strength and stiffness to stabilize the discontinuous braces in accordance with Section 6.2.1 of AISC 360 Appendix 6 in combination with the required axial strength in tension. Discontinuous brace member sizes must match the member size of the opposing continuous brace.

4.1.3 Connection Design: Connections must be designed to resist the forces calculated in accordance with Section 4.1.2. The connection of brace-to-gusset plates using Fuse Plates must be in accordance with the connection details of Figures 3 and 4.

YLBC system brace splices must be located 1 ft or more away from the end of the Fuse Plates at the brace-to-Fuse Plate connection as shown in Figure 3. The brace splice must not occur in the middle third of the brace member. The required strength of brace splices must be the available flexural strength of the brace member taken about each axis. Splicing of braces is not permitted without prior approval of the registered design professional.

When the YLBC system is designed and detailed in accordance with AISC 341-16 (R=8), the connections must conform to the following:

1. Column splices must be designed in accordance with the requirements of Section D2.5 and Section F4.6d of AISC 341-16.
2. Column bases must be designed in accordance with the requirements of Section D2.6 of AISC 341-16.
3. Welds must be considered demand critical in accordance with Section F4.6a of AISC 341-16.
4. Where a gusset plate connects to both members at a beam-to-column connection, the connection must be designed in accordance with the requirements of Section F4.6b of AISC 341-16.

4.1.4 Fuse Configuration Design: The amplified deformation in an individual Fuse Configuration corresponding to the design story drift, $D_{a,fuse}$, is,

$\Delta_{a,fuse} = C_d P_u / k_{e,fuse}$ (EQ 1)

where

C_d = Deflection Amplification Factor as defined in Section 4.1.1

Pu = required strength of the Fuse Configuration as determined from the Load and Resistance Factor Design (LRFD) of the IBC

$Ke,fuse$ = elastic axial stiffness of the Fuse Configuration determined from testing provided in Table 1, kips/inch (kN/m).

The expected deformation in an individual Fuse Configuration corresponding to two times the design story drift must not exceed the design deformation capacity, $\Delta_{max,fuse}$, where,

$\Delta_{max,fuse}$ = 1.5 inches (38 mm)

Where Fuse Configurations are located at both ends of a single brace member according to Section 4.1.4.1, the total deformation within the assembly attributed to the Fuse Configurations may be assumed to be shared equally between the Fuse Configurations at each end of the brace.

The expected deformation in a brace and Fuse Configuration assembly are those corresponding to two times the story drift in addition to brace deformations resulting from deformation of the frame due to gravity loading.

The stiffness of the brace and Fuse Configuration assembly in the YLBC system must be determined in accordance with Section 4.1.5.

4.1.4.1 Fuse Configuration Locations: Where structures are assigned to SDC A, B or C, Fuse Configurations need only be located at one end of a single brace member. Where structures are assigned to SDC D, E or F, Fuse Configurations must be located at each end of a brace member unless one of the following two conditions is satisfied, in which case Fuse Configurations need only be located at one end of the brace at the story considered.

(1) The maximum calculated deformation of an individual Fuse Configuration associated with a story drift equal to 2% of the story height is less than the design deformation capacity, $\Delta_{max,fuse}$, as defined in Section 4.1.4.

(2) The maximum deformation of an individual Fuse Configuration as determined from a nonlinear analysis in accordance with Section C3 of AISC 341-16 is less than the design deformation capacity, $\Delta_{max,fuse}$, as defined in Section 4.1.4.

4.1.4.2 Available Fuse Configuration Strength: The available Fuse Configuration strength, ΦP_n, for each predefined Fuse Configuration is tabulated in Table 1. The available Fuse Configuration strength must exceed the required strength as determined from the Load and Resistance Factor Design (LRFD) of the IBC. Effect of horizontal seismic forces must be included as defined in Section 12.4.2.1 of ASCE 7-16.

4.1.4.3 Adjusted Fuse Configuration Strength: The adjusted Fuse Configuration strengths in compression and tension, P_{prT} or P_{prC}, are:

$$P_{prT} = R_{y,fuse}(P_n + k_{iT}\Delta_{fuse}) \quad \text{(EQ 2a)}$$

$$P_{prC} = R_{y,fuse}(P_n + k_{iC}\Delta_{fuse}) \quad \text{(EQ 2b)}$$

where

$R_{y,fuse}$ = ratio of expected yield stress to minimum specified yield stress of the Fuse Plates taken as 1.1 (See Section 3.3.2 for Fuse Plate material)

P_n = nominal strength of a Fuse Configuration at the minimum specified yield stress determined from testing provided in Table 1, kips (kN)

k_{iT} = inelastic tensile stiffness factor for a Fuse Configuration determined from testing provided in Table 1, kips/inch (kN/m)

k_{iC} = inelastic compressive stiffness factor for a Fuse Configuration determined from testing provided in Table 1, kips/inch (kN/m)

Δ_{fuse} = expected deformation of an individual Fuse Configuration as defined in Section 4.1.4, in (mm).

Alternatively, for any Fuse Configuration regardless of the calculated design deformation, the adjusted Fuse Configuration strength used in calculations of the columns, beams, braces, and connections may conservatively be taken as the governing maximum expected strength at the design deformation capacity in compression or tension, P_{prC_max} or P_{prT_max}, as tabulated in Table 1 and determined as:

$$P_{prC\_max} = R_{y,fuse}P_C \quad \text{(EQ 3a)}$$

$$P_{prT\_max} = R_{y,fuse}P_T \quad \text{(EQ 3b)}$$

where

P_C = anticipated compression strength of the Fuse Configuration at the design deformation capacity taken as $P_n + k_{iC}\Delta_{max,fuse}$ as provided in Table 1, kips (kN)

P_T = anticipated tensile strength of the Fuse Configuration at the design deformation capacity taken as $P_n + k_{iT}\Delta_{max,fuse}$ as provided in Table 1, kips (kN)

4.1.5 Brace and Fuse Configuration Assembly Stiffness: The stiffness of the brace and Fuse Configuration assemblies may be represented directly in analysis by applying $k_{e,fuse}$ from Table 1 to one or both ends of brace members as appropriate. Alternatively, the effective elastic axial stiffness of assemblies, $k_{e,eff}$, may be approximated as:

$k_{e,eff}$ = $1/(1/k_{e,brace}+n_{fuse}/k_{e,fuse})$ (EQ 4)

where

$k_{e,fuse}$ = the elastic axial stiffness of the Fuse Configuration determined from testing provided in Table 1, kips/inch (kN/m)

$k_{e,brace}$ = approximate axial stiffness of the brace less the effective length of the Fuse Configurations $A_{brace}E_s/(L_{wp} - n_{fuse}L_{fuse})$, kips/inch (kN/m)

n_{fuse} = number of specified Fuse Configurations in the brace and Fuse Configuration Assembly (n_{fuse} = 1 for Fuse Configuration at one end of brace, n_{fuse} = 2 for Fuse Configuration at each end of brace)

A_{brace} = gross cross-sectional area of the brace member, inches^2 (m^2)

E_s = the elastic modulus of elasticity of steel taken as 29,000 ksi (200 GPa)

L_{wp} = work point-to-work point dimension along the brace and Fuse Configuration, inches (m)

L_{fuse} = approximate effective length of the yielding portion of a Fuse Plate taken as 12 inches (0.3 m)

4.2 Installation:

4.2.1 General: The YLBC system must be installed in accordance with the manufacturer's installation or replacement instructions as applicable, the IBC or IEBC, as applicable, this report, and the approved construction documents prepared by a registered design professional and approved by the authority having jurisdiction.

4.2.2 Joint Type: Class A slip-critical joints must be used at primary Fuse Plate connections. Welding is not permitted in Fuse Plate connections. The secondary YLBC8 Fuse Plate connections, within the protected zone, may be specified as snug-tight. When mechanical fasteners are used at the connection of slotted flange plates to brace flanges, the joint type may be specified as snug-tight.

4.2.3 Orientation of Yield-Link® Fuse Plates: Fuse Plates must be oriented with the end nearest the expected yielding region, as indicated by the identification tag described in Section 7.1, attached to the brace member such that the expected yielding region is concealed by Cover Plates in the final condition. The opposite end of the Fuse Plates must be attached to a gusset plate at the beam-to-column, brace-to-beam, and column-to-base plate connections.

Fuse Plates must be oriented concentric to the longitudinal axis of brace members as shown in Figure 3.

4.2.4 Protected Zone: The protected zone consists of the portion of the Fuse Plates bounded by the extents of the expected yielding region as illustrated in Figure 1. Welding and mechanical fastening, other than what is detailed in Figure 3, are not permitted within the protected zone.

4.3 Special Inspections:

Special inspections, testing and structural observations are required in accordance with Sections 1705.1.1, 1705.2, 1705.12, 1705.13 of the IBC, Chapter N of AISC 360-16 and Chapter J of AISC 341-16. The special inspector must verify, at a minimum, whether the components of seismic force resisting system (SFRS) and steel elements that are used for connections within the SFRS are recognized in ESR-4342 and comply with the approved construction documents.

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 authority having jurisdiction.

5.0 CONDITIONS OF USE:

The YLBC system 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 YLBC system design, including structural notes and details, must be in accordance with this report and the IBC or IEBC, as applicable, and must be prepared by a registered design professional and subjected to approval of the authority having jurisdiction. In the event of a conflict between this report and the IBC or IEBC, as applicable, the more restrictive requirement shall govern.

5.2 Structural design drawings and specifications should include items required by AISC 360-16, AISC 303-16, and the applicable building code. When the YLBC system is designed and detailed in accordance with AISC 341-16, the structural design drawings and specifications must comply with Section A4 of AISC 341-16.

5.3 Installations must be performed 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 authority having jurisdiction. In the event of a conflict between this report and the manufacturer's installation instructions, this report governs.

5.4 Galvanizing, cutting, and grinding of Fuse Plates are not permitted. Mechanical fastening and welding in the protected zone of Fuse Plates are not permitted as outlined in Section 4.2.4 of this report.

5.5 Special inspections must be in accordance with Section 4.3 of this report and the approved construction documents.

5.6 The Yield-Link® Brace Connection is manufactured under an approved quality control program with inspections by ICC-ES.

6.0 EVIDENCE SUBMITTED

Data in accordance with the ICC-ES Acceptance Criteria for Qualification of Building Seismic Performance of Alternative Seismic Force-Resisting Systems (ICC-ES Guidance Document to FEMA P695), Annex B, Steel Braced Frame System with Fuse Element Connectors (AC494), dated February 2022.

The acceptable values of Adjusted Collapse Margin Ratio (ACMR), that were used to evaluate conformance to collapse prevention objectives, of Section 7.5 of FEMA P695 were as follows:

For YLBC6 Fuse Plates with a total system collapse uncertainty of β_{TOT} = 0.60, $ACMR_{10\%} \geq 2.16$ and $ACMR_{20\%} \geq 1.66$.

For YLBC8 Fuse Plates with a total system collapse uncertainty of β_{TOT} = 0.53, $ACMR_{10\%} \geq 1.97$ and $ACMR_{20\%} \geq 1.56$.

7.0 IDENTIFICATION

7.1 The Yield-Link® Fuse Plate Part ID, barcode for shipping and material tracking are marked on each Fuse Plate at the brace member connection end. A label including the Quote number, Fuse Configuration mark, ICC-ES evaluation report number (ESR-4342), Simpson Strong-Tie® logo and website address for YLBC system patent number must be visible on the outside vertical face of each Cover Plate of each YLBC connection.

On each sheet of the structural drawing/shop detail drawing that contains technical information showing the YLBC connections, the following notice of intellectual property must be affixed before release for intended use: Yield-Link® Brace Connection system is protected under one or more the following patents and applications: US Patent Nos. 9,514,907, 10,544,577, 11,203,862 and 11,346,121.

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

SIMPSON STRONG-TIE COMPANY INC.
5956 W LAS POSITAS BOULEVARD
PLEASANTON, CALIFORNIA 94588
925-560-9000
www.strongtie.com

TABLE 1—YIELD-LINK® BRACE CONNECTION DESIGN PARAMETERS

Fuse Configuration Kit Name	ΦP_n, kips	P_n, kips	P_T, kips	P_C, kips	$P_{prT,max}$, kips	$P_{prC,max}$, kips	$k_{e,fuse}$, kips/inch	k_{iT}, kips/inch	k_{iC}, kips/inch	$A_{equiv,fuse}$[1], inches^2
YLBC6-30-1	30	33	50	50	55	55	415	11	11	0.12
YLBC6-60-2	60	67	100	100	110	110	830	22	22	0.34
YLBC6-90-3	90	100	150	150	165	165	1245	33	33	0.52
YLBC6-120-4	120	133	199	199	219	219	1660	44	44	0.69
YLBC6-150-5	150	167	249	249	275	275	2075	55	55	0.86
YLBC6-180-6	180	200	299	299	329	329	2490	66	66	1.03
YLBC6-210-7	210	233	349	349	384	384	2905	77	77	1.20
YLBC8-30-1	30	33	48	51	53	56	300	10	12	0.12
YLBC8-60-1	60	67	86	90	95	99	600	13	15.6	0.25
YLBC8-90-2	90	100	135	141	148	156	900	23	27.6	0.37
YLBC8-120-2	120	133	172	180	190	198	1200	26	31.2	0.50
YLBC8-150-3	150	167	221	232	243	255	1500	36	43.2	0.62
YLBC8-180-3	180	200	259	270	284	297	1800	39	46.8	0.74
YLBC8-210-4	210	233	307	322	337	354	2100	49	58.8	0.87
YLBC8-240-4	240	267	345	360	379	396	2400	52	62.4	0.99
YLBC8-270-5	270	300	393	412	432	453	2700	62	74.4	1.12
YLBC8-300-5	300	333	431	450	474	495	3000	65	78	1.24
YLBC8-330-6	330	367	479	502	527	552	3300	75	90.0	1.37
YLBC8-360-6	360	400	517	540	569	594	3600	78	93.6	1.49
YLBC8-390-7	390	433	565	592	622	651	3900	88	105.6	1.61
YLBC8-420-7	420	467	603	631	664	694	4200	91	109.2	1.74

For **SI**: 1 inch = 0.0254 m, 1 kip = 4.45 kN, 1 kip/in. = 174.3 kN/m, 1 inch^2 = 0.00064 mm^2

[1] Area of a 12-inch length of section producing axial stiffness equivalent to $k_{e\ fuse}$.

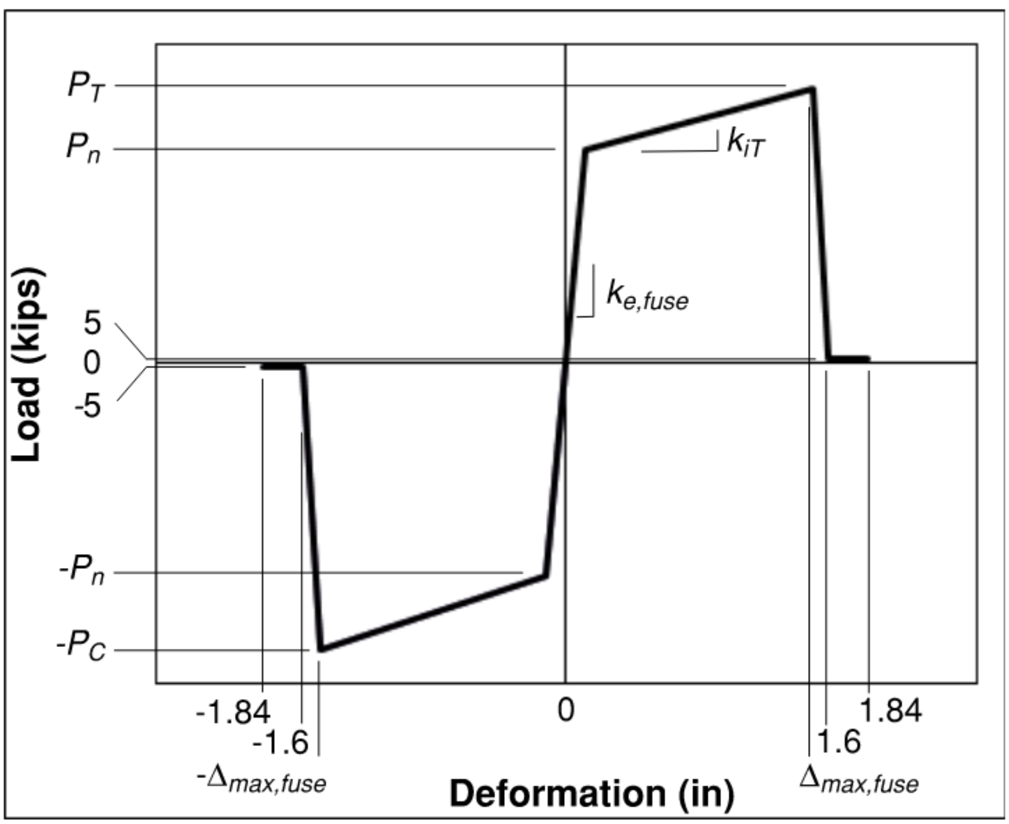
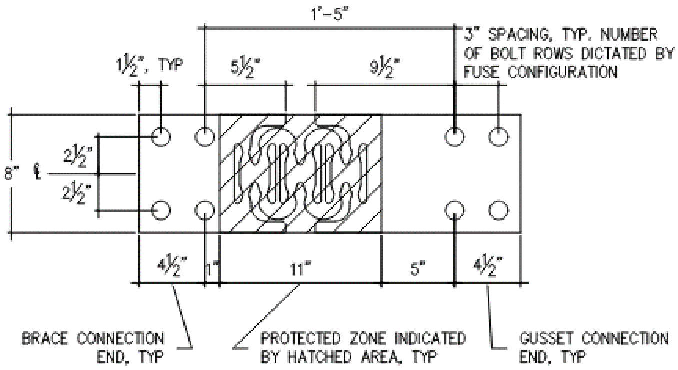
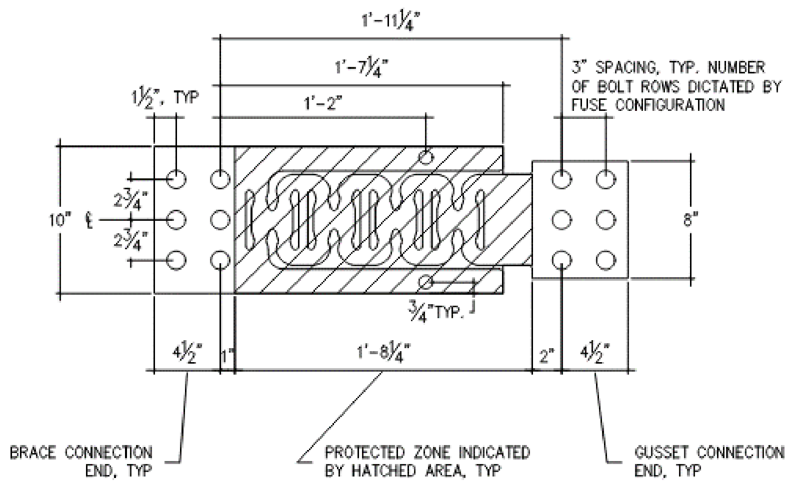


FIGURE 1 – YIELD LINK® BRACE CONNECTION FUSE CONFIGURATION BACKBONE CURVE



YLBC6



YLBC8

FIGURE 2 - YIELD LINK® BRACE CONNECTION FUSE PLATES

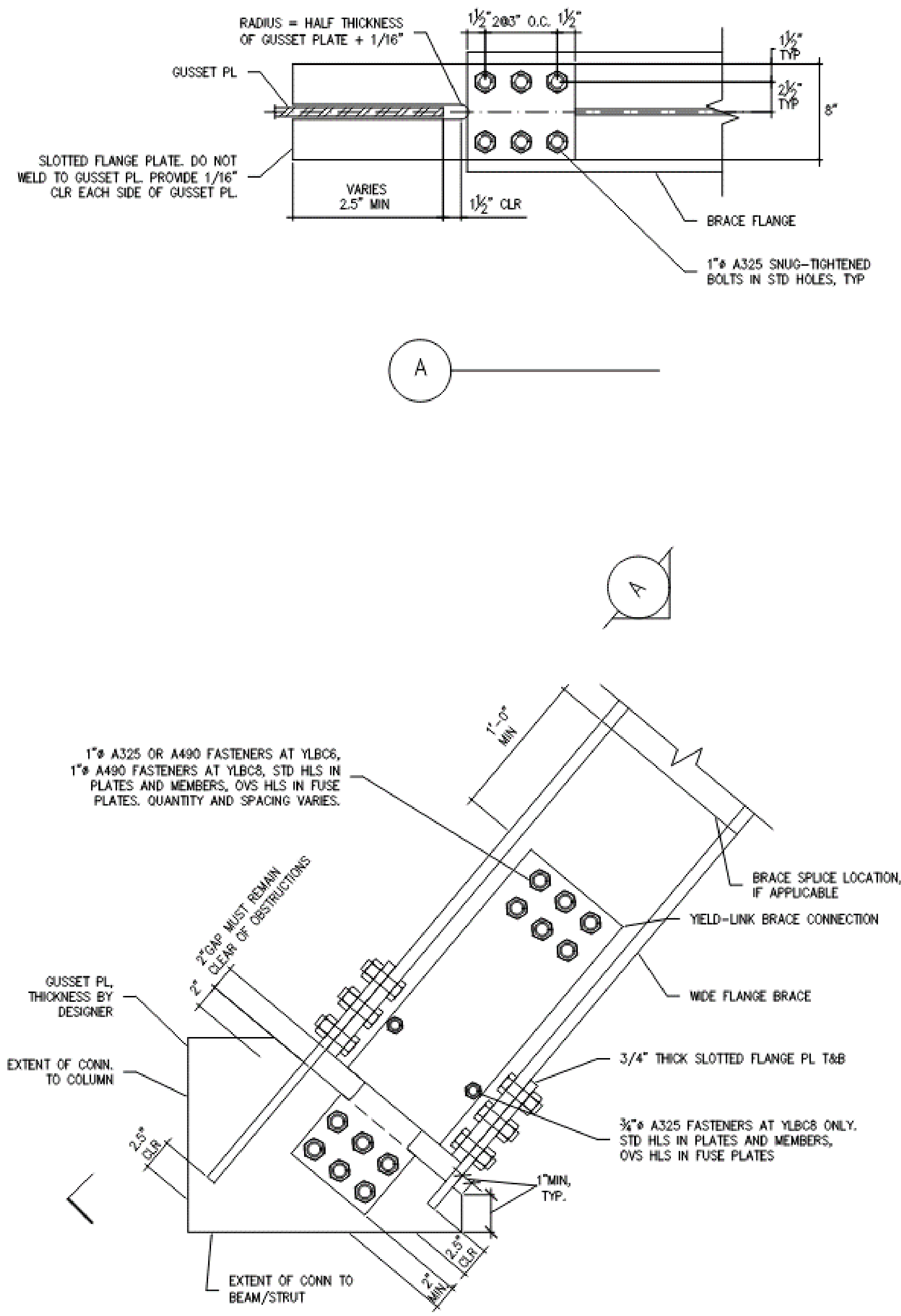
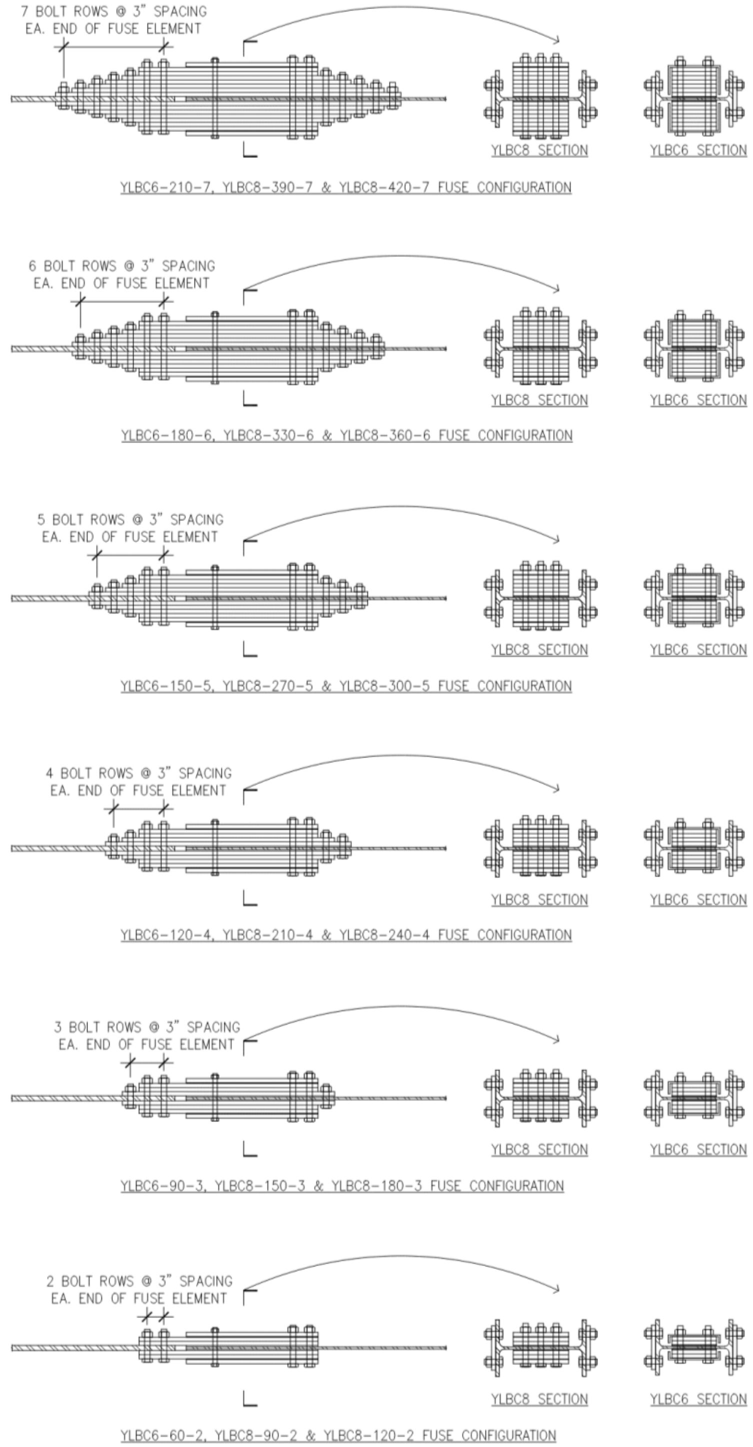
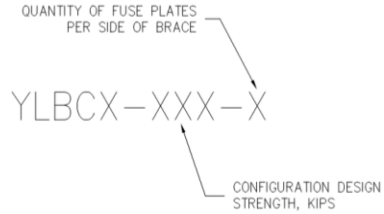


FIGURE 3 - YIELD-LINK® BRACE CONNECTION GENERAL PARAMETERS





YLBC8 FUSE CONFIGURATION TABLE	
FUSE CONFIGURATION NAME	QUANTITY OF FUSE PLATES PER SIDE
YLBC8-420-7	(2) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5 (1) YLBC8-F6 (1) YLBC8-F7
YLBC8-390-7	(1) YLBC8-F1 (1) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5 (1) YLBC8-F6 (1) YLBC8-F7
YLBC8-360-6	(2) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5 (1) YLBC8-F6
YLBC8-330-6	(1) YLBC8-F1 (1) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5 (1) YLBC8-F6
YLBC8-300-5	(2) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5
YLBC8-270-5	(1) YLBC8-F1 (1) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4 (1) YLBC8-F5
YLBC8-240-4	(2) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4
YLBC8-210-4	(1) YLBC8-F1 (1) YLBC8-F2 (1) YLBC8-F3 (1) YLBC8-F4
YLBC8-180-3	(2) YLBC8-F2 (1) YLBC8-F3
YLBC8-150-3	(1) YLBC8-F1 (1) YLBC8-F2 (1) YLBC8-F3
YLBC8-120-2	(2) YLBC8-F2
YLBC8-90-2	(1) YLBC8-F1 (1) YLBC8-F2
YLBC8-60-1	(1) YLBC8-F2
YLBC8-30-1	(1) YLBC8-F1

YLBC6 FUSE CONFIGURATION TABLE	
FUSE CONFIGURATION NAME	QUANTITY OF FUSE PLATES PER SIDE
YLBC6-210-7	(2) YLBC6-yl1 (1) YLBC6-yl2 (1) YLBC6-yl3 (1) YLBC6-yl4 (1) YLBC6-yl5 (1) YLBC6-yl6
YLBC6-180-6	(2) YLBC6-yl1 (1) YLBC6-yl2 (1) YLBC6-yl3 (1) YLBC6-yl4 (1) YLBC6-yl5
YLBC6-150-5	(2) YLBC6-yl1 (1) YLBC6-yl2 (1) YLBC6-yl3 (1) YLBC6-F4
YLBC6-120-4	(2) YLBC6-yl1 (1) YLBC6-yl2 (1) YLBC6-yl3
YLBC6-90-3	(2) YLBC6-yl1 (1) YLBC6-yl2
YLBC6-60-2	(2) YLBC6-yl1
YLBC6-30-1	(1) YLBC6-yl1

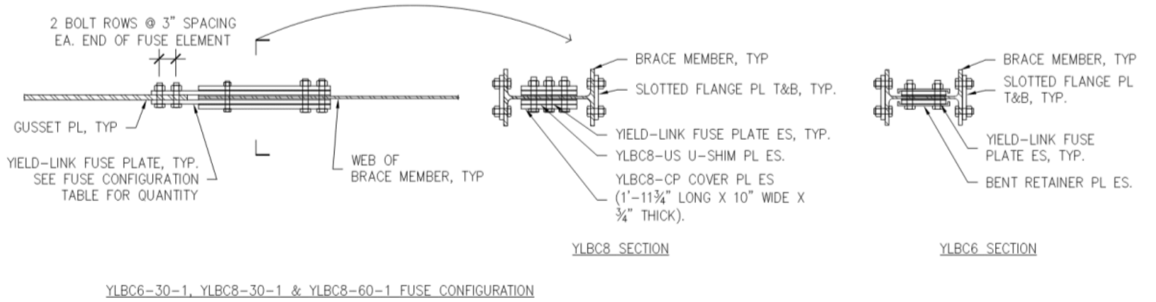


FIGURE 4 - YIELD-LINK® BRACE CONNECTION FUSE CONFIGURATIONS

Annex A: Design Example

Using the Equivalent Lateral Force Procedure of ASCE 7-16 with a Response Modification Coefficient, *R*, of 8, preliminary analysis of a typical five-story building located in Seismic Design Category (SDC) D results in the forces shown in Figure A-1 under the critical load combination including earthquake loads. The brace and drag beam forces shown in the figure are the effects of the horizontal seismic forces, Q_E. The column and governing beam forces shown in the figure are determined from the load combinations of the 2018 IBC considering vertical seismic load effects, E_v. It is assumed the critical load combination including earthquake loads governs the design over the load combinations with wind.

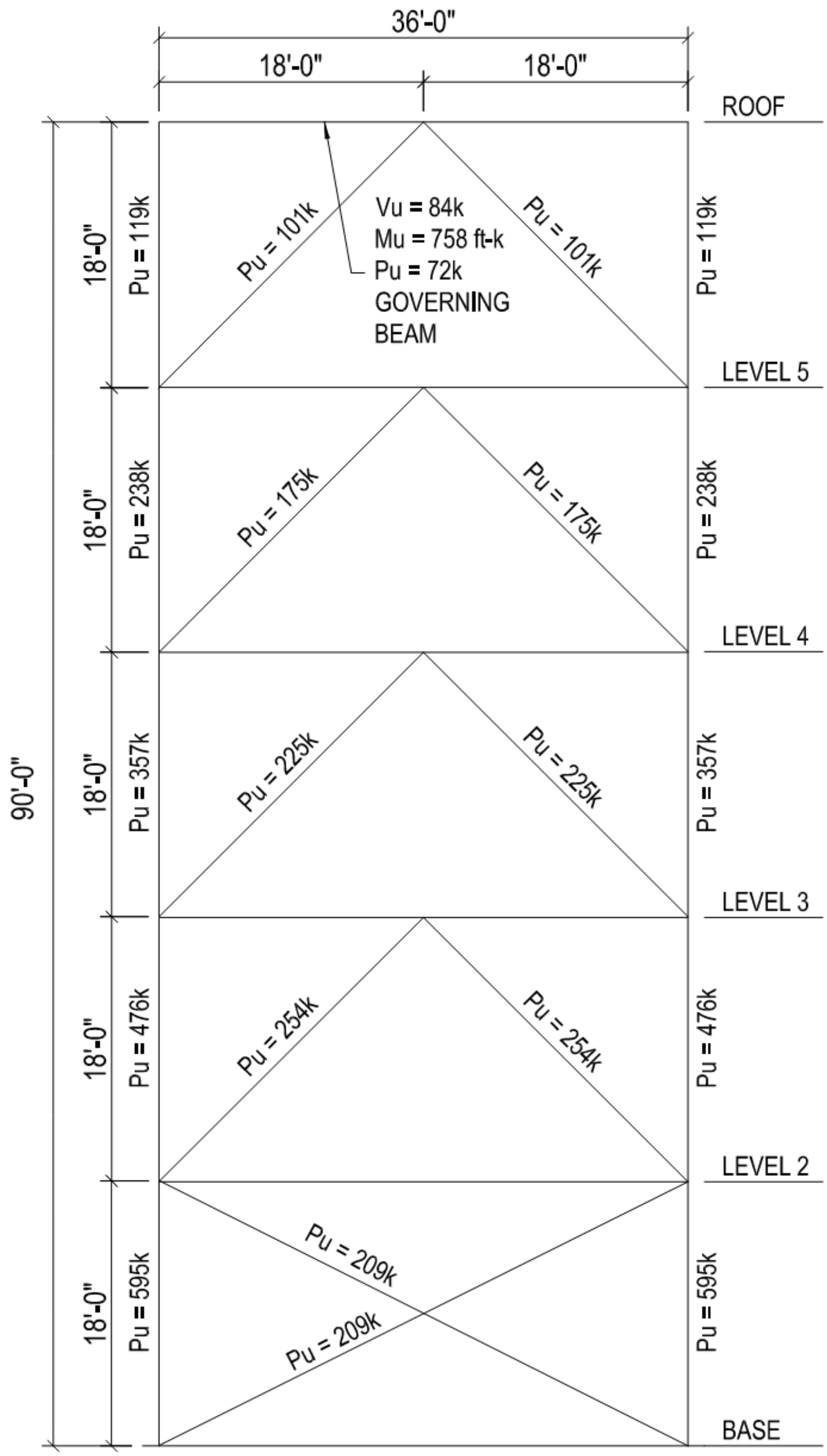


FIGURE A-1 – EXAMPLE BRACED FRAME CONFIGURATION AND FRAME FORCES

Step 1 – Fuse Configuration Selection and Location

Step 1.1 – Fuse Configuration Selection: The ultimate forces in each brace resulting from the preliminary analysis are summarized in Table A-1. A Fuse Configuration is selected for each story such that the capacity, ϕP_n, taken from Table 1 meets or exceeds the ultimate force in the brace member from the analysis.

Step 1.2 – Fuse Configuration Location: Because the building is located in SDC D, Fuse Configurations are required at each of the braces unless one of the conditions outlined in Section 4.1.4.1 is satisfied. It is assumed that nonlinear analysis will not be performed in accordance with the second condition therefore a check of the first condition is illustrated below for the braces in inverted V-braced configurations. Because inelastic deformation is substantially isolated to the Fuse Plates, it is reasonable and conservative to assume that all deformation occurs in the Fuse Configurations.

The work point-to-work point length of each brace, L_{br}, is determined from the frame geometry as:

$$L_{br} = \sqrt{H^2 + L^2} = \sqrt{(18\ ft)^2 + (18\ ft)^2} = 25.5 \text{ ft}$$

The expected deformation in the Brace Assembly corresponding to a story drift of 2% the story height is compared to the Fuse design deformation capacity below:

$$\Delta_{fuse} \approx 0.02H(L/L_{br}) = 0.02(18 \text{ ft})((18 \text{ ft})/(25.5 \text{ ft}))(12 \text{ in./ft}) = 3.05 \text{ in.} > \Delta_{max,fuse} = 1.5 \text{ in.}$$

Because Δ_{fuse} is larger than the fuse design deformation capacity, $\Delta_{max,fuse}$, the first condition of Section 4.1.4.1 is not satisfied and Fuse Configurations are required at each end of the braces as indicated in Table A-1.

TABLE A-1 - FUSE CONFIGURATION SELECTION SUMMARY

Story	Brace Force P_u, kips	Fuse Configuration	Fuse Capacity ϕP_n, kips
5	101	YLBC8-120 each end	120
4	175	YLBC8-180 each end	180
3	225	YLBC8-240 each end	240
2	254	YLBC8-270 each end	270
1	209	YLBC8-210 each end	210

Step 2 – Brace Design

The required strength of the brace, P_{ubr}, is the brace force determined from the preliminary analysis multiplied by the overstrength factor, Ω_0, but not less than the adjusted Fuse Configuration strength determined from EQ 2. For this example, the adjusted Fuse Configuration strength is conservatively taken as $P_{prC,max}$ from Table 1 and determined from EQ 3a. For the brace at level 2 in the inverted V-braced configuration, the required strength is determined as follows:

$P_{u\text{-}2} = P_{QE}\Omega_0 = (254 \text{ kips})(2.5) = 635 \text{ kips} \geq P_{prC,max} = 453 \text{ kips}$, therefore $P_{ubr} = 635$ kips

As indicated in Step 1, the work point-to-work point length of each brace, L_{br}, is 25.5 ft. Using Table 4-1a of the AISC *Steel Construction Manual, 15th Edition* for an unbraced length about the weak axis of 26 ft, a W14x109 (moderately ductile diagonal brace per AISC 341-16) is selected for which $\phi P_n = 863$ kips. Alternatively, a more precise capacity may be determined by applying the equations of AISC 360 using the actual workpoint-to-workpoint length.

For the braces at level 1 in the X-braced configuration, the design requirements of Section 4.1.2.6 are considered. The required strength of the continuous brace in compression is determined as follows:

$P_{u\text{-}1} = P_{QE}\Omega_0 = (209 \text{ kips})(2.5) = 523 \text{ kips} \geq P_{prC,max} = 354 \text{ kips}$, therefore $P_{ubr} = 523$ kips

The workpoint-to-workpoint length, L_{br}, is 40.2 ft. Selecting a W14x132 section, the slenderness ratio of the brace about the weak axis is determined as:

$KL/r_y = 1.0(40.2 \text{ ft}((12\text{in./ft})/3.76 \text{ in.} = 128.3$

Utilizing Table 4-14 of the AISC *Steel Construction Manual, 15th Edition*, the critical compression stress is interpolated as $\phi F_{cr} = 13.7$ ksi. The compressive strength considering buckling about the weak axis is then determined as:

$\phi P_n = \phi F_{cr} A_g = (13.7 \text{ ksi})(38.8 \text{ in.}^2) = 533 \text{ kips} \geq 523 \text{ kips}$, therefore OK

In accordance with the panel bracing requirement for columns in Appendix 6 of AISC 360-16, the out-of-plane force applied to the midpoint of the continuous brace by each discontinuous brace is determined as:

B_{br} = 0.005P_u = 0.005(523 kips) = 2.62 kips

The resulting moment applied to the continuous brace is determined as:

$M_u = 2B_{br}L_{br}/4$ = 2(2.62 kips)(40.2 ft)/4 = 52.7 k-ft

Utilizing Table 6-2 of the AISC *Steel Construction Manual, 15th Edition,* the tensile strength and weak axis flexural strength is ϕT_n = 1,750 kips and ϕM_n = 424 k-ft respectively. The tension and flexural interaction at the midpoint of the brace is determined as:

$P_{prT,max}/\phi T_n$ = (337 kips)/(1,750 kips) = 0.19

$P_{prT,max}/\phi T_n + 8M_u/(9\ \phi M_n)$ = 0.19 + 8(52.7 k-ft)/(9(424 k-ft) = 0.303 ≤ 1.0, therefore OK

The tensile rupture strength at the end of the brace conservatively taken from AISC Table 6-2 assuming a net effective area of 0.75A_g is ϕT_n = 1,420 kips ≥ 337 kips, therefore the section is adequate.

The required out-of-plane stiffness of the continuous brace in accordance with Appendix 6 of AISC 360 is determined as:

$\beta_{br} = 2P_u/(\phi L_{br,discont})$ = 2(523 kips)/(0.75*(40.2 ft)(12in./ft)/2) = 5.8 k/in.

The out-of-plane flexural stiffness of the continuous brace under a concentrated load the midpoint is determined as:

$\beta_{act} = 48E_sI_y/L_{br}^3$ = 48(29,000 ksi)(548 in.^{4})/(40.2 ft(12in./ft))^3 = 6.8 k/in. ≥ 5.8 k/in., therefore OK

A W14x132 section is selected for the discontinuous brace segment also to closely match the axial stiffness of the continuous brace. Because the unbraced length of the discontinuous segments, $L_{br,discont}$, is half that of the continuous brace, the W14x132 is adequate by inspection for the required brace forces in the tension and compression.

The brace design for each story is summarized in Table A-2 wherein the brace capacities shown are calculated using AISC 360 for the actual workpoint-to-workpoint lengths.

TABLE A-2 - BRACE DESIGN SUMMARY

Level/ Tier	Amplified Fuse Configuration Strength $\Omega_0 P_{QE}$, kips	Adjusted Fuse Configuration Strength $P_{pr,max}$, kips	Required Brace Strength P_{ubr}, kips	Brace Section	Brace Design Strength ϕP_n, kips
5	252	198	252	W14x61	259
4	438	297	438	W14x109	883
3	564	396	564	W14x109	883
2	635	453	635	W14x109	883
1	523	354	523	W14x132	533

Step 3 – Beam Design

The maximum required strengths for the beam are the shear, moment, and axial forces determined from load combination 6 of ASCE 7-16 Section 2.3.6 given as $1.2D + E_v + E_{mh} + L + 0.2S$. Applying Section 12.4.2.2 for the definition of E_v and Section 12.4.3.1 for the definition of E_{mh}, the combination becomes $(1.2 + 0.2S_{DS})D + \Omega_0 Q_E + L + 0.2S$.

Taking the uniformly distributed load along the beam attributed to dead, live, snow, and vertical seismic loads $[(1.2 + 0.2S_{DS})D + L + 0.2S]$ as $w_u = 1.5$ klf, required shear and moment strength of the beams at the floor levels are as follows:

$V_{u\text{-}b}$ = (4.68 klf)(36 ft)/2 = 84.2 kips

$M_{u\text{-}b}$ = (4.68 klf)(36 ft)^2/8 = 758 k-ft

Investigating the beam at level 5, a W24x103 with flanges meeting the requirements for moderately ductile beam members per AISC 341-16 is selected preliminarily based on strength and stiffness requirements from gravity loading. The force delivered to the frame by the diaphragm at level 5 is assumed to be 143 kips and is assumed to be applied evenly through collector connections at each end of the beam. The seismic axial load including overstrength used for the design of the beam and beam connection, as well as collector beams and collector connections in the LFRS outside the YLBC frame, is determined as:

$$P_{u\text{-}5} = \Omega_0 Q_E = 2.5(143\text{ kips})/2 = 179\text{ kips}$$

Investigating the beam at level 3, the capacity-limited maximum axial force in the inverted V-braced configuration is the horizontal component delivered by the second story braces, which is determined as:

$$P_{u\text{-}2} = P_{pr,max1}(L/L_{br}) = (453\text{ kips})(18\text{ ft})/(25.5\text{ ft}) = 320\text{ kips (governs over level 5)}$$

In accordance with Table 1-3 of the AISC *Seismic Design Manual, 3rd Edition*, the axial force limit for the web of a W24x103 to meet the requirements for moderately ductile members is $P_{u\,max} = 1{,}190$ kips. This is greater than maximum applied axial force of 320 kips, therefore the web of the section meets the requirement for moderately ductile members.

The slenderness ratio of the beam about the strong axis is determined as:

$$KL/r_x = 1.0(18\text{ ft})(12\text{in./ft})/10\text{ in.} = 21.6$$

Table 1-3 of AISC 341-16 provides the maximum unbraced length to meet moderately ductile requirements, $L_{b\,max} = 16.6$ ft. Because bracing is required to be present at the brace intersection, it is assumed bracing will be provided at ¼ points along the beam to meet the maximum unbraced length requirement. The resulting unbraced length is 9 ft.

The slenderness ratio of the beam about the weak axis is determined as:

$$KL/r_y = 1.0(9\text{ ft})(12\text{in./ft})/1.99\text{ in.} = 54.3\text{ (governs)}$$

Utilizing Table 6-2 of the AISC *Steel Construction Manual, 15th Edition*, the compressive strength and flexural strength with an unbraced length of 9 ft is $\phi P_n = 1{,}100$ kips and $\phi M_n = 996$ k-ft respectively. The compression and flexural interaction is determined as:

$$P_u/\phi P_n = (320\text{ kips})/(1{,}100\text{ kips}) = 0.29$$

$$P_u/\phi P_n + 8M_u/(9\phi M_n) = 0.29 + 8(758\text{ k-ft})/(9(996\text{ k-ft})) = 0.97 \leq 1.0\text{, therefore OK}$$

The shear strength taken from AISC Table 6-2 is $\phi V_n = 404$ kips ≥ 84 kips, therefore the section is adequate.

A W24x103 beam is conservatively chosen for all stories assuming an enveloped design approach using the governing forces for the beam at level 3.

Step 4 – Fuse Deformation Capacity

The expected deformation in a brace and Fuse Configuration assembly are those corresponding to a story drift of two times the design story drift determined in accordance with ASCE 7-16, in addition to brace deformations resulting from deformation of the frame due to gravity loading. The expected deformation of a single Fuse Configuration, assuming the deformation is shared equally between the Fuse Configurations at each end of the brace in accordance with Section 4.1.4, is calculated using the following steps. All deformation is conservatively assumed inelastic and therefore attributed to the Fuse Configurations.

Step 4a – Fuse Deformation corresponding to two times the story drift: Using EQ1, the expected inelastic deformation in an individual Fuse Configuration at level 2 corresponding to two times the design story drift is determined as:

$$\Delta_{2a,fuse} = 2C_d P_u/(k_{e,fuse}(2\text{ Fuse Configurations})) = 2(5)(254\text{ kips})/(2{,}700\text{ k/in.}(2)) = 0.47\text{ in.}$$

Step 4b – Fuse Deformation corresponding to Frame Deformation: The uniformly distributed load along the beam attributed to dead, live, snow, and vertical seismic loads $[(1.2 + 0.2S_{DS})D + L + 0.2S]$ is $w_u = 1.5$ klf. The deflection at middle of the beam, $\Delta_{b,mid}$, is determined as:

$$\Delta_{b,mid} = 5w_uL^4/(384EI_x) = 5(4.68\text{ klf})(36\text{ ft})^4(1728)/(384(29{,}000\text{ ksi})(3000\text{ in.}^4)) = 2.03\text{ in.}$$

The axial deformation of the individual Fuse Configurations, $\Delta_{fuse,beam}$, due to the deflection of the beam once the assembly enters into the inelastic range assuming equal distribution between Fuse Configurations is determined as:

$$\Delta_{fuse,beam} = \left(L_{br} - \sqrt{(H - \Delta_{b,mid})^2 + L^2}\right)/2 = \left(305.47\,in. - \sqrt{(216\,in. - 2.03\,in.)^2 + (216\,in.)^2}\right)/2 = 0.72\text{ in.}$$

Step 4c – Determine Maximum Fuse Deformation: The total maximum expected deformation of an individual fuse, $\Delta_{fuse,total}$, is determined as:

$$\Delta_{fuse,total} = (\Delta_{2a,fuse} + \Delta_{fuse,beam}) = 0.47 \text{ in.} + 0.72 \text{ in.} = 1.19 \text{ in.} \leq \Delta_{max,fuse} = 1.5 \text{ in., therefore OK}$$

Step 5 – Column Design

The maximum required compressive strength of the column, P_{uc}, is determined from load combination 6 of ASCE 7-16 Section 2.3.6 given as $1.2D + E_v + E_{mh} + L + 0.2S$. Applying Section 12.4.2.2 for the definition of E_v and Section 12.4.3.2 for substitution of E_{cl} for E_{mh} for capacity-limited design, the combination becomes $(1.2 + 0.2S_{DS})D + E_{cl} + L + 0.2S$. The axial forces in each column tier attributed to dead, live, snow, and vertical seismic loads, P_u, are shown in Figure A-1 and provided in Table A-3. The axial force in the column tier under consideration associated with the adjusted fuse configuration force in all brace assemblies framing into and above the upper end of that tier, *Ecl*, is determined below.

The adjusted fuse strengths in tension and compression can be determined using EQ 2a and EQ 2b respectively by assuming half the amplified axial deformation of the brace assembly at the design story drift as illustrated Step 7 is distributed to the fuse configuration at each end of a given brace in accordance with Section 4.1.4.3.

Alternatively, the adjusted fuse strength may conservatively be taken as $P_{pr,max}$ as tabulated in Table 1. While this method is more conservative, using $P_{pr,max}$ does not require calculation of the design story drift to determine the force to the columns. The force in the column tier at level 1, E_{cl1}, associated with the adjusted brace force taken as $P_{prC,max}$ for all braces above the column tier is:

$$E_{c11} = = P_{prC,max1}H/L_{br1} + (P_{pr,max2} + P_{pr,max3} + P_{pr,max4} + P_{pr,max5})(H/L_{br2}) = 155 + 320 \text{ kips} + 280 \text{ kips} + 210 \text{ kips} + 140 \text{ kips} = 1{,}105 \text{ kips}$$

The required column strength is determined by combining the force associated with the adjusted brace force with the axial forces due to dead, live, snow, and vertical seismic loads at the column tier at level 1 is:

$$P_{uc1} = P_{u1} + E_{cl1} = 595 \text{ kips} + 1{,}105 \text{ kips} = 1{,}700 \text{ kips}$$

Using Table 4-1a of the AISC *Steel Construction Manual, 15th Edition* with an unbraced length about the weak axis of 18 ft, a W14x159 (moderately ductile column section per AISC 341-16) is selected for which $\phi P_n = 1{,}700$ kips. The column design for each story is summarized in Table A-3 assuming a splices above levels 3 and 5.

TABLE A-3 - COLUMN DESIGN SUMMARY

Story/ Tier	P_u, kips	$P_{pr,max}(H/L_{br})$, kips	$Ecl = \sum P_{pr,max}(H/L_{br})$, kips	Required Column Strength P_{uc}, kips	Column Section	Column Axial Compression Design Strength ϕP_n, kips
5	119	140	0	119	W14x48	249
4	238	210	140	378	W14x61	456
3	357	280	350	707	W14x109	1,130
2	476	320	629	1,105	W14x109	1,130
1	595	155	1,105	1,700	W14x159	1,700

Step 6 – Connection Design

The required strength of the brace-to-gusset connection is the adjusted Fuse Configuration strength calculated with EQ 2a and 2b. Alternatively, the adjusted fuse strength may conservatively be taken as $P_{pr,max}$ as illustrated in Step 5 – Column Design. The maximum required axial force in the beam delivered through the end connection and the required shear in the beam was determined in Step 3 – Beam Design. With the axial forces in the brace and beam and the shear force in the beam identified, the connections are designed for these forces using conventional connection design approaches. Refer to Figure A-2 for the forces to be used for connection design.

Step 7 – Story Drift Limit

Expected Inelastic Frame Deformation: Typically, the calculated effective stiffness of all the Brace Assemblies are represented within an analysis model that includes P-Δ effects to determine elastic story drifts, which are amplified in accordance with ASCE 7-16 to determine design story drifts. The stiffness of the Fuse Plates and Brace Assemblies in an analysis model may be represented using many methods, three of which are as follows:

1. Some structural analysis software allows for an axial spring or partial-fixity restraint to be applied to the end of frame elements. In such an instance, model the braces using the member sizes as determined in Step 2 – Brace Design with rotationally unrestrained end conditions ("pinned" ends). Assign the stiffness of the Fuse Plates, $k_{e,fuse}$, taken from Table 1 as a partial-fixity axial restraint at the brace ends where Fuse Configurations occur.
2. Model braces using the member size as determined in Step 2 – Brace Design. At each brace end where Fuse Configurations occur, model a 12-inch long steel element to connect the end of the brace member to the beam-column intersection. Assign the cross-sectional area, $A_{equiv,fuse}$, taken from Table 1 to the 12-inch long steel elements to represent the Fuse Configuration stiffness. The connectivity between the 12-inch long steel elements and the brace member shall be fully restrained in all directions to maintain stability. The connectivity between the 12-inch long steel elements and beam-column intersections shall be rotationally unrestrained ("pinned").
3. Model the brace as an element with modulus of elasticity and cross-sectional area proportioned to provide a stiffness equal to the total effective stiffness of the brace and fuse assembly, $k_{e,eff}$, as determined from EQ 4.

The contribution of axial stiffness of the beams and columns to the inelastic drift of the frame is minimal and P-Δ effects are relatively small for braced frame systems, therefore these effects are neglected for the illustrative purposes of this example, wherein the story drift between level 3 and level 2, Δ_2, is:

$k_{e,brace} = A_{brace}E_s/(L_{wp} - n_{fuse}L_{fuse})$ = (32.0 in.^{2})(29,000 ksi)/((25.5 ft)(12 in./ft) – 2(12 in.)) = 3,290 k/in.

$k_{e,fuse}$ = 2,700 k/in.

$$k_{e,eff} = \frac{1}{\frac{1}{k_{e,brace}} + \frac{n_{fuse}}{k_{e,fuse}}} = 1/(1/(3{,}290 \text{ k/in.}) + 2/(2{,}700 \text{ k/in.})) = 957 \text{ k/in.}$$

$\Delta_{bm\text{-}2} = C_d\, P_u / k_{e,eff}$ = (5)(209 kips)/(957 k/in.) = 1.09 in.

Using the bracing formulas of Table 17-24 of the AISC *Steel Construction Manual, 15th Edition*, the story drift between level 3 and level 2, Δ_2, is determined as:

$$\Delta_2 = \sqrt{(L_{br} + \Delta_{bm})^2 - H^2} - \sqrt{{L_{br}}^2 - H^2} = \sqrt{(305.47\ in. + 1.09\ in.)^2 - 216^2} - \sqrt{305.47 - 216^2}$$

Δ_2 = 1.54 in.

Assuming a Risk Category II structure, the allowable story drift in accordance with Table 12.12-1 of ASCE 7-16 is 2% of the story height. The allowable story drift for each story is determined as:

$\Delta_{a2} = 0.02H$ = 0.02(18 ft)(12 in./ft) = 4.32 in. ≥ Δ_2 = 1.54 in., therefore OK

Final Frame Design

The final frame design including member sizes, Fuse Configuration types, and maximum forces for brace connection and drag beam connection design is shown Figure A-2 below.

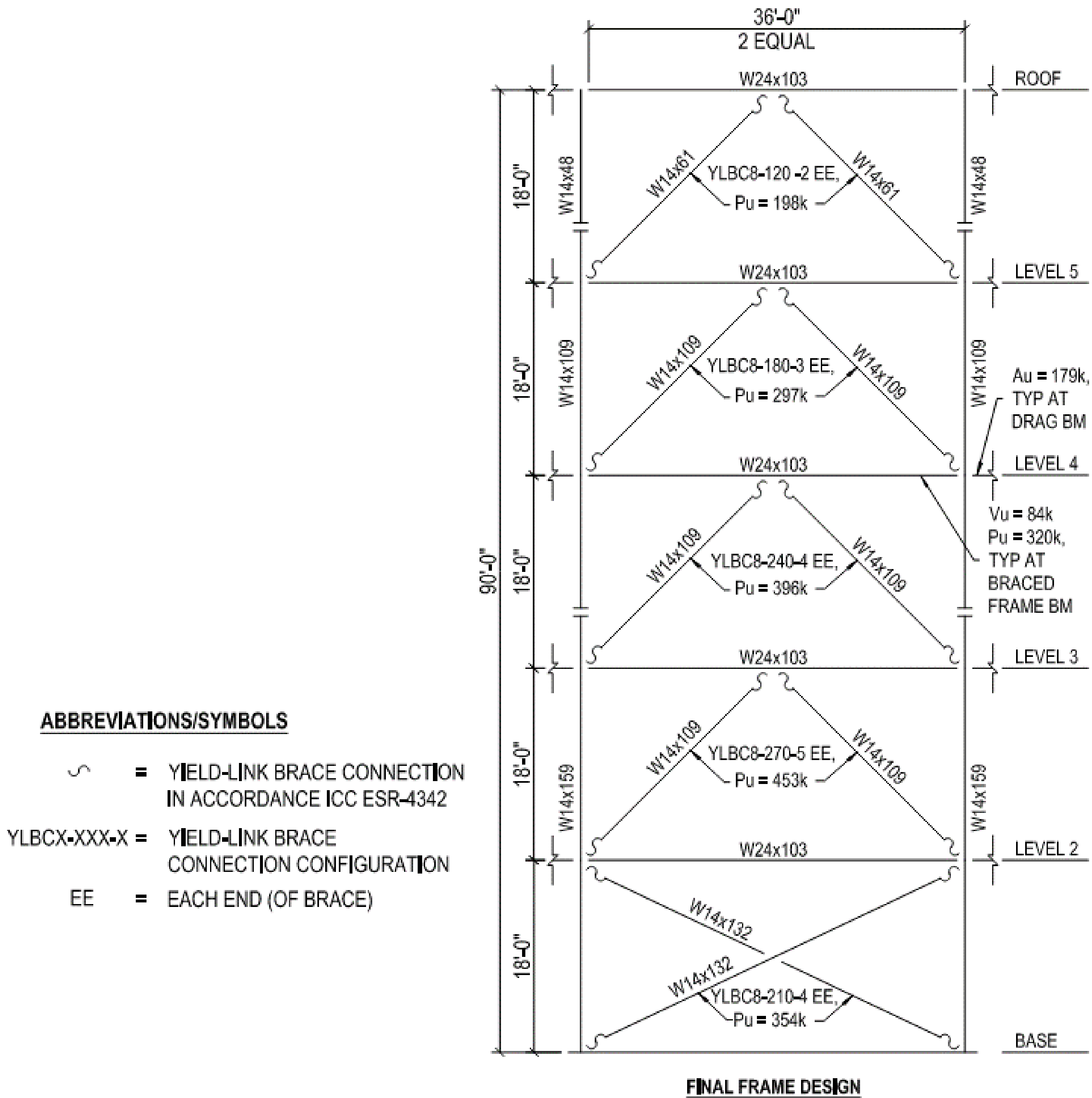


FIGURE A-2 FINAL FRAME DESIGN

ICC-ES Evaluation Report

ESR-4342 LABC Supplement

Reissued March 2024

This report is subject to renewal March 2025.

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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:

YIELD-LINK® BRACE CONNECTION STEEL BRACED FRAME SYSTEM WITH FUSE PLATES

1.0 REPORT PURPOSE AND SCOPE

Purpose:

The purpose of this evaluation report supplement is to indicate that the Yield-Link® Brace Connection steel braced frame system utilizing steel Yield-Link® Fuse Plates, described in ICC-ES evaluation report ESR-4342, has also been evaluated for compliance with the codes noted below as adopted by the Los Angeles Department of Building and Safety (LADBS).

Applicable code editions:

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

2.0 CONCLUSIONS

The Yield-Link® Brace Connection steel braced frame system utilizing steel Yield-Link® Fuse Plates, described in Sections 2.0 through 7.0 of the evaluation report ESR-4342, complies with the LABC Chapter 22 and is subject to the conditions of use described in this supplement.

3.0 CONDITIONS OF USE

The Yield-Link® Brace Connection steel braced frame system utilizing steel Yield-Link® Fuse Plates described in this evaluation report supplement must comply with all of the following conditions:

- All applicable sections in the evaluation report ESR-4342.
- The design, installation, conditions of use and identification of the Yield-Link® Brace Connection steel braced frame system utilizing steel Yield-Link® Fuse Plates are in accordance with the 2021 *International Building Code®* (IBC) provisions noted in the evaluation report ESR-4342.
- The design, installation and inspection are in accordance with additional requirements of LABC Chapters 16 and 17, as applicable.
- Special inspection by Deputy Inspectors shall be provided during the installation of the steel braced frame system.
- The steel braced 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 March 2024.

ICC-ES Evaluation Report

ESR-4342 CBC Supplement

Reissued March 2024

This report is subject to renewal March 2025.

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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:

The Yield-Link® Brace Connection steel braced frame system utilizing steel Yield-Link® Fuse Plates, described in Sections 2.0 through 7.0 of the evaluation report ESR-4342, 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 March 2024.